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Highway Drainage Inlet Research

DEVELOPMENT OF IMPROVED DRAINAGE INLETS PHASE 1: LITERATURE SURVEY

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CIVIL ENGINEERING DEPARTMENT
FRITZ ENGINEERING LABORATORY
HYDRAULICS AND SANITARY ENGINEERING DIVISION

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PHASE 1
LITERATURE SURVEY

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ABSTRACT

A literature survey was made to accomplish the first phase of the research project for development of improved drainage inlets. Hydrologic, hydraulic, and numerical studies were searched, and the important principles are discussed. A more detailed evaluation of the studies of primary significance is also made. References are presented in a classified form. Altogether, 76 references are cited.

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1. INTRODUCTION

Runoff from rainfall must be removed from highways. This is done by placing drainage inlets at intervals at the roadside. The most efficient positioning of the inlets is determined particularly by the drainage capacity of the inlet and by approach conditions. An efficient design should avoid water overflowing the inlet, because sufficient repetitions of overflowage could lead to a possible flooding of the highway. Hydraulic conditions of the flow into inlets cannot be determined analytically. An experimental study must be made using either prototype or model inlets to determine the flow conditions for each particular type of inlet.

The principal objectives of the research would be laid out in the following manner:

Phase 1 - Literature Survey

A search of literature would be made for information pertaining to highway drainage inlets in order to evaluate the results available for applicability to this study, particularly to subsequent phases of the research.

Phase 2 - Determination of Capacity of Inlets

In Phase 2 laboratory tests would be conducted on prototype or model inlets currently in use along highways in Pennsylvania to determine the maximal volumetric rate that each inlet will take before overflowage occurs.

Phase 3 - Determination of Design Criteria for Inlets

Herein the program is to be extended toward the development of design curves or nomographs based on the information obtained in the preceding phase so as to enable design engineers to select the combination of the inlet type and spacing that is to be used for the particular class of highway under consideration.

Phase 4 - Development of Improved Inlets

Phase 4 is to consist in evaluating all the information obtained in the preceding phases, looking toward the possibility of improving inlet efficiency by changing the geometry of the inlet and any other factors that are present near the inlet.

This report is the outcome of Phase 1, Literature Survey. Hydrologic, Hydraulic, and Numerical Studies are the main parts. Several investigations that cannot directly be included in this classification were also presented under Miscellaneous Studies.

Hydrologic studies deal with several methods applied to determine the amount of the runoff that is to be removed by the inlets as a result of the rainfall in the vicinity of the highway. Hydraulic studies consist of the theoretical and experimental investigations conducted to determine the hydraulic behavior and the efficiency of the inlets. Outstanding investigations to date were conducted for several specific types of inlets currently used in different parts of the United States. An all-encompassing representation of the hydraulic behavior of inlets is still lacking. The numerical studies deal with the methods applied to establish mathematically the flow pattern over the highway surface toward inlets. It can be considered as a special application of flood routing techniques in open-channel hydraulics.

A more detailed study was made of the four reports of highest significance in regards to the present research, and is presented as an evaluation.

References cited are given in four groups in accordance with the classification made previously.

Based on the information obtained heretofore, several conclusions can be drawn. However, it is clear that essential information required for the design of a particular inlet can only be obtained by conducting tests on the inlet.

2. DISCUSSION

2.1 Hydrologic Studies

2.1.1 General Remarks

Hydrology is the science dealing with the properties, distribution, and circulation of water on the surface of the land, in the soil, and in the atmosphere. As such, it involves the study of precipitation, evaporation and evapotranspiration, runoff, and ground water. The two phases of the hydrologic cycle that are pertinent to the present study are precipitation and runoff.

Precipitation falling upon the entire right of way of a highway produces runoff from the back slopes or embankments and from the paved surface; thus overland flow is developed in relatively thin sheets over the paved surface. Usually, this runoff is collected in a side-channel gutter at the edge of the pavement, and it is defined away from the roadway by means of inlets as shown in Figure 1. The purpose of an inlet is to divert the storm water away from the highway without damage to the highway. Obviously, the hydrologic study is of extreme importance in the design of drainage inlets. However, the numerous variables involved in such a study make it clear that the hydrology involved is very complex. Be that as it may, most of those variables must be considered in the development of a formulation that is to have practical value.

The most significant variables to be considered in a hydrologic study for a highway drainage inlet are the following:

(1) Rainfall frequency,
(2) Local rainfall intensity,

(3) Size of the drainage area,

(4) Longitudinal and cross slope of the pavement and of the embankments,

(5) Roughness of the road surface and of the embankments,

(6) Nature of the overland flow, whether laminar or turbulent, steady or unsteady, uniform or nonuniform, and

(7) Hydrologic properties of the soil in the embankments, such as infiltration, capacity, erodibility, and granulometry.

Apparently then, a simplified relationship containing a few of the foregoing variables and including all the effects involved is very difficult, if not impossible, to obtain or to develop. Primarily, what is required is to determine the action of the variables over a wide range of conditions.

For the present investigation some of the approximate methods that are most commonly used for the computation of runoff will be considered.

2.1.2 Determination of Runoff from Rainfall

2.1.2.1 Rational Formula

The most widely used statement to determine the runoff from a rainfall is the rational formula, the equation whereof is given as:

$$Q = C\ I\ A \qquad (1)$$

where Q is the peak runoff rate, cfs,

C is the rumoff coefficient,

I is the uniform rainfall intensity, in. per hr., and

A is the drainage area, acres.

The runoff coefficient is dimensionless because the unit, cfs, of the runoff rate, Q, is numerically equal to the unit of the product of I and A, acre-inches per hour, within an error of 0.8%. Hence, the formula is dimensionally consistent, and it is from this fact that the equation is given its name of rational formula.

Although the formula is based on several questionable assumptions, it is quite popular owing to its simplicity. One of the main assumptions, which cannot readily be justified, is that the peak runoff rate is maintained for a time period equal in length to the time of concentration of flow at the inlet. The time of concentration is defined as the time that elapses for a raindrop falling at the most remote distance from the inlet to reach the inlet; it depends on the characteristics of the surface. The reliability of the formula is directly related to the accuracy with which the time of concentration is known[1,13].

The uniform rainfall intensity, I, is a crude approximation, especially for a precipitation that has a short duration. The approximation increases in accuracy as the duration increases. It is determined by a consideration of both the design frequency and the duration of the rainfall, and the determination is a problem of considerable difficulty. Guillou's[3] charts can be considered to be significant contributions in this respect as aids in determining the design rainfall intensity in terms of local rainfall frequency and the duration time, see Figure 2.

The drainage area, A, includes the entire road surface as well as the embankments. Usually, no distinction is made between the different surfaces.

The most important element of the rational formula is the runoff coefficient, C, which is a measure of the imperviousness of the drainage area. Inaccuracies owing to several assumptions and lack of consideration for other characteristics of the drainage area are assigned to this coefficient. However, it is quite obvious that the proper coefficient to use is entirely dependent upon the judgement of the design engineer. Table 1 lists a range of runoff coefficients that are in common use. The rational formula is restricted to small drainage areas.

Table 1 Runoff Coefficients for Different Surfaces and Different Slopes

Type of Drainage Area	Runoff Coefficient, C
Sandy Soil: flat, 2%	0.05-0.10
Sandy Soil: average, 2-7%	0.10-0.15
Sandy Soil: steep, 7%	0.15-0.20
Heavy Soil: flat, 2%	0.13-0.17
Heavy Soil: average, 2-7%	0.18-0.22
Heavy Soil: steep, 7%	0.25-0.35
Slightly Pervious Soil: flat, 2%	0.15-0.40
Pervious Soil with Turf: flat, 2%	0.30-0.55
Impervious Soil or Clay: flat, 2%	0.40-0.65
Gravel or Macadam Pavement	0.35-0.70
Concrete Pavement	0.70-0.90
Asphalt Pavement	0.80-0.95

Izzard[6] has developed an empirical method for determining the volumetric rate of flow, which is similar to that present in the rational formula. However, the Izzard procedure differs from the rational formula in that it deals with the temporal variation in the runoff hydrograph which variation is not considered in this study because the maximal flow rate is of greatest importance.

2.1.2.2 Horton's Formula

Horton[1] developed a formula for determining runoff in the case of turbulent flow using an entirely different approach. The rainfall intensity is assumed to be uniform, I, as in the rational formula. The resulting runoff is given by:

$$q = I \tanh^2 \left[0.922t \left(\frac{I}{nL}\right)^{1/2} S^{1/4}\right] \qquad (2)$$

where q is the runoff rate, cfs/acre or in. per hr.,

I is the uniform rainfall intensity, in. per hr.,

t is the elapsed time since the beginning of the rainfall, min.,

L is the effective length of flow, ft.,

S is the average surface slope, percent, and

n is the retardance coefficient which depends on the surface roughness.

Determining the retardance coefficient is very important, being somewhat similar to the runoff coefficient of the rational formula. Table 2 is a list of different surfaces and their concomitant retardance coefficients. The Horton formula has been widely used by the U S Corps of Engineers particularly for the design of drainage inlets along airport runways.

Chow[1] comments that the Izzard method, which is applicable only to laminar-flow conditions, seems to be much more reliable than either the rational formula or the Horton formula. The latter formula is widely used by the Corps of Engineers. The exponent of each quantity in Equation (2) depends on the nature of the flow; the exponents that are given are considered reliable for the conditions wherein 75% of the flow is turbulent. Different exponents, however, give the runoff hydrograph for laminar, for transitional transition, or for fully-developed turbulent flow[1].

Table 2 Retardance Coefficients for Different Surfaces[1]

Surface Characteristics	Retardance Coefficient, n
Smooth Pavement	0.02
Bare, Compacted Soil Free of Stone	0.10
Sparse, Grass Cover, Moderate Surface	0.30
Average Grass Cover	0.40
Dense Grass Cover	0.80

2.1.3 Summary

Among many formulas the rational formula is the one that is most widely used. The success of its application is directly dependent upon the choice of the runoff coefficient, C, which depends upon the characteristics of the drainage area. Another limitation of the rational formula is that it is applicable to small drainage basins only. However, it is quite suitable for use in highway design because highways usually have small drainage areas associated with them. The rational formula is applicable only for steady flow or equilibrium conditions, which implies that the peak discharge has been reached. For the unsteady phases of the runoff hydrograph the method can be applied successfully; however, this is beyond the province of the current study.

The Horton formula is restricted in application to at least 75% turbulent flow, although it has been widely used for airports by the U S Corps of Engineers.

2.2 Hydraulic Studies

Investigations conducted to date about the hydraulic behavior of inlets can be placed into two groups. The first group consists of basic research on several types of inlets, whereas the second group deals with properties of inlets, such as rating curves and efficiency. In other words, investigations have been made for the purposes of basic research and of design.

The study made at Johns Hopkins University[10] appears to be the most comprehensive one; in that study a number of different inlets were investigated theoretically and experimentally. Attempts were made to interpret the conditions in terms of the discharge formulas for devices such as orifices and weirs. Experiments were used in developing semi-empirical relationships. Li[16] gives a good summary of the hydraulic theory involved, whereas the detailed information for both the entire series of tests and the theoretical considerations appears in the final report[10].

Elaborate model investigations were made by the U S Army Corps of Engineers[21] to determine the hydraulic characteristics of airfield drainage inlets. Extensive data were obtained pertaining to flow characteristics, efficiency, and rating curves. The data are considered to be applicable to highway drainage as well, due to the essential similarity of the hydraulic conditions.

The laboratory study conducted at the University of Illinois[7] consisted primarily of an experimental determination of interception characteristics of four standard inlets used by the Illinois Division of Highways. Theoretical analyses related to the efficiency of the inlets were also made in addition to the full scale model tests. The laboratory investigations made by Larson et al.[11] and by Cassidy[4] are of the same nature, consisting of model tests conducted to determine the efficiency of certain inlets.

2.2.1 Theories of Inlets

2.2.1.1 General Remarks

After the design discharge for a drainage inlet has been determined, as through an hydrology study, the hydraulic design of the inlet can be made; the first step whereof is investigating the hydraulic characteristics of the inlet.

Some inlets have grates, whereas others do not. The main function of the grates is to prevent debris that is being carried by the storm water from entering the inlet and clogging the drainage system.

An inlet can be classified according to position as a side, a drop, or a combined inlet. Side inlets are placed vertically along the curbs and usually have grates. Drop inlets, on the other hand, are placed in side channels of highways at relatively flat angles. Combined inlets are commonly used, the side and drop inlets being placed together.

2.2.1.2 Inlets without Grates

2.2.1.2.1 Drop Inlets

Assuming a uniform velocity distribution for the flow in a side channel, considering a freely falling body, would yield the following relationships (Figure 3)[16]:

$$L_o = V_o \sqrt{\frac{2D_o}{g}} \tag{3}$$

and

$$\frac{D}{D_o} = \left(\frac{L}{L_o}\right)^2 \tag{4}$$

where L is the actual length of the inlet, ft.

L_o is the theoretical length of the inlet required to catch the entire flow, ft.

V_o is the average velocity of flow in the side channel, ft. per sec.

D is the actual depth of water flowing into the inlet, ft.

D_o is the depth of flow in the side channel, ft.

g is the gravitational acceleration, ft. per sec.^{2}

Hence, if both the velocity and the depth of flow in the side channel are known, the ideal length of the inlet, L_o, can be computed. If the inlet has a length, L, smaller than the ideal length, L_o, the total amount of water flowing in the side channel cannot be caught by the inlet. The depth of water caught can be computed from Equation (4). The discharge into the inlet for a side channel flow of width, W, is given by:

$$Q = W\ D_o\ L\sqrt{\frac{1}{2} g\ y_o} \tag{5}$$

2.2.1.2.2 <u>Side Inlets</u>

The hydraulics of the flow into a side inlet is essentially similar to the flow into a drop inlet (Figure 4)[16]. However, water motion is maintained by the component of the gravitational acceleration in the direction of flow, or:

$$a = g \cos \theta_o$$

where θ_o is the angle between the bottom of the side channel and the vertical, as given in Figure 4.

The width of the flow in the side channel is given by:

$$W_o = y_o \tan \theta_o \tag{6}$$

where y_o is the depth in ft. of flow at the edge of the side channel, given in Figure 4.

Thus the following relationship, analogous to Equation (3), would hold:

$$L_o = V_o \sqrt{\frac{2y_o \tan \theta_o}{g \cos \theta_o}} \tag{7}$$

where L_o and V_o are defined the same as in the case of the drop inlet. Because the discharge in the side channel is given by:

$$Q_o = V_o \left(\frac{1}{2}\right) y_o \cdot y_o \tan \theta_o$$

$$= \frac{1}{2} V_o y_o^2 \tan \theta_o \tag{8}$$

substituting V_o from Equation (7) into Equation (8) leads to:

$$\frac{Q_o}{L_o\ y_o \sqrt{g y_o}} = \sqrt{\frac{\sin\ \theta_o}{8}} \tag{9}$$

For side channels used in common practice transverse grades are small, thus $\theta \approx 90^\circ$ or $\sin\ \theta_o \approx 1$. Hence, the following can be written:

$$\frac{Q_o}{L_o\ y_o \sqrt{g y_o}} = \frac{1}{\sqrt{8}} = 0.35 = K_T \tag{10}$$

In the development of Equation (10) the effect of the friction has been neglected. Therefore, $K_T = 0.35$ is only a theoretical constant, and an empirical coefficient should be applied to the equation. Experiments conducted by Li[16] have shown that this coefficient is roughly a constant, or:

$$\frac{Q_o}{L_o\ y_o \sqrt{g y_o}} = K_E \tag{11}$$

$K_E = 0.20$, for $\tan\ \theta_o = 24$ and 48, corresponding to a transverse slope of the side channel of 1/2 and 1/4 inch per foot, respectively; and $K_E = 0.23$, for $\tan\ \theta_o = 12$, corresponding to a transverse slope of 1 inch per foot.

If the actual length of the inlet L is less than the ideal length, L_o, the following relationship holds, which is similar to Equation (4):

$$\frac{b}{y_o\ \tan\ \theta_o} = \left(\frac{L}{L_o}\right)^2 \tag{12}$$

where b is the width of flow captured by the inlet of length, L, with a cross-sectional area, A, as shown in Equation (13):

$$A = y_o b - \frac{b^2}{2 \tan \theta_o} \tag{13}$$

Hence, the discharge, Q, captured by the inlet is:

$$Q = \left(y_o b - \frac{b^2}{2 \tan \theta_o}\right) V_o \tag{14}$$

The ratio of actual discharge to ideal discharge is then:

$$\frac{Q}{Q_o} = \frac{y_o b - \dfrac{b^2}{2 \tan \theta_o}}{\dfrac{y_o^2 \tan \theta_o}{2}}$$

$$= 2 \left(\frac{L}{L_o}\right)^2 - \left(\frac{L}{L_o}\right)^4 \tag{15}$$

by using Equation (12). If the actual length of the inlet is more than 60% of the ideal length, that is, $\frac{L}{L_o} \geq 0.6$, then Equation (15) can be approximated by the following relationship:

$$\frac{Q}{Q_o} = \frac{L}{L_o} \tag{16}$$

The errors introduced by this approximation are minimal*. This implies that as long as the carry-over discharge is within 40% of the total

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*For example: for $L/L_o = 0.6$, $\frac{Q}{Q_o} = 2 \cdot (0.6)^2 - (0.6)^4 = 0.59$; so the error is 1.7%. For $L/L_o > 0.6$, the error becomes smaller.

discharge, the capacity and the length of the inlet are proportional to the ideal capacity and length. This result can be used in Equation (11) to give:

$$\frac{Q}{L\, y_o \sqrt{g\, y_o}} = K_E \tag{17}$$

where K_E was determined by Li[16] and was previously discussed.

2.2.1.3 Inlets with Grates

2.2.1.3.1 Drop Inlets without Local Depression

Drop inlets usually have grates for the purposes of safety and of retaining debris carried by the runoff. Longitudinal placement of the bars is quite a common practice. However, the bars might also be placed at an angle to the main axis of the highway. Li[16] investigated two types of grate inlets with longitudinal bars namely, without and with local depression.

The most general case of the grate inlets without local depression is the one for which both pavement and side channel carry water. Such an inlet is placed on a slope that is steeper than that of the highway. This type is commonly called an undepressed grate inlet or straight gutter inlet.

A grate inlet might have a given length, L, (Figure 5) which can be determined according to Equations (3) and (4). However, under such condictions carry-over flow occurs. In fact, carry-over flow can take place in three forms: (1) q_1, the flow past the inlet between th curb and the first slot which is practically negligible; (2) q_2, the flow outside the last slot; and (3) q_3, the flow past acróss the bars of the grate itself. Therefore, the ideal length of the inlet should be such that the entire carry-over flow consisting of q_1, q_2, and q_3 are eliminated, as is now discussed.

q_1: It was pointed out previously that q_1 is negligible.

q_2: Referring to Figure 5, one might carry out the following computations to find the necessary length, L_o', to eliminate q_2 (16). The flow outside the last slot is similar to the flow into a side inlet, as can be observed by a comparison of Figure 4 and Figure 5. Therefore, recalling Equation (11),

$$\frac{Q_o}{L_o\, y_o \sqrt{g y_o}} = K_E \qquad (11)$$

and realizing that the discharge, Q_o, in the side channel is given by:

$$Q_o = \frac{1}{2} V_o\, y_o^2 \tan \theta_o \quad , \qquad (8)$$

then Equation (11) reduces to:

$$\frac{L_o}{V_o}\sqrt{\frac{g}{y_o}} = \frac{\tan \theta_o}{2\, K_E} \qquad (18)$$

Defining L_o', y', θ', and V' for the flow outside the last slot (Figure 5), equivalently as L_o, y_o, θ_o, and V_o for the side inlet (Figure 4), from a similarity consideration to Equation (18), the following can be written:

$$\frac{L_o'}{V_o}\sqrt{\frac{g}{y'}} = \frac{\tan\,\theta'}{2\,K_E} \tag{19}$$

Realizing the convenience of working with V_o rather than V', Equation (19) can be multiplied by $\frac{V'}{V_o}$, to give:

$$\frac{L_o'}{V_o}\sqrt{\frac{g}{y'}} = \frac{V'}{2\,K_E\,V_o}\tan\,\theta' \tag{20}$$

Li[16] has carried out numerous experiments for a variety of conditions with q_2 having a width that ranged from 25% to 100% of the grate width. It was found that the expression, $\frac{V'}{2\,K_E\,V_o}$, remains roughly a constant such that,

$$\frac{L_o'}{V_o}\sqrt{\frac{g}{y'}} = 1.2\,\tan\,\theta' \tag{21}$$

Hence, Equation (21) can be used to determine the length L_o' which will reduce q_2 to zero, if the depth, y', and the average velocity, V_o, are known (Figure 5).

The carry-over discharge, q_2, for an inlet of length, L, less than L_o' can be computed using Equations (11) and (16). Thus the following can be written by analogy:

$$\frac{Q}{Q'} = \frac{L}{L_o'} \tag{22}$$

where L_o' is the length which drains all of the discharge, Q', flowing outside of the last slot. Recalling Equation (11), the following relationships can be developed:

$$\frac{Q'}{L_o'} = K_E \, y' \sqrt{g \, y'} \tag{23}$$

From Equation (22) we have:

$$Q = Q' \frac{L}{L_o'}$$

so that the carry-over is given by:

$$q_2 = Q' - Q$$

$$= Q' \left(1 - \frac{L}{L_o'}\right)$$

or,

$$q_2 = \frac{Q'}{L_o'} (L_o' - L)$$

Thus,

$$q_2 = (L_o' - L) \, K_E \, y' \sqrt{g \, y'} \tag{24}$$

The coefficient, K_E, has been found to be 0.25 for the carry-over conditions rather than 0.20 to 0.23 obtained previously for inlets without grates. Hence, we have:

$$q_2 = 0.25\ (L_o' - L)\ y' \sqrt{g y'} \tag{25}$$

Therefore, once the length, L_o', for full inlet capacity and the depth, y', of carry-over flow are known, q_2 can be determined.

q_3: For the part of flow carried over the bars of the grate itself, the position of the grate bars becomes extremely significant. Recalling Equation (3), it can be written:

$$\frac{L_o}{V_o} \sqrt{\frac{g}{y_o}} = \sqrt{2} \tag{26}$$

However, the reduction in the flow area due to the bars is not usually negligible. Thus above Equation (26) should be written in a more general way as:

$$\frac{L_o''}{V_o} \sqrt{\frac{g}{y_o}} = m \tag{27}$$

where, L_o'', is the required length of the inlet to eliminate q_3. m is always greater than $\sqrt{2}$ and depends on the ratio of the openings to the width of the bars. Li[16] has found that $m \cong 4$ for equal opening and bar widths, $m \cong 2$ for small bar widths, and $m = 8$ for grates with three cross bars put for the reinforcement purposes. Other quantities can be found for m, if experiments are carried out with different caracteristics. Thus, if m is chosen, L_o'', can be obtained from Equation (27).

The carry-over flow, q_3, for the inlets of length, L, smaller than the full capacity length, L_o'', can be estimated, as given by Li[16], as follows:

The depth of flow in the side channel at a distance, x, from the curb (Figure 5) is:

$$y = y_o - \frac{x}{\tan \theta_o}$$

$$= y_o \left(1 - \frac{x}{y_o \tan \theta_o}\right) \tag{28}$$

The required length, L_1, of the inlet to catch completely the flow of this depth should then be, by Equation (27):

$$L_1 = m\ V_o \sqrt{\frac{y}{g}} \qquad \text{or}$$

$$L_1 = \frac{m\ V_o}{\sqrt{g}} \sqrt{y_o \left(1 - \frac{x}{y_o \tan \theta_o}\right)} \qquad \text{or}$$

$$L_1 = m\ V_o \sqrt{\frac{y_o}{g}} \sqrt{1 - \frac{x}{y_o \tan \theta_o}}$$

If $m\ V_o \sqrt{\frac{y_o}{g}}$ is replaced by L''_o, we obtain:

$$L_1 = L''_o \sqrt{1 - \frac{x}{y_o \tan \theta_o}} \tag{29}$$

If the length, L, of the inlet is less than, L_1, as given by Equation (29), part of the flow will not be caught by the inlet. Recalling from Equation (4) that the depth of flow is proportional to the square of the length of the inlet, the depth, y_1, of the flow caught by the inlet of length, L, is:

$$y_1 = y \frac{L^2}{{L_1}^2} \tag{29a}$$

Hence, the depth of flow not caught by the inlet is:

$$y - y_1 = y\left(1 - \frac{L^2}{L_1^2}\right) \tag{30}$$

Inserting y and L_1 from Equations (28) and (29) into Equation (30); we get:

$$y - y_1 = y_o\left(1 - \frac{x}{y_o \tan\theta_o}\right)\left(1 - \frac{L^2}{L''^2\ 1 - \frac{x}{y_o \tan\theta_o}}\right)$$

which reduces to:

$$y - y_1 = y_o\left(1 - \frac{L^2}{L_o''^2}\right) - \frac{x}{\tan\theta_o} \tag{31}$$

Referring to Figure 5, the width, x_o, over which carry-over flow, q_3, takes place can be computed by realizing that the depth of flow not caught by the intake should be zero at a distance, x_o, from the curb. Thus:

$$y_o\left(1 - \frac{L^2}{L_o''^2}\right) - \frac{x_o}{\tan\theta_o} = 0$$

or,

$$y_o\left(1 - \frac{L^2}{L_o''^2}\right) = \frac{x_o}{\tan\theta_o}$$

or,

$$x_o = y_o \tan\theta_o\left(1 - \frac{L^2}{L_o''^2}\right) \tag{32}$$

It is obvious that x_o should be as small as possible to maintain an efficient inlet. In any case the carry-over, q_3, can be computed by integrating Equation (31) over the width of flow associated with the assumed average velocity, V_o, to give:

$$q_3 = \int_o^{x_o} \left[y_o \left(1 - \frac{L^2}{L_o''^2} \right) - \frac{x}{\tan \theta_o} \right] V_o \, dx \tag{33}$$

$$q_3 = V_o \, y_o \left(1 - \frac{L^2}{L_o''^2} \right) x \Big|_o^{x_o} - V_o \frac{x^2}{2 \tan \theta_o} \Big|_o^{x_o}$$

$$q_3 = V_o \, y_o \left(1 - \frac{L^2}{L_o''^2} \right) x_o - V_o \frac{x_o^2}{2 \tan \theta_o}$$

Replacing x_o from Equation (22), we obtain:

$$q_3 = V_o \, y_o \left(1 - \frac{L^2}{L_o''^2} \right) y_o \tan \theta_o \left(1 - \frac{L^2}{L_o''^2} \right) - \frac{V_o \, y_o^2 \tan \theta_o \left(1 - \frac{L^2}{L_o''^2} \right)}{2 \tan \theta_o}$$

$$q_3 = V_o \, y_o^2 \tan \theta_o \left(1 - \frac{L^2}{L_o''^2} \right)^2 - \frac{1}{2} V_o \, y_o^2 \tan \theta_o \left(1 - \frac{L^2}{L_o''^2} \right)$$

thus:

$$q_3 = \frac{1}{2} V_o \, y_o^2 \tan \theta_o \left(1 - \frac{L^2}{L_o''^2} \right)$$

Inasmuch as $Q_o = \frac{1}{2} V_o \, y_o^2 \tan \theta$ we obtain:

$$q_3 = Q_o \left(1 - \frac{L^2}{L_o''^2} \right)^2 \tag{34}$$

Therefore, the carry-over across the bars can be computed from Equation (34) for any inlet of length, L.

2.2.1.3.2 Drop Inlets with Local Depression

In some places drop inlets are mounted slightly lower than the adjacent surface so as to serve more or less as a sink for the runoff. Recently, this has ceased to be practiced in order to have improved safety at the inlets.

An approximate theory has been developed by Li[16]. He claims that, although this method is based on rather crude approximations and significant assumptions, it gives good results for carry-over flows up to 10% of the total discharge in the side channel.

2.2.1.3.3 Side Inlets with Grates

Li's theory for drop inlets with grates is essentially applicable to side inlets with grates, also. Obviously, the gravitational acceleration acts only with one of its components, that is, $a = g \cos \theta_o$. The formulas are essentially similar to the ones developed for the drop inlets with grates. Transformation of the relationships from the drop inlet to the side inlet performed in Section 2.2.1.2.2 is applicable in exactly the same manner. However, the computations for carry-over flows especially would have to be made with extreme care. This detail is considered beyond the present scope.

2.2.1.4 Combined Drop and Side Inlets

It is a common practice to place drop and side inlets at the same location. Such a system usually provides a greater overall efficiency. Combined inlets are considered preferable especially to reduce the carry-over flow and to increase the flushing and self-cleaning action.

Li's[16] experiments have agreed with the previous argument in that the m to be used in Equation (27) is reduced from 4 to 3.3. This results in a smaller length of the inlet being required to drain a certain discharge.

2.2.1.5 Other Types of Inlets

Other types of inlets are also in use. Their discussion is considered to be beyond the present scope. However, the design criteria, developed by Black et al.[3] for circular grate inlets, have been reported to be quite useful. The criteria were developed for use in agricultural drainage. By proper choice of the discharge coefficient involved the formulas might very well be applied to circular inlets for the surface drainage of highways.

2.2.2 Design and Efficiency of Inlets

The efficiency of an inlet is defined as the ratio of the intercepted discharge, Q_i, to the total discharge, Q_o, from the entire drainage area. The efficiency of the inlet depends on many variables; some are related to the inlet itself, whereas others are related to the entire drainage system. The variables of the inlet which influence the efficiency are the geometric characteristics, the inlet openings, and the self-cleaning ability. In the drainage system the variables are the spacing of inlets, the nature of gutter and overland flows, whether laminar or turbulent, steady or unsteady, and the capacity of the drainage system. The effects of some important variables are summarized in the following statements.

2.2.2.1 Geometric Characteristics of Inlets

(a) Inlet Width: The inlet width is measured normal to the direction of flow in the gutter and has very significant effects on the efficiency of the inlet. As mentioned in Section 2.2.1.3.1, carry-over flow can occur in three ways, of which one is water flowing on the pavement side of the grate, which condition results from an inlet of inadequate width. No inlet can intercept most of the discharge without extending well into the path of the flow. The importance of inlet width is best illustrated by an experiment conducted by Larson et al.[11], wherein they used two inlets which differed primarily in overall dimensions. Regardless of the velocity of flow, the inlet with the larger width always had the higher rating curve or the higher capacity.

(b) Inlet Length: Equation (16), $Q_1/Q_o = L/L_o$, shows that an essential factor for the efficient operation of an inlet is the length of the inlet opening. As the velocity of approach increases, the required length of the inlet increases accordingly. A short inlet will cause a large amount of carry-over flow, and the efficiency will decrease very rapidly.

(c) Inlet Openings: Another essential factor for efficient inlet operation is the net clear area of the inlet. This is usually expressed as the ratio of the total width of the grates to the total width of the openings. Inlets with small ratios will have high efficiencies, particularly at high velocities of flow, because of the large openings. If the approach velocity is low, the ratio is less important; instead, the governing factor will be the net clear area of the inlet. Obviously, the most efficient operation obtains if no grating is present at the inlet.

2.2.2.2 Self-Cleaning Ability

The main purpose of a grating at an inlet is to screen the objectionable material from the storm water which then enters the drainage system. The ability of inlet to separate the debris and floating trash from the water markedly affects the efficient operation of the inlet. Only a few tests have been made, none with significant results. This is due to the fact that storm runoff contains a variety of debris and each kind of debris has its own characteristics. A test by Larson et al.[11], wherein the debris was made from paper, showed that an inlet with longitudinal openings had the best ability to remove debris from the water. Such inlets screened out 95% of the debris being carried to them. He also claimed that longitudinal openings performed equally well on water carrying leaves. This is because the large particles were first screened out by the longitudinal openings and bridged the openings between the bars. With an increase in flow rate the depth and velocity of the flow in the gutter increased also. Equation (3) shows that both the increasing depth and velocity require longer inlet to accommodate the flow. As the inlet is lengthened, in like measure the accumulation of trash will be carried farther downstream from the upper end of the inlet. In this way the longitudinal grate has the best self-cleaning ability. Guillou notes that the cleaning ability of smooth parallel bars is better than that of rough parallel bars[7]. As the smoothness of the bars increases the self-cleaning ability increases also.

2.2.2.3 Inlet Discharge System

The system which drains away the water coming from an inlet is also a prominent factor in regard to the efficiency of the inlet. Almost all inlet designs are based on the assumption that the water which falls through the inlet grate is removed rapidly. Usually there is a catchment structure under the inlet wherein water accumulates until the head is sufficient to cause the outflow to the discharge system to equal the inflow from the inlet. This accumulating of the water in the catchment dissipates most of the kinetic energy of the freely falling water. One important problem of optimal inlet design is trying to use this energy in order to eliminate the ponded water. Considerable research has been carried out in order to study that problem, the most important work being that of Guillou[7].

2.2.2.4 Spacing Interval

For a given drainage area and a given design rainfall intensity the runoff coefficient, C, can be determined. Then in the rational formula the design runoff will be directly related to the drainage area, A. In highway surface drainage the width of the drainage area is fixed by the highway system; consequently, the predominant factor that controls the total rate of flow to an inlet is the spacing interval. From the hydraulic theory of inlets, if the total rate of flow is fixed, both the theoretical length and width to catch all the water are fixed as well. Obviously, if the actual inlet is different from the theoretical one, the efficiency will not be 100%. Consequently, for a given inlet different spacing intervals will lead to different inlet efficiencies. According to Larson et al.[11], the inlet spacing is shown by Equation (42):

$$L = \frac{43,200\ Q}{CIW} \tag{42}$$

where Q is the normal or design capacity of the inlet, cfs.,

L is the spacing interval, ft.,

W is the width of the drainage area, ft., and

I is the design rainfall intensity, in. per hr.

This equation is based upon his conclusion that in a series of uniformly spaced inlets all will operate at an equal capacity except the first three or four inlets.

2.2.2.5 Inlet Efficiency

Cassidy(4) investigated the discharge efficiency of six different types of inlets considering the influence of the following dimensionless parameters:

$$Q_i/Q_o = \varphi\left[\frac{V_o}{\sqrt{gD}}, \frac{L}{D}, \frac{D}{W}, \beta, S, S_o\right] \tag{43}$$

where Q_i/Q_o is the discharge efficiency,

$\frac{V_o}{\sqrt{gD}}$ is the Froude number of the flow in the gutter,

L is the length of the inlet, ft.,

W is the width of the inlet, ft.,

D is the depth of flow upstream from the inlet, ft.,

β is a dimensionless characteristic which is assumed to describe completely the geometric configuration of the grate,

S is the cross slope of the gutter, and

S_o is the longitudinal slope of the gutter.

A set of charts was presented in terms of all the previous variables except longitudinal slope. He concluded that it is possible to eliminate the longitudinal slope because it is important only in determining the velocity of flow in the gutter. Generally, the velocity in the gutter can be included in the Froude number, except if the gutter slope is extremely steep, for which condition the longitudinal slope becomes a controlling factor of inlet efficiency. He claims that his nondimensional charts are suitable for any size of inlets. Additionally, he gives a sequence of the most efficient inlets.

Guillou[7] reports an extensive research that has been made at the University of Illinois for the capacity of eleven most commonly used types of inlet grates. Through capacity curves that were developed it is possible to obtain the amount of intercepted flow directly from the total discharge. A series of curves and data involved terms such as the flow depth at any distance from the inlet, the total discharge, and grate characteristics. Thus knowing the hydraulic conditions of approach the type of inlet can be selected by noting its efficiency together with the amount of flow intercepted and drained by the inlet.

Results of Cassidy's[4] and Guillou's[7] investigations are not directly applicable for the inlets commonly in use in Pennsylvania because of their different characteristics.

2.2.3 Summary

Investigations made to date have yielded information for design purposes. Meaningful points are the following items:

1. Maximal efficiency and significant economy can be attained, if 5% to 10% of the flow is allowed to pass over the inlet.

2. The capacity of the inlet is proprotional to the overall dimensions thereof.

3. Inlets with the bars parallel to the direction of flow in the gutter have higher efficiency and self-cleaning ability than inlets, wherein the bars are not parallel to the direction of the flow.

4. Curb or side inlets are inefficient in comparison to drop or grate inlets, except if clogging of an inlet with debris is a serious problem. Combination inlets give the highest efficiencies if appreciable clogging occurs.

5. The efficiency of an inlet decreases as its slope increases.

The foregoing are general points only. However, this information is not enough, and for each kind of inlet specific model tests must be made to attain the optimal design under prescribed conditions.

2.3 Numerical Studies

2.3.1 General Remarks

The surface runoff resulting from the rainfall flowing towards a highway surface drainage system can be described by means of a mathematical model. The runoff from rainfall usually takes place at shallow depths. Overland flow is the name given to this flow. Its main features are that the flow does not necessarily take place in well-defined stream channels, and that the rate of change of runoff in the direction of flow, known as lateral inflow, has to be considered. In hydraulics it is classified as an unsteady, spatially-varied, open-channel flow. If the runoff takes place over relatively impervious surfaces, that is, if the infiltration is negligible, the flow is described by means of the shallow-water equations. Consequently, the runoff from rainfall on highway surfaces can be described by means of a mathematical model consisting of the solutions of the shallow-water equations with appropriate initial and boundary conditions.

Flood routing is the name given to the procedure of computing the unsteady, spatially-varied flow in an open-channel. The shallow-water equations constitute a class of the general equations of flood routing. They are, in general, quasi-linear partial differential equations. Inasmuch as closed form analytical solutions are not available, numerical finite-difference techniques must be applied to obtain approximate solutions of the equations. The convergence of the numerical approximation to the exact solution of the equation must be assumed. The method of computation might be quite different for each particular situation in order to attain this convergence.

In the following the basic principles of flood routing are discussed; the governing equations are derived; the method of solution is compared; and the most appropriate method of solution for the shallow-water equations, which are of the present concern, is explained. Finally, the applicability of the numerical techniques are discussed as to the extent that they are appropriate for the determination of both the runoff over the highway surfaces due to the rainfall and the discharge received by each individual drainage inlet.

2.3.2 Flood Routing Methods

2.3.2.1 Basic Principles

The equations of flood-routing are based on the laws of conservation of mass, energy, and momentum. Owing to the fact that viscous effects are negligible in open-channel flow, the laws of conservation of momentum and of energy become almost identical. The mathematical model, therefore, consists of the mass or continuity and momentum equations, which form hyperbolic partial differential equations together with the initial and boundary conditions based on the flow characteristics. Numerous methods have been used for the numerical solution of these equations by means of finite-difference techniques with the aid of high-speed digital computers. Amein discusses them briefly[1,2,3,4] and classifies them into either the fixed-mesh finite-difference methods or the method of characteristics. In the former the solutions are obtained at fixed, predetermined points in a rectangular grid of time and distance. In the method of characteristics the solutions are obtained along curvilinear characteristic curves in the time-distance plane. Points in the plane at which the solutions are obtained are not known before the computations are made. Amein points out that the method of characteristics has distinct theoretical and practical advantages over the fixed-mesh finite-difference method, in regard to the stability and reliability of the solutions. He applies the method of characteristics with suitable initial conditions.

Baltzer et al.[5], Brakensiek[6], Cunge[10], Isaacson[16], Lawler et al.[18], Lin[22], and Swain[26] offer numerical and computer techniques about this topic. Ragan[25] made a comparison of the experimental and numerical runoff hydrographs and water profiles for several lateral flow distributions, and pointed out the significance of properly choosing the Manning roughness coefficient, n, to duplicate actual conditions. Crandall[9], Fletcher et al.[12], and Hildebrand[15] present the most general numerical techniques; Chow[8] and Gilcrest[13] apply those methods to solve the flood routing equations. Morgali[24] applies the theory of characteristics of differential equations to overland flow; however, he obtains the solutions along the points in a fixed rectangular mesh and not along the curvilinear characteristic coordinates.

2.3.2.2 Continuity Equation

The governing equations have been derived by several writers. The equation of continuity is derived using the notation in Figure 6. For an increment of time, dt, one must have the relationships:

Inflow - Outflow = Change in storage

or,

$$(AV + q\,dx)\,dt - \left(A + \frac{\partial V}{\partial x}\,dx\right)\left(V + \frac{\partial V}{\partial x}\,dx\right)dt = \frac{\partial A}{\partial t}\,dt\,dx, \quad (44)$$

which reduces to,

$$\frac{\partial\,(AV)}{\partial x} + W\,\frac{\partial y}{\partial t} = q\ , \quad (45)$$

with the assumed average velocity, V, over the section area. The symbols are defined in Figure 6.

2.3.2.3 Momentum Equation

The momentum equation, on the other hand, can be derived with reference to Figure 7; that equation states that the resultant of forces on the control section should be equal to the change of momentum within the section with respect to time. Hence, it can be written that:

$$\gamma A \; dx \; (S_o - S_f) - \frac{\partial}{\partial x} (\gamma \bar{y} A) \; dx = \rho A \; dx \left(\frac{\partial V}{\partial t} + V \frac{\partial V}{\partial x} \right) - \rho q \; dx V \qquad (46)$$

Equation (46) reduces to:

$$- \frac{qV}{A} + g \; (S_o - S_f) = \frac{g}{A} \frac{\partial \; (\bar{y} A)}{\partial x} + \frac{\partial V}{\partial t} + V \frac{\partial V}{\partial x} \qquad (47)$$

The principle assumptions made in deriving Equations (46) and (47) are that there is a hydrostatic pressure distribution on the vertical surfaces of a flow element, that V is the average velocity over the entire cross section, that the channel slope is small, and that the momentum and energy correction factors are unity.

For a channel of unit width Equations (45) and (47) reduce to:

$$V \frac{\partial y}{\partial x} + y \frac{\partial V}{\partial x} + \frac{\partial y}{\partial t} = q \qquad (48)$$

$$V \frac{\partial V}{\partial x} + \frac{\partial V}{\partial t} + g \frac{\partial y}{\partial x} = g \; (S_o - S_f) - \frac{qV}{y} \qquad (49)$$

We also have available the total derivatives:

$$dV = \frac{\partial V}{\partial x} dx + \frac{\partial V}{\partial t} dt \qquad (50)$$

$$dy = \frac{\partial y}{\partial x} dx + \frac{\partial y}{\partial t} dt \tag{51}$$

2.3.2.4 Methods of Solution

Equations (48), (49), (50), and (51) form a set of simultaneous, first order, partial differential equations of the hyperbolic type. Manipulating this set of equations eventually results in the following four conditions:

$$\frac{dx}{dt} = V + c \tag{52a}$$

$$\frac{dx}{dt} = V - c \tag{52b}$$

$$dV + 2dc - g\ (S_o - S_f - \frac{qV}{y})\ dt - g\ dx = 0 \tag{53a}$$

$$dV - 2dc - g\ (S_o - S_f - \frac{qV}{y})\ dt - q\ dx = 0 \tag{53b}$$

where $c = \sqrt{gy}$. The momentum term $\frac{qV}{y}$ of the inflow is commonly neglected because it is small in comparison to the momentum of the total flow in the principal direction. From the nature of this set of partial differential equations the solutions are such that they propagate over the time-distance plane, starting from the initial and the boundary conditions, along the characteristic directions given by Equations (52a) and (52b). Equation (52a) gives a family of α-characteristics or forward-characteristics, and Equation (52b) gives a family of β-characteristics or backward-characteristics. Referring to Figure 8, if y and V are known along the x-axis, along an interval, as indicated by initial conditions, that is, at time t_o the unique solution falls into the shaded area determined by the two characteristics. M and P refer to the same point over the domain. Thus the initial conditions along x, that is, the conditions all over the region at $t = t_o$, are propagated along the time $t > t_o$, always remaining bounded by the characteristic lines.

2.3.2.5 Dimensionless Shallow-Water Equations

Equations (48) and (49) for a channel of unit width can be normalized to bring into dimensionless form[21]:

$$\frac{\partial y_*}{\partial t_*} + V_* \frac{\partial y_*}{\partial x_*} + y_* \frac{\partial V_*}{\partial x_*} = 1 \tag{54}$$

$$\frac{\partial V_*}{\partial t_*} + V_* \frac{\partial V_*}{\partial x_*} + \frac{1}{F_o^2} \frac{\partial y_*}{\partial x_*} = k\left(1 - \frac{V_*^2}{y_*}\right) - \frac{V_*}{g_*} \tag{55}$$

The dimensionless parameters are defined as:

$$V_* = \frac{V}{V_o} \; , \; y_* = \frac{y}{y_o} \; , \; x_* = \frac{x}{L_o} \; , \; t_* = t\,\frac{V_o}{L_o} \; , \tag{56}$$

and

$$F_o = \frac{V_o}{\sqrt{g y_o}} \; , \; k = \frac{S_o \, L_o}{F_o^2 \, y_o} \tag{57}$$

The Chezy equation is used for friction slope:

$$S_f = \frac{V^2}{C^2 R}$$

V_o and y_o refer to the end of the reach where $x = L_o$.

The dimensionless equations have the advantage of containing only two parameters V_* and y_* instead of five, including g, in the original equations. In general, the numerical calculation can be made based on either the original Equations (48) and (49) or on the dimensionless Equations (54) and (55). The choice between the two usually depends on the nature of the boundary conditions.

2.3.3 Numerical Techniques

2.3.3.1 General Remarks

Numerical solutions of differential equations are based on the principle that derivatives are replaced by finite differences to form algebraic equations. This procedure is extremely difficult for nonlinear partial differential equations. The equations are handled in either exact or approximate forms. In the case of exact equations many complexities related to the convergence, stability, accuracy, and efficiency of the numerical procedures enter into the picture. Although the procedure can be reduced to a set of simple mathematical operations the number of operations is extremely large and objectionable, despite the speed of the digital computers. Therefore, the approximate methods are much more useful and popular than the exact methods. Nearly all approximate numerical methods for the solution of the flood-routing equations are based on the continuity equation. The momentum equation is either neglected or significantly simplified. Quite commonly empirical relationships are substituted for the momentum equation.

The numerical methods for the solution of the exact equations of unsteady flow are classified into direct methods and characteristic methods, each of which is further subdivided into explicit and implicit methods[4]. Direct methods of the finite-difference representation are based upon the exact equations, Equations (48) and (49) in the present context, whereas the characteristic methods are based upon the characteristic forms of the equations, Equations (53a) and (53b). The explicit methods, on the other hand, usually consist of linear algebraic expressions for the finite-difference equations from which the unknowns are evaluated explicitly a few at a time. But, in the case of the implicit methods the finite-difference equations are in the form of nonlinear algebraic equations. Consequently, the unknowns are implicit at each step and cannot be determined explicitly before the end of the computation. Each method can also be carried on with either a fixed, predetermined, rectangular mesh of points for the propagation of the solution or with a characteristic network, that is, the solutions are obtained at the intersections of the characteristic curves. Thus the equations can be solved by any combination of the methods and techniques discussed in the context of the above classification. In most of the literature the name, the method of characteristics, is given to the method of implicit characteristics using a characteristic network; the explicit method refers to the direct explicit method using a fixed-mesh network; and the implicit method applies to the direct implicit method using a fixed-mesh network. These three methods are the ones most commonly used.

Based on an essentially similar classification, Liggett[21] presents a detailed discussion of a variety of methods. He examines all of them, theoretically and experimentally, with respect to the convergence of the finite-difference solution to the true solution.

The conclusions are that both the characteristic and the implicit methods provide stable solutions, the former being much faster than the latter. The explicit method is theoretically stable, but practically it is inflexible with respect to stability and, therefore, not very feasible. Amein[3] claims that the implicit method is the most convenient one of all especially for river flood-routing problems. Liggett[21] on the other hand shows that the method of characteristics is the most convenient one, being the fastest, most stable, and most accurate method for solving the shallow-water equations.

2.3.3.2 Solution by the Method of Characteristics

The method of characteristics was found to be the most suitable general method in that it gave good results over a wide range of the parameters[21]. By making analogy to the wave celerity, if c_* is defined as $\sqrt{y}/F_o$ where y and F_o are given by Equations (56) and (57); then the directional derivatives lie along the α-forward-characteristics and β-backward-characteristics as indicated by:

$$\left(\frac{dx}{dt}\right)_\alpha = V_* + c_* \tag{58}$$

and

$$\left(\frac{dx}{dt}\right)_\beta = V_* - c_* \tag{59}$$

Equations (54) and (55) can then be integrated along the characteristics by dropping the asterisks: writing them in the form

$$(V + c) \frac{\partial}{\partial x} (V + 2c) + \frac{\partial}{\partial t} (V + 2c) = K\left(1 - \frac{V^2}{y}\right) - \frac{1}{y} (c - V) \quad (60)$$

$$(V - c) \frac{\partial}{\partial x} (V - 2c) + \frac{\partial}{\partial t} (V - 2c) = K\left(1 - \frac{V^2}{y}\right) - \frac{1}{y} (c + V) \quad (61)$$

Referring to Figure 9, the solutions at points L and R being known with the application of the trapezoidal rule the solution at point M is given by:

$$V_M = 0.5 \ (V_R + V_L + 2c_L - 2c_R + G_{ML} + G_{MR}) \quad (62)$$

and

$$c_M = 0.25 \ (V_L - V_R + 2c_L + 2c_R + G_{ML} - G_{MR}) \quad (63)$$

where G_{ML} and G_{MR} are abbreviations for certain expressions in terms of known quantities at L and R.

The residual functions can also be determined so as to define the error surfaces; by setting them equal to zero we get the intersection of the solutions along the two characteristics, and hence obtain the solutions u_M and c_M through iteration. At the initial line and at the upstream and downstream boundaries the same procedure is applied with simplification. At the corners of boundary and initial lines the situation is different and complicated, but still an averaging assumption seems to be justified for most purposes(9).

The numerical calculation is usually made in such a way so as to accomplish a second-order accuracy in time or distance increment.

The spacing of the initial characteristics depends on the number of data points specified on the initial line. Six is the least number of initial points used in a meaningful calculation. The data are linearly interpolated for the points where the characteristics intersect the boundaries.

2.3.4 Application to Highway Surface Drainage

If a portion of a highway having drainage inlets is subjected to rainfall, the water accumulates and flows over the pavement in the direction of the steepest slope (Figure 10). The catchment area of any one inlet is that area from which precipitation after falling to the ground flows to that inlet. Two slopes are present on a highway, longitudinal and transverse. Depending on the steepness of each slope, the movement over the surface of each raindrop has a particular path. If the top view of a highway segment is considered, as in Figure 11-c, a raindrop falling at Point 1 starts to flow in a direction between the directions of S_L, the longitudinal slope, and S_T, the transverse slope. While flowing downslope more rainfall joins with it, and the increasing mass of water flows with greater inertia.

At this point the highway surface drained by each inlet must be considered. The simplest configuration is to assume rectangular segments, as shown in Figure 11-a; however, this seems to be true if only the transverse slope is present. Another configuration is the parallelogram, the skewness of which depends on the steepness of each slope, as shown in Figure 11-b. The argument that the surface water should follow straight paths does not seem very probable, particularly, if the increasing inertia, the internal friction of the flow, the lateral mixing, and the overall turbulence of the flow are taken into consideration. The drainage area for each individual inlet might very well have an irregular shape, as illustrated in Figure 11-c. Woo[27] also points out the significance of the effects caused by the impact of the raindrops. However, these arguments might be reserved for considerations of the second-order accuracy. Consequently, for the present purpose a parallelogram seems to be an acceptable configuration for the drainage area of each individual inlet.

Once the rainfall intensity is known the flow over a particular drainage area corresponding to each inlet can then be determined mathematically by solving the shallow-water equations. The routing procedure could be performed either in the form of consecutive one-dimensional strips or by means of a two-dimensional routing technique. The initial and the boundary conditions are of vital importance in any case.

2.3.5 Summary

Unsteady, spatially-varied open-channel flow equations find an important field of application in highway surface drainage problems. Numerical techniques have to be applied to obtain the solutions of these quasi-linear partial differential equations. Among many the method of characteristics appears to be the most suitable one.

2.4 Miscellaneous Studies

Other research is indirectly related to highway drainage systems, such as that dealing with economics and with overland flow with infiltration. If runoff from the backslope is considered in determining the capacity of inlets, studies of surface flow over permeable material, such as soil, are significant. Such investigations, however, are of secondary importance for the present study.

3. EVALUATION

3.1 General Remarks

The topics to be discussed herein are: (1) the results presented by the investigators, (2) the models, (3) the channels, (4) the slopes used in the studies, and (5) the kinds of measurements made.

This portion of the report deals primarily with four publications, those of Larson and Straub (Ref. 11, Hydraulic Studies), USCE (Ref. 21, Hydraulic Studies), Guillou (Ref. 7, Hydraulic Studies), and Johns Hopkins University (Ref. 10, Hydraulic Studies). No attempt is made to substantiate a statement as coming from any one paper; rather the remarks are based on one or more papers. As a result, individual credit is not given in this discussion.

3.2 Results

Usually graphs represent the results of the tests performed by the investigators. The curves have one parameter as the total flow and the other as (1) the grade of the channel that approaches the inlet, (2) the depth of water in the channel, (3) the intercepted flow or the throughflow, or (4) the efficiency. The effect of the cross or transverse slope is considered by having individual curves of some parameters aforementioned for different slopes; no attempt was made to use the transverse slope itself as a parameter of a curve. The cross slope was one of the qualifying items associated with a curve; other such items were the Manning roughness coefficient, type of grate, size of grate, depth of inlet depression, and size of inlet depression. However, each of the foregoing items was not necessarily present on every curve.

At depressed inlets the intercepted flow is significantly greater than at undepressed inlets. The depression commonly used on streets with curbs was 2½ to 3 inches below the grade of the gutter. Additional points of importance are that a combined curb-and-gutter inlet has greater capacity than a gutter inlet alone, but only if the latter is clogged; if the grating holds no debris, the capacity of a gutter inlet is almost the same as that of a combined inlet. Cross bars in a grate will cause a reduction of intercepted flow from what it would be without such bars; in other words, cross bars lead to carry-over flow. If an inlet becomes ponded or if a downstream obstruction backs up the water so that its free surface is above the grate, the inlet intercepts water in accordance with the formula for a weir. No study used a flow of water in the gutter above 6.0 cfs, owing to the thought that very few gutters in streets could possibly take a discharge greater than 6.0 cfs. The capacity of an inlet can be increased by permitting some water to bypass the grate, that is, by permitting carry-over to occur, in other words, the maximal quantity of water that any one inlet can intercept with no carry-over is less than that intercepted if carry-over flow is also present. Grates with round-top bars have virtually the same efficiency as grates with flat-top bars.

The use of the Manning equation in some calculations required data pertaining to the cross section of flow; these data, a number of depths across the stream channel, were more reliably obtained by means of a point gage rather than by means of piezometers. The Manning equation is based on uniform flow, that is, the velocity becomes constant as the water progresses downslope toward the inlet. Nonuniform flow indicates a less quantity of water flowing than that of uniform flow. However, the nonuniformity has such a slight effect upon the flow, particularly with flat cross slopes, that uniform flow was assumed to be present. No forthright, distinct statement was made by an investigator pertaining to the measurement or determination of uniform flow, although inferences can be made from some discussions that the length of flow was adequate to produce uniform flow. In one study where cross flow was present, a small jump or swell occurred where that flow entered the longitudinal flow.

3.3 Model

The ratios of model scales ranged from 1 to ¼, that is, the model:prototype relationship of length ranged from 1:1 to 1:4. The latter ratio was used in one study owing to the fact that a sufficiently high rate of flow was not available in the laboratory for a 1:1 relationship. The Froude Law of similarity is the governing criterion for an open-channel model smaller than the prototype; for a 1:1 ratio that fact can be conveniently overlooked. However, the disadvantage of such a ratio is that any inlet used is quite heavy, some weighing almost 500 lb. Fortunately, this feature can be overcome by the use of wooden models, which are quite satisfactory.

3.4 Channel

Model channels used by different experimenters were not unusually long; the customary range in length was from 20 to 35 feet, and the width about 3 feet, although for one study the model had a size of 67 by 21 feet. The inlet was placed two-thirds of the distance downstream from the entrance to the channel, the assumption being that uniform flow develops within that length. The depth of flow was not measured at the entrance into the inlet, rather it was measured upstream therefrom in order to be certain that the drawdown of the inlet did not affect the measurement. Particularly with large flows, a downward curve of the water surface is present as the water approaches the inlet; consequently, the water surface at that place is not parallel to the surface of the invert which requirement must be met for uniform flow to be present.

Channels have commonly been made of wood; concrete and masonite have also been used. In order to develop a suitable Manning roughness coefficient, coatings have been applied to the channel. One coating was cement mortar. A Manning roughness coefficient appropos the current study can probably be selected from any of several lists available.

The shape of the channel used more than others is that present at the sides of city streets, one side having a gentle slope toward the center of the street and the other being a curb that is almost vertical. Relatively flat slopes on both sides of the inlet center line were not used.

3.5 Slopes

The drainage area over which the water runs off to an inlet has both a longitudinal slope and a transverse or cross slope. In the studies here discussed, the minimal longitudinal slope ranged from 1/8% to 1%, and the maximal longitudinal slope from 2% to 10%. The minimal cross slope was 1.5%, and the maximal was 8.3% or 1:12. In two investigations the same slope was maintained throughout the studies. In another, merely the capacity of different inlets was determined with no attempt being made to consider the cross slope; actually no cross slope was mentioned in the report. The ranges of slopes used by the investigators ensued from the particular roadway-drainage system under investigation.

3.6 Measurements

In a study such as is here considered three volumetric rates of water flow must be known; they are, (1) the water coming to the grate inlet, (2) the intercepted water or the water flowing through the inlet, and (3) the carry-over or the water flowing past the inlet The inflow in each case was determined by a meter in a pipe line; such a device was either an elbow, a venturi, a propeller, or an orifice meter. The propeller unit was used at flows above 3.0 cfs. Of the three quantities of water desired, the usual procedure is to measure any two, and to obtain the third by arithmetic. In one study a fourth quantity was determined; that was the flow down the cross slope. The intercepted flow as well as the carry-over were individually measured by means of either a weir or a volumetric tank. The maximal inflow used was 6.0 cfs.

The profile of the water surface was determined by either piezometers or point gages, and the latter gave the more reliable results as indicated by means of the Manning equation.

The mean velocity in a channel was measured in one case by means of a pitot tube which was placed at the standard depths as followed by the USGS, the reading obtained from the instrument being either at the 0.6 depth or at the 0.2 depth and the 0.8 depth, the average of the latter two giving the mean velocity. In another study the pattern of the velocity of the water entering the inlet was obtained by using a midget current meter.

3.7 Comments

The foregoing sections of EVALUATION summarize significant studies that have been made of flow in channels into inlets that are in the bottom and/or at the side of the channel. Some of the information, such as that dealing with procedures and techniques, is pertinent to the study of Pennsylvania highway drainage inlets.

Other material is of use as background information only because most of the work has been done primarily on flow into inlets that are installed along curbs, such as are customarily used on city streets, whereas little has been done in regard either to inlets installed along the edge of a highway where both banks of the channel are relatively flat slopes or to inlets along the center of a median separating traffic lanes. Consequently, in order to understand fully the capability of any one inlet, it really should be tested under different conditions of flow.

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SUMMARY

Phase 1 of the project was a Literature Survey. Research conducted to date was classified as Hydrologic, Hydraulic, Numerical, or Miscellaneous Studies. Main conclusions are herewith listed:

1. The rational formula is the one that is most widely used to determine the volumetric rate of runoff from rainfall. The success of its application is directly dependent upon the choice of runoff coefficient, C, which depends upon the characteristics of the drainage area. The rational formula is applicable only for steady flow conditions.

2. The Horton formula is restricted in application to at least 75% turbulent flow. It is widely applied to airports by the U S Corps of Engineers.

3. Maximal efficiency of an inlet is attained if a carry-over flow of 5% to 10% is allowed. Efficiency decreases as the slope of the inlet increases. Capacity of an inlet is dependent upon its geometrical characteristics.

4. Maximal efficiency and self-cleaning ability is attained if the bars of the inlet are parallel to the direction of the flow. Unless appreciable clogging occurs side inlets and combination inlets are inefficient.

5. The flow pattern over the highway surface is described by unsteady, spatially-varied open-channel flow equations. Numerical techniques have to be applied to obtain solutions to these quasi-linear partial differential equations. Among the numerous techniques

available the method of characteristics appears to be the most suitable. The initial and boundary conditions are of vital importance in any case.

FIGURES

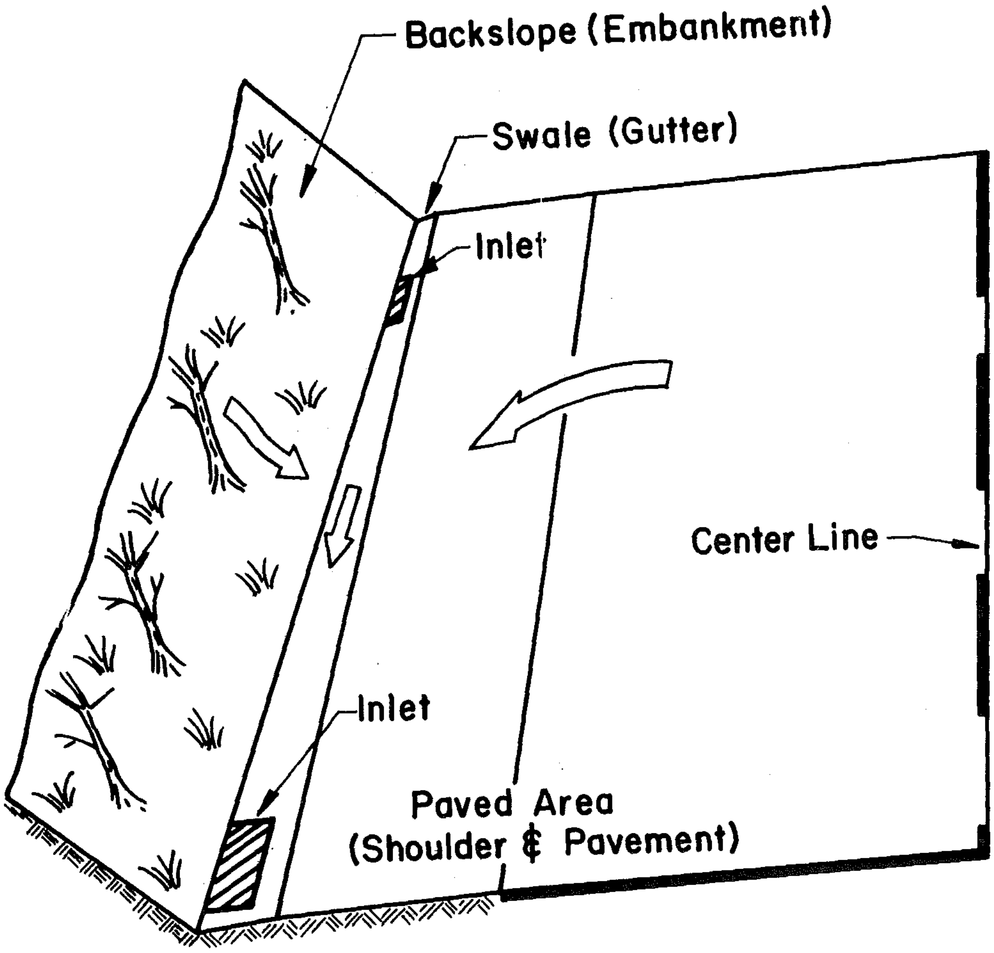

Figure 1 General Layout for Highway Surface Drainage Inlet

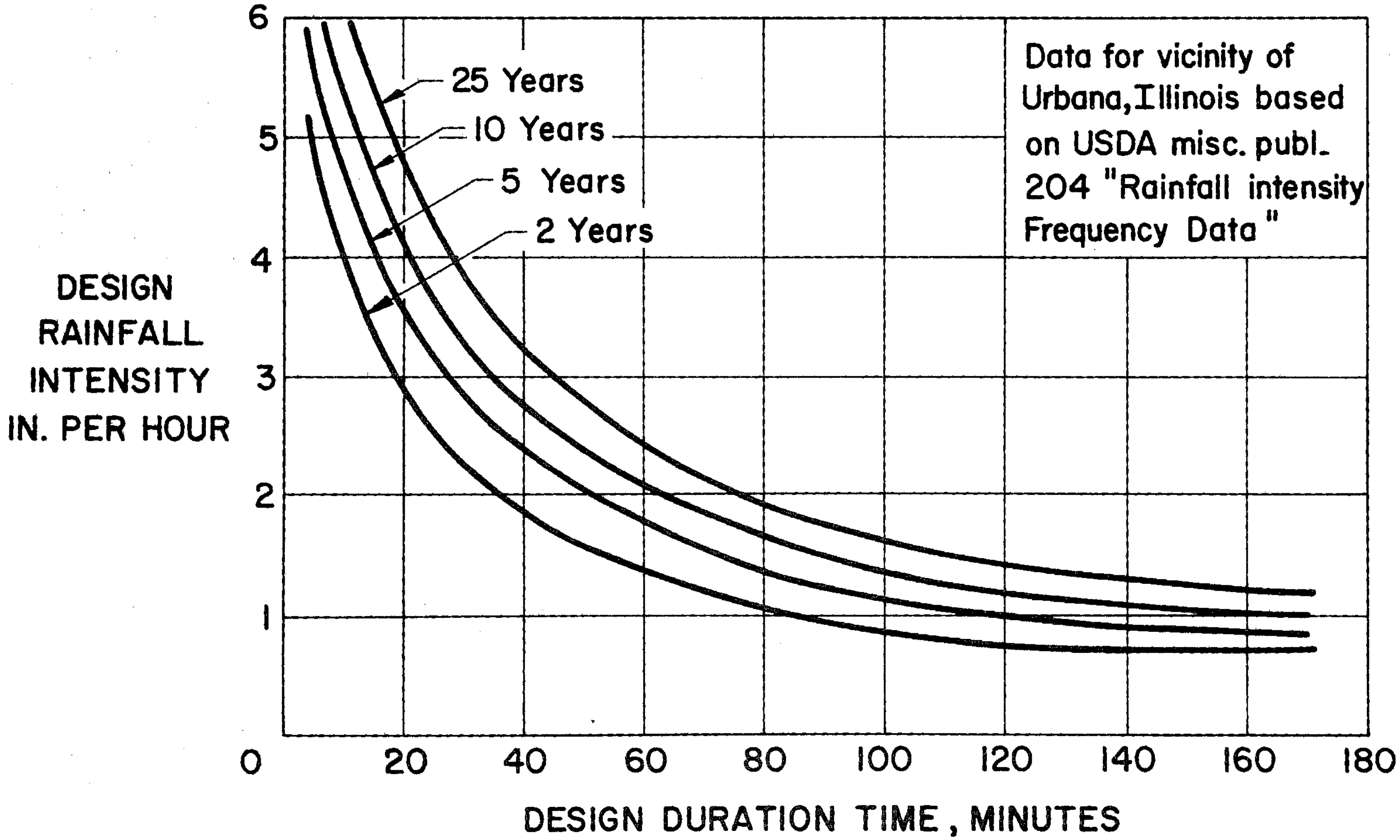

Figure 2 Rainfall Frequency Curves

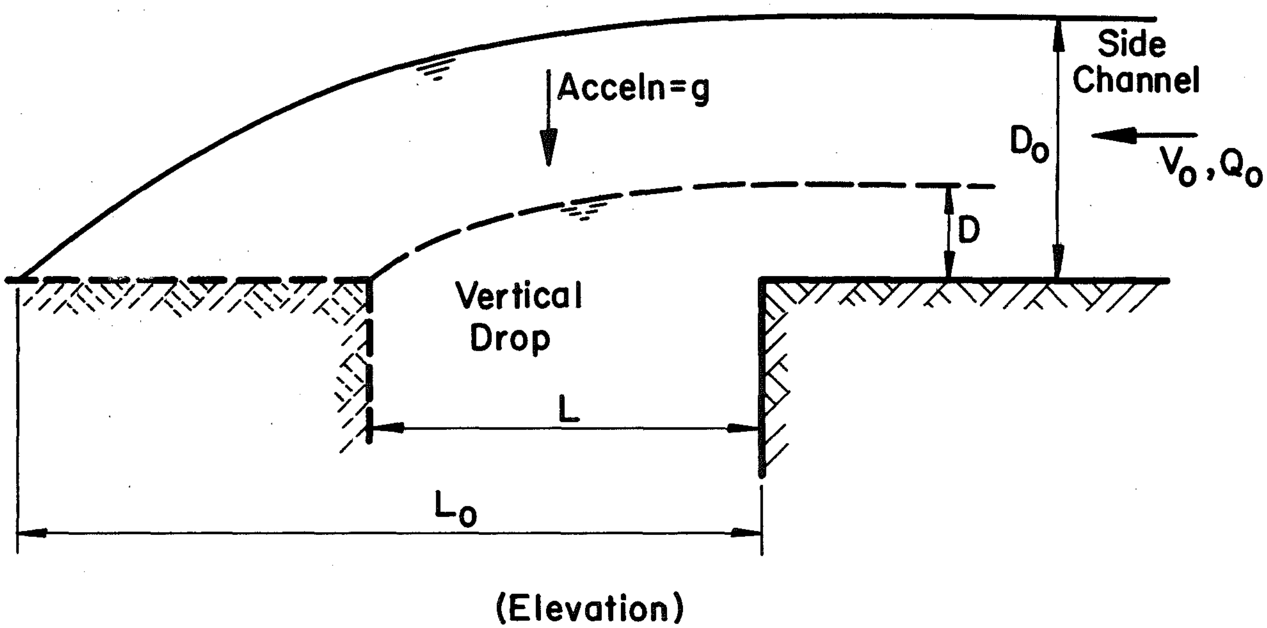

Figure 3 Drop at the End of a Channel

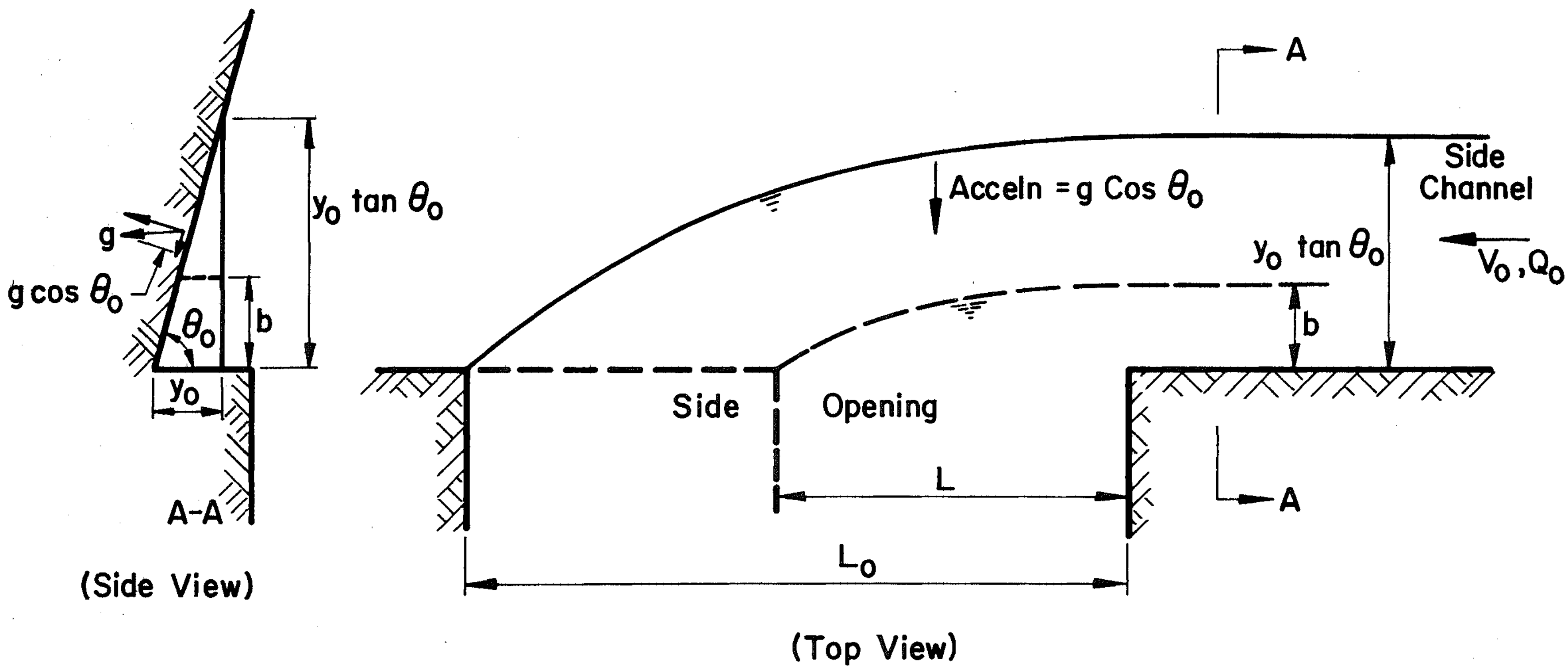

Figure 4 Side Inlet without Grate

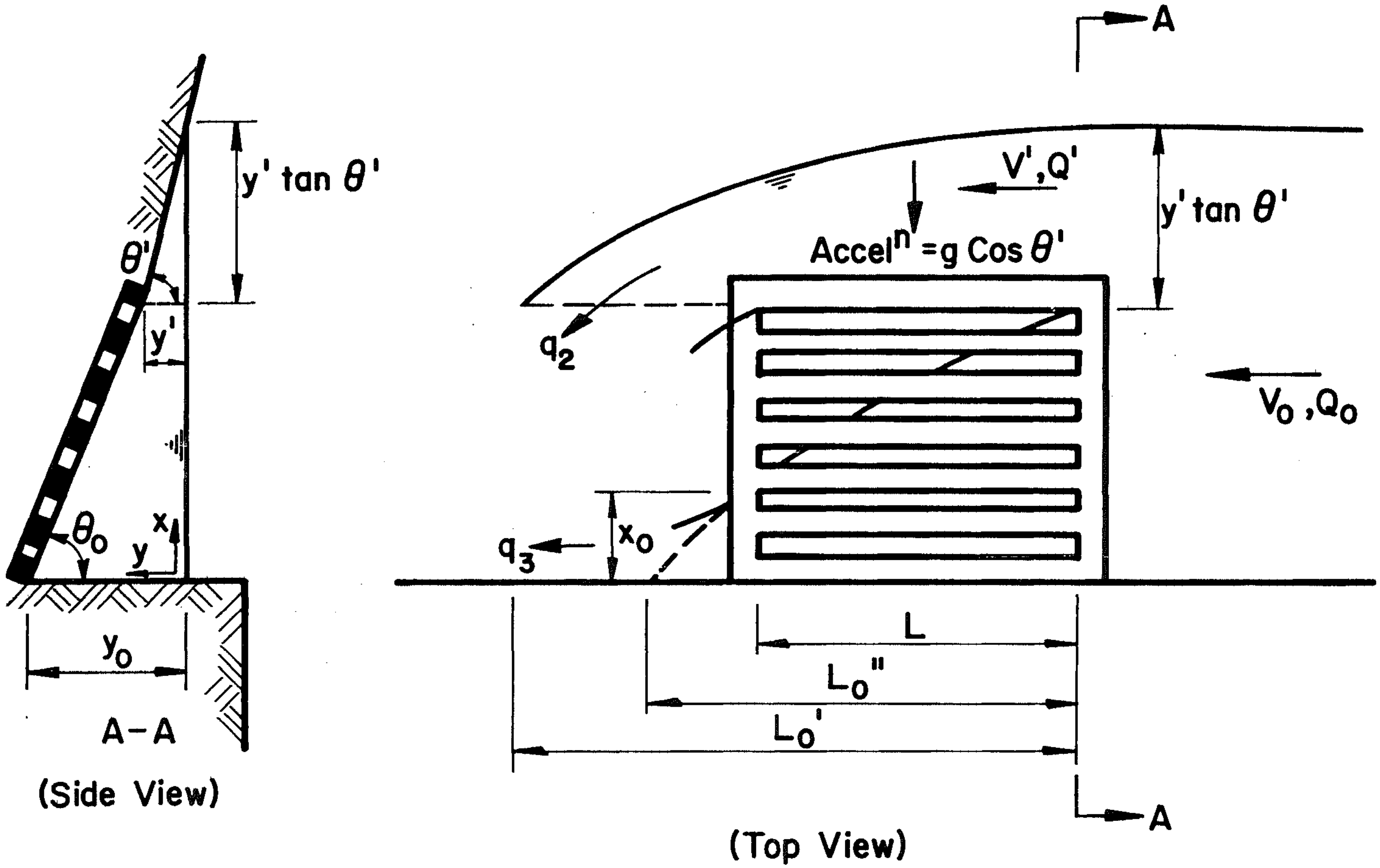

Figure 5 Undepressed Drop Inlet with Grate

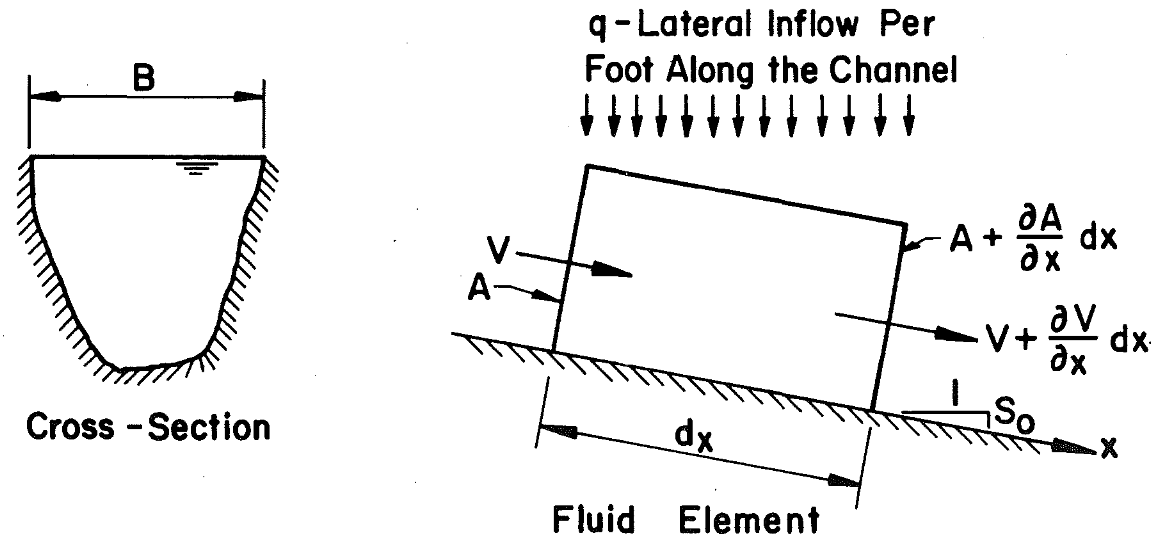

Figure 6 Derivation Sketch for Continuity Equation

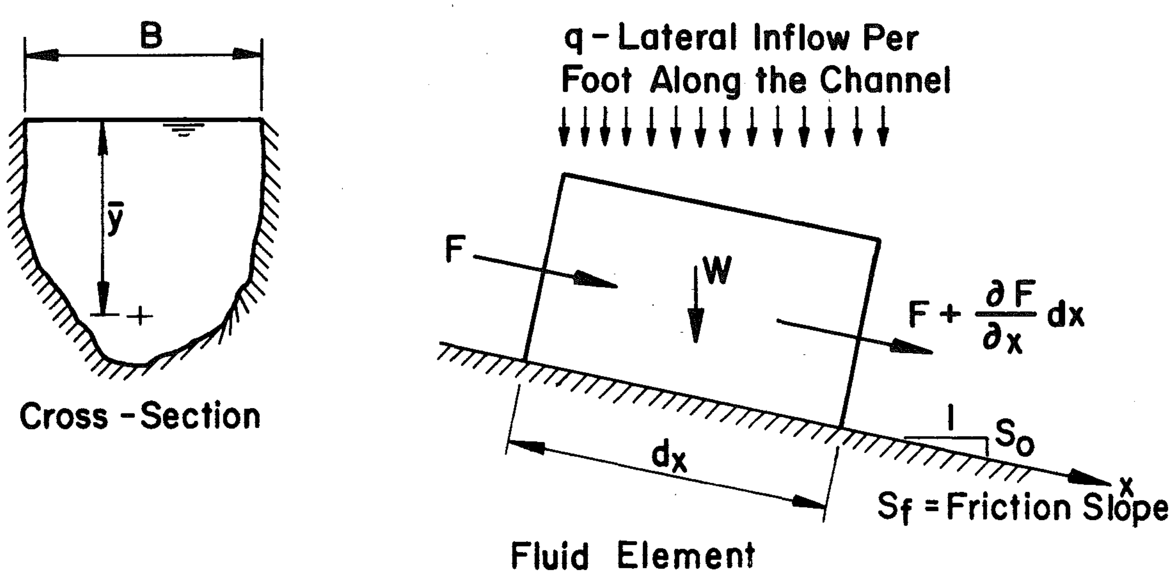

Figure 7 Derivation Sketch for Momentum Equation

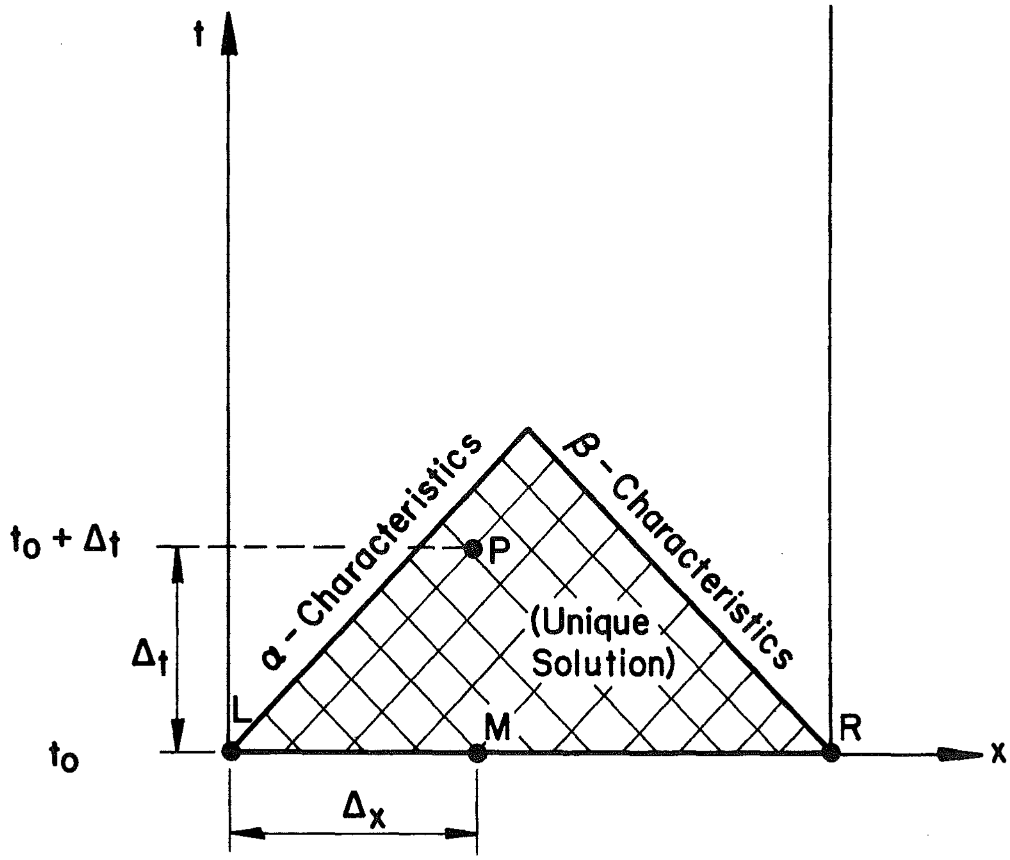

Figure 8 Propagation of Solutions

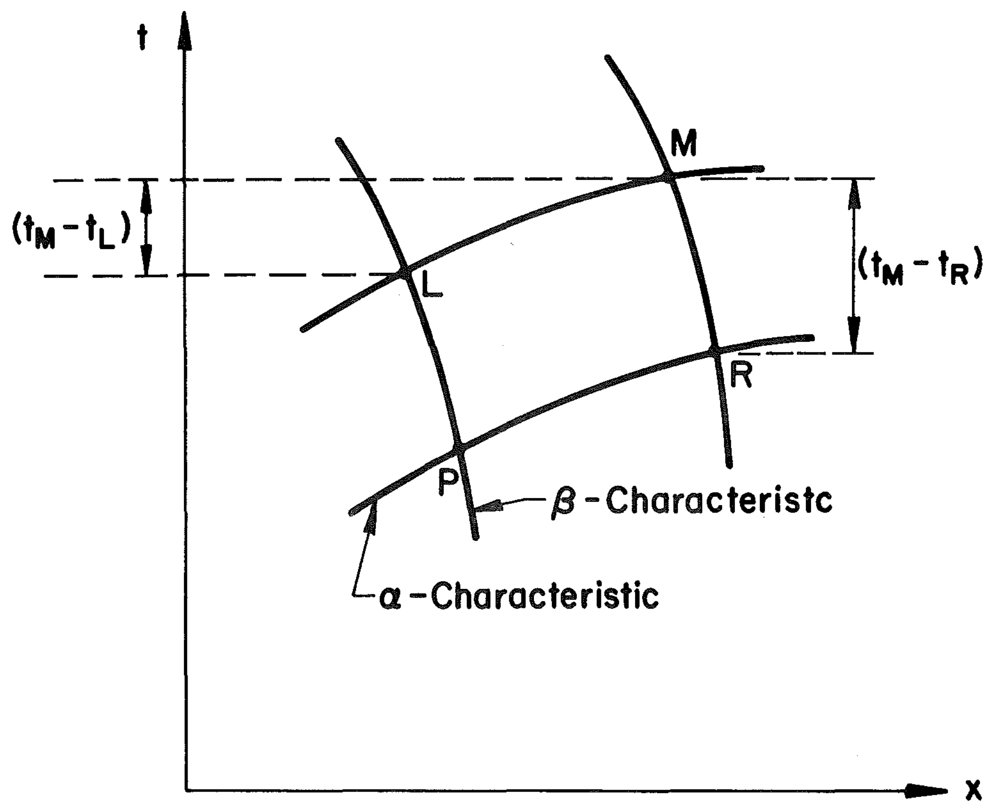

Figure 9 Advancement in Characteristic Method

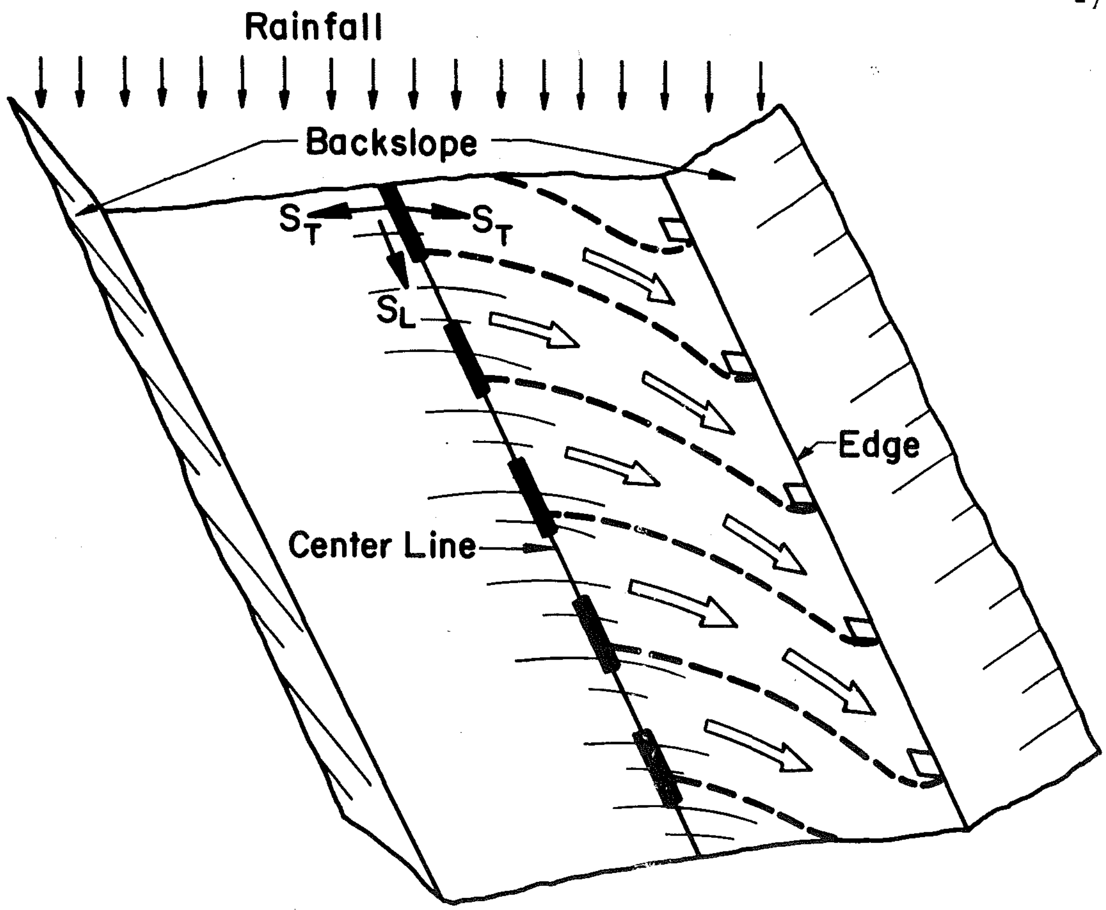

Figure 10 General Flow Pattern on a Highway

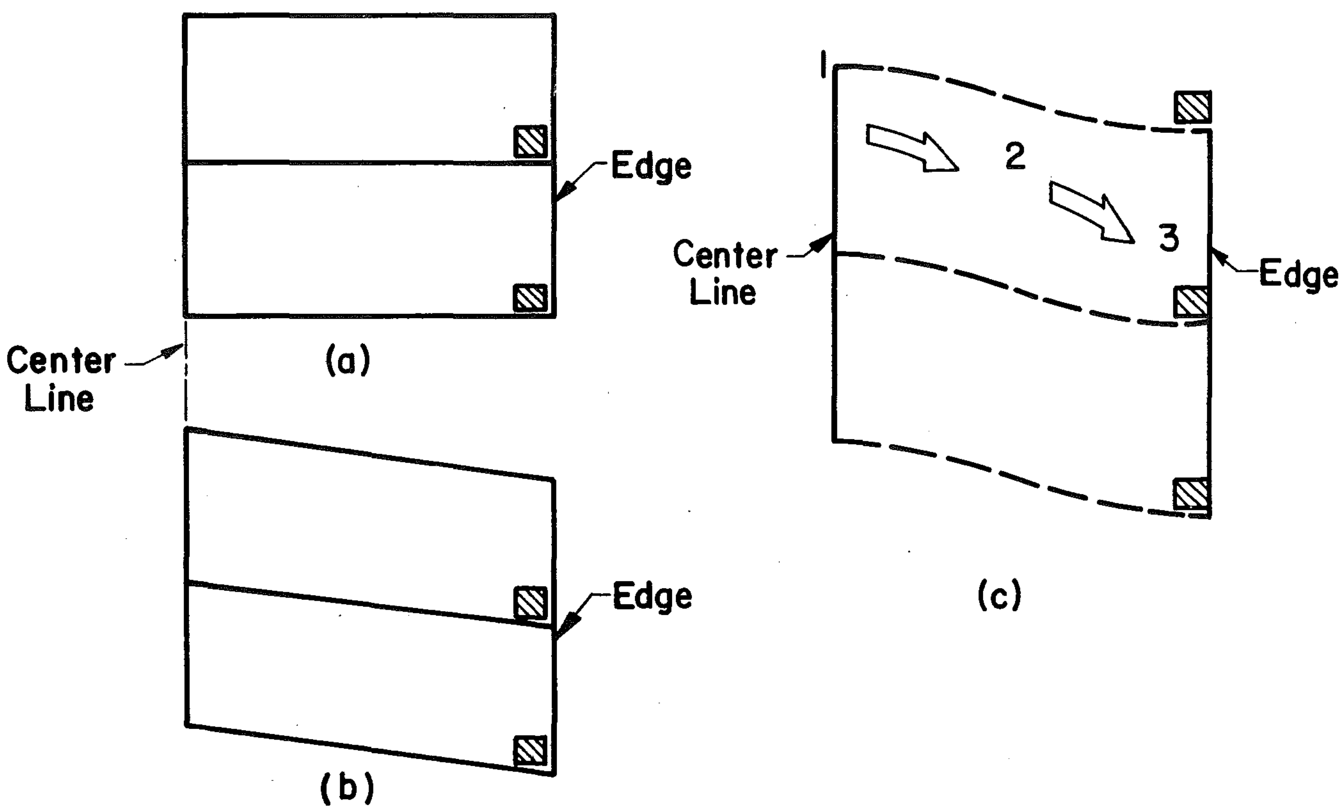

Figure 11 Flow Toward an Inlet along a Highway

NOMENCLATURE

A	drainage area, acres
	cross-sectional area of flow, sq. ft.
b	width of flow caught by the side inlet of length L, ft.
C	discharge coefficient in Chezy equation
	runoff coefficient in rational formula
c	wave celerity, fps.
D	depth of water flowing into the inlet, ft.
F	Froude number
G	abbreviated expressions in numerical procedure
g	gravitational acceleration, fpsps.
I	uniform rainfall intensity, in. per hr.
K	coefficient
k	dimensionless quantity
L	effective length of overland flow
	length of the inlet
n	retardance coefficient
Q	peak runoff rate, cfs.
	discharge, cfs.
q	runoff rate, cfs. per acre, or in. per hr.
	lateral inflow per foot along the channel, cfs. per ft.
	carry-over flow, cfs.
S	average slope

t	elapsed time since the beginning of rainfall, sec.
	time, sec.
V	average velocity of flow, fps.
W	width of flow, ft.
x	distance, ft.
y	depth of flow, ft.
α	characteristic (forward) direction
β	characteristic (backward) direction
γ	specific weight of water, pcf.
ρ	mass density of water, lb-sec^2/ft^4
θ	angle between channel bottom and vertical, degrees

<u>Subscripts</u>

o	conditions at the end section of the channel
	uniform conditions
	average conditions
	ideal conditions
i	instantaneous conditions over the inlet
	flow between the curb and the first slot
*	dimensionless quantity
E	experimental quantity
f	frictional aspect of the energy line
R,L,M	points on the characteristic network
T	theoretical quantity

Superscripts

'	flow outside the inlet
"	flow across the inlet
$\bar{}$	average quantity