probably be pH, alkalinity, and dissolved oxygen; therefore, the ammonia buildup will depend on adjustments of these parameters. If the system is run in the summer without any adjustments in pH and alkalinity, the pH and the alkalinity will decrease, and the ammonia will increase. The winter pH should approach 8.3 and the summer pH should be between 5.5 and 6.0.

In addition to the satisfactory performance of the biological system, the sand filtration system adequately removed the suspended solids and required only infrequent backwashing.

Sodium fluorescein appeared to be an acceptable dye for coloring the flush water in all respects except for its greenish yellow color. It deteriorates in sunlight and is easily removed by activated carbon. Because blue is normally an appealing color, a blue food coloring such as FDC blue No. 1 may be more acceptable. This color can also be removed by activated carbon.

Evaporation as a means of producing zero discharge from a water-reuse system was evaluated by the study of a typical rest area in Virginia that treats 37 900 L (10 000 gal)/d, recycling 90 to 95 percent of the water, and having a final holding pond with a surface area of 500 m^2 (5380 ft^2). The data compiled indicate that, if holding ponds of the size currently used at rest areas in Virginia are appropriately covered, zero discharge is feasible. In addition to solar evaporation, the application of evaporation technology may be an acceptable means for producing zero discharge.

The Virginia Department of Highways and Transportation has constructed a prototype water-recycling system that is now operating at a rest area on I-81 in Rockbridge County. When compared with other alternatives for treating wastewater and conserving water, this system has an estimated saving of $30 000 annually.

CONCLUSIONS

Rest areas can use extended aeration followed by sand filtration in a scheme such as the one shown in Figure 1 to recycle and reuse water for flushing toilets. The system should be capable of recycling 90 to 95 percent of the water used. Water from water fountains and lavatories can provide the 5 to 10 percent of additional water necessary to ensure a steady-state dissolved-solids composition in the recycled water. The system will have a wastage of 5 to 10 percent of the average daily flow; however, evaporation of equivalent volumes may be a means of producing zero discharge. In certain locations, solar evaporation may be used to produce zero discharge.

The wastewater treatment described is applicable to areas with deficient water supplies and to areas where there are problems with wastewater disposal. It will not meet the needs of all rest areas, but in certain locations it can provide a viable alternative to current practice. The system can be added to existing extended-aeration systems, or it can be incorporated in the design of new facilities.

ACKNOWLEDGMENTS

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The opinions, findings, and conclusions expressed in this paper are those of the authors and are not necessarily those of the Virginia Highway and Transportation Research Council or the Federal Highway Administration.

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Simplified Method for Design of Curb-Opening Inlets

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The purpose of this paper is to expand on and simplify the method of designing curb-opening inlets given in the Hydraulic Engineering Circular 12. A reanalysis of the experimental data has shown that the performance of a curb-opening inlet can be represented by a single dimensionless graph of the interception ratio of the curb-opening inlet (Q_i/Q) as a function of the length of the curb-opening inlet (L_i) divided by the product of the Froude number of the flow at the outer edge of the inlet depression (F_w) and the width of the spread of uniform flow in the street. The unit discharge of the inlet, up to a value of Q_i/Q defined by the cross slope alone, conforms closely to the unit discharge of the same inlet for the sump condition if the effective length of the weir crest and same total head are used in the latter case. Above this value of Q_i/Q, the required length of inlet varies as the 0.4th power of the ratio of L_i to 1.65 $F_w T$, regardless of cross slope. A design method is presented that enables computation, with reasonable confidence, of the required length of inlet for any cross slope, any grade, any width of depression, any spread of flow on the pavement, and any pavement roughness. The results agree well with the experimental data on subcritical and supercritical slopes. The analysis disclosed a number of deficiencies in the experimental data. Recommendations for remedying these deficiencies are given.

The data on curb-opening inlets first reported by Bauer and Woo (1) and their subsequent design charts (2) have been widely reproduced in hydraulic design manuals. Unfortunately, these charts are confined to a maximum longitudinal slope of 4 percent, a fixed manning n-value of 0.016, inlet lengths of 1.5, 3.05 4.6 m (5, 10 and 15 ft), and a range of flow spread up to 3.05 m (10 ft).

The original experimental data for supercritical slopes were reported by Karaki and Haynie (3). The experiments were full-scale and made on longitudinal slopes of 0.01 and 0.04 percent and cross slopes of 0.015 and 0.06 percent. Two surfaces having Manning n-values of approximately 0.01 and 0.016 were used. After initial tests to establish an optimum shape for the depression, tests were run with a depression width of 0.6 m (2 ft) (Figure 1) and widths of flow on the street of 1.5 and 3.1 m (5 and 10 ft). For each configuration, tests were begun with an inlet either 0.75 or 1.5 m (2.5 or 5 ft) long, which was then increased in 1.5-m (5-ft) increments to the length required to intercept all of the flow, or to the maximum discharge available in the apparatus.

The data were quite consistent except for a few sets that departed from the norm. In each run, the flow intercepted and the total flow were measured. This paper presents the results of a reanalysis of the original data. The symbols used are defined in Figures 1 and 2 and below.

- a = vertical distance of depression plane of curb face measured from intersection of normal street surface and curb face (feet),
- d = depth of water of uniform gutter flow at curb face (feet),
- F_W = Froude number of flow depth at distance W from curb face,
- L_i = length of curb-opening inlet (feet),
- L_1 = length of inlet when $Q_i/Q = 1$ on first section of curve for Q_i/Q = function of L_i/F_WT (Figure 2),
- L_2 = length of inlet at point of intersection of first and second sections of Q_i/Q = function of L_i/F_WT,
- L_3 = length of inlet when $Q_i/Q = 1$ on second section of curve for Q_i/Q = function of L_i/F_WT,
- n = roughness coefficient in modified Manning formula for triangular gutter flow (Equation 3),
- Q = approach flow to inlet (feet^3/second),
- Q_i = portion of Q intercepted by inlet (feet^3/second),
- Q_i/Q = interception ratio of curb-opening inlet,
- Q_2 = value of Q_i at L_2 (feet^3/second),
- Q_c = portion of Q carried past inlet = $Q - Q_i$,
- S = lontitudinal slope of pavement,
- S_x = cross slope of pavement,
- W = width of depression of curb-opening inlet (feet),
- Q_{comp} = value of Q_i estimated for composite section,
- T = width of spread of uniform flow in street (feet), and
- Z = $1/S_x$.

[SI units are not given for the variables in this model since it was derived for use with U.S. customary units. This nomenclature is the same as that used in the Hydraulic Engineering Circular (2) except for dropping the subscript on S_0 and adding the symbols used in Figure 2.]

The reanalysis shows that the interception ratios (Q_i/Q) can be represented as dimensionless functions of the parameter L_i/F_WT and the cross slope S_x.

The paper first presents the basic equations defining these functions, demonstrates how these equations were derived, and compares the results to experimental data for subcritical and supercritical slopes. It then compares the performance of these equations to those of a weir of the same dimensions, using the latter relation to verify the reasonableness of the interpolations for cross slopes intermediate to those tested, and hypothesizes the performance of untested inlets on street sections having a composite cross section. Finally, a computation table demonstrating the application of the method to practical design problems is given.

GENERAL PERFORMANCE CHARACTERISTICS OF CURB-OPENING INLETS

The performance of curb-opening inlets can be described by a single dimensionless diagram in which the interception ratio (Q_i/Q) is a function of L_i/F_WT (Figure 2). For lengths of inlet less than L_2, Q_i/Q is directly proportional to L_i/L_1, where L_1 is the intercept at $Q_i/Q = 1$. For lengths of inlet greater than L_2, Q_i/Q is proportional to $(L_i/L_3)^{0.4}$, where $L_3 = 1.65\ F_WT$.

The product F_WT is a measure of the gravity force acting on the flow and L_1, L_2, and L_3 are directly proportional to F_WT. In terms of the dimensions of the street section and the inlet,

$$F_w = (0.262/n)[(T - W)S_x]^{1/6} S^{1/2} \tag{1}$$

The term in brackets is the depth of the flow at the outer edge of the inlet depression.

The discharge Q is usually the independent variable, and the width of the flow upstream from the inlet is

$$T = (Qn/0.56S^{1/2})^{3/8} S_x^{-5/8} \tag{2}$$

which is based on the general equation for flow in a shallow triangular channel (2).

$$Q = 0.56(Z/n)d^{8/3}S^{1/2} \tag{3}$$

ANALYSIS OF EXPERIMENTAL DATA

Karaki and Haynie (3) plotted Q_i/Q as a function of L_i/F_WT, with W/T as a third parameter, to demonstrate the effects of geometric variables and surface roughness, but did not develop conclusions.

The analysis reported here is based on plotting Q_i/QL_i as a function of L_i/F_WT as shown in the lower portions of Figures 3 and 4 for cross slopes of 0.015 and 0.06 respectively. The symbols separate the data for each set of runs; L_i is the sole variable.

Q_i/QL_i is constant up to $L_i/F_WT = 0.4$ for $S_x = 0.015$ and $L_i/F_WT = 0.8$ for $S_x = 0.06$. Each curve then breaks downward with a slope of -0.6. If the constant value of Q_i/QL_i is taken from the plot and $Q_i/Q = 1$, a new value of L_i that is now defined as L_1 is obtained. The upper curves in Figures 3 and 4 are then plotted as L_i/L_1 versus the corresponding value of L_i/F_WT. This collapses all the data into a single line whose equation is

$$Q_i/Q = L_i/L_1 \tag{4}$$

L_1 is the x-intercept at $Q_i/Q = 1$.

At the breakpoint, $Q_i/Q = L_2/L_1 = 0.567$ and 0.749 for $S_x = 0.015$ and $S_x = 0.06$ respectively. The value of L_1/F_WT can be computed algebraically as $(1/0.567)0.4 = 0.705$ for $S_x = 0.015$ and $(1/0.749)0.8 = 1.07$ for $S_x = 0.06$.

The two characteristic lengths (L_1 and L_2) are functions of S_x and W. The length (L_3), at which 100 percent interception is attained, is $1.65\ F_WT$.

For lengths less than L_2, the inlet is performing essentially as a weir. The unit discharge is $Q_i/L_i = Q/L_1$ for inlet lengths up to L_2. Beyond that point, Q_i/QL_i varies as $(L_i/F_WT)^{-0.6}$ (Figures 3 and 4). Therefore, Q_i/L_i for a given configuration is constant for $L_i < L_2$ and decreases rapidly for $L_i > L_2$. This fact is economically significant because the cost of added increments of length is not justified by the rapidly decreasing flow increments.

The equations for L_1 and L_2 are

$$L_1/F_wT = 2.79W^{-1/6}S_x^{\ 0.3} \tag{5}$$

$$L_2/F_wT = 3.67W^{-1/6}S_x^{0.5} \quad (6)$$

The ratio (L_2/L_1) is also the interception ratio (Q_i/Q) at which the two curves intersect and is

$$L_2/L_1 = 1.315S_x^{0.2} \quad (7)$$

For both cross slopes and $L_i > L_2$, the exponent of Q_i/QL_i is -0.6, so that the exponent of Q_i/Q for the line to the right of the breakpoint must be (1 - 0.6) = 0.4. The form of the equation is thus

$$Q_i/Q = (L_i/L_3)^{0.4} \quad (8)$$

L_3 is the value of L_i when $Q_i/Q = 1$, or $L_3 = (Q/Q_i)^{2.5} L_i$. If the values of Q_2/Q and L_2 are substituted into this equation,

$$L_3 = 1.65F_wT \quad (9)$$

for both cross slopes.

Since the intersection of the two lines is not exactly on the 0.8 ordinate line in Figure 4, the coefficients in the above equations were adjusted slightly to satisfy Equation 9. The lack of definition is due to the small number of data points for $L_i > L_2$, which in turn is due to the limitation in discharge capacity of the experimental apparatus.

Not all sets of runs were used in Figures 3 and 4 because certain sets departed from the norm for unex-

Figure 1. Graphical definition of symbols.

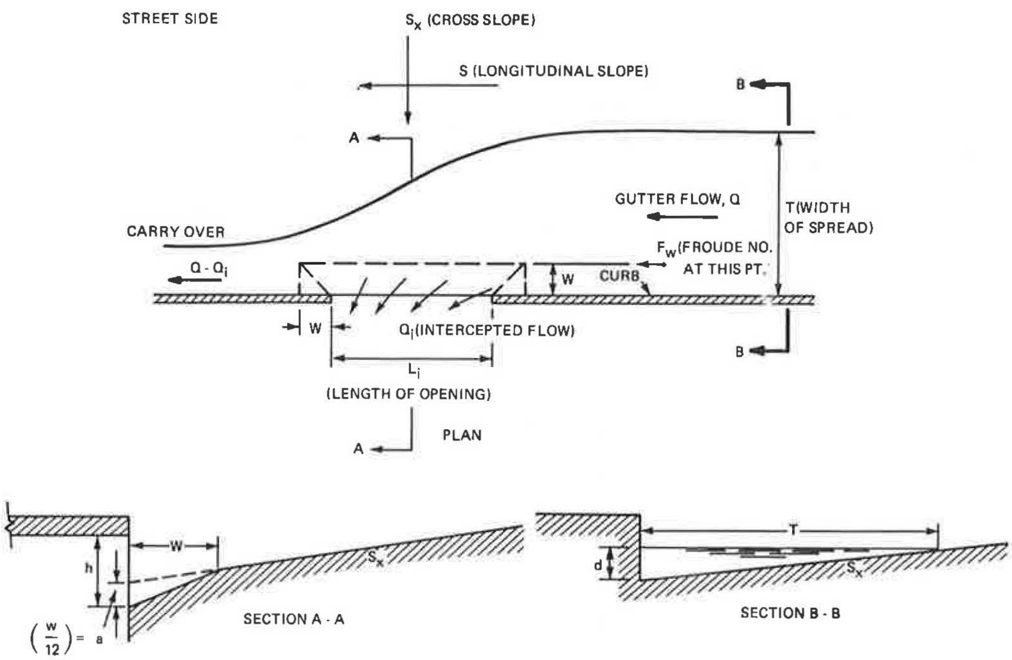
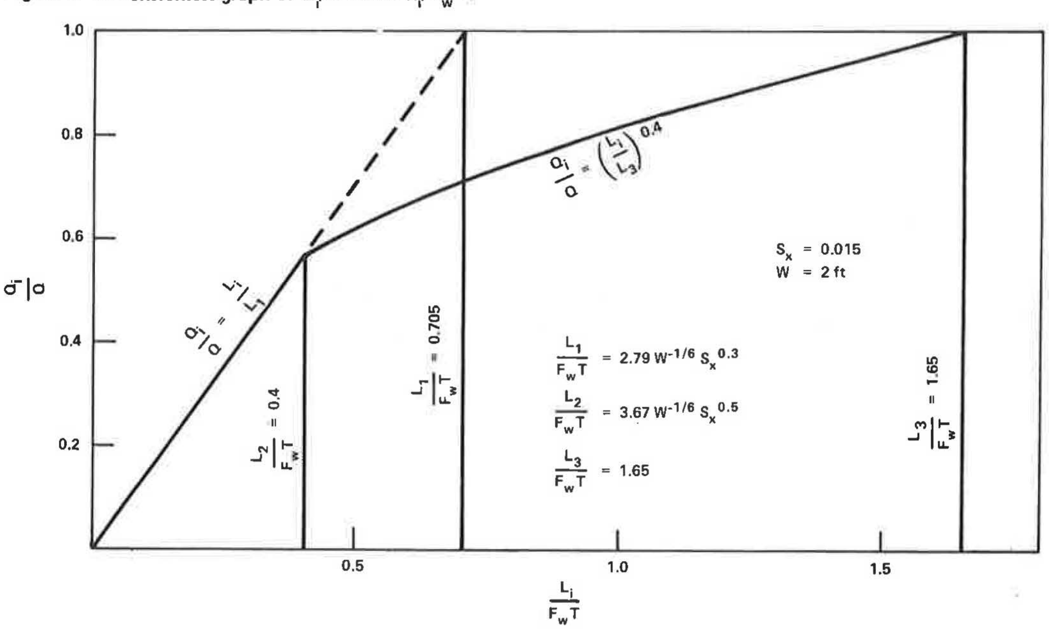


Figure 2. Dimensionless graph of Q_i/Q versus L_i/F_wT.



plained reasons. None of the runs for T = 5 and S_x = 0.015 were usable because there were too few points for $L_t < L_2$. There were only two sets of runs for W = 0.305 m (1 ft) and unfortunately they departed from the norm. To estimate the apparent relationship of W in Equations 5 and 6, the Froude model law was used by taking inlet configurations that were geometrically similar to those of a tested inlet and computing L_1 for inlets of several different ratios and the same W/T. F_W was computed by Equation 1 for the appropriate W in the range from 0.31 to 1.83 m (1 to 6 ft), and F_WT was then computed for each model. L_1 was computed by similitude and divided by the corresponding value of F_WT. This gave a new value of the coefficient in Equation 5, which varied from the initial value in inverse proportion to $W^{1/6}$. This relation was reasonably constant over a range of $0.1 < W/T < 0.3$, but may not be entirely correct and should be verified by additional experiments.

Comparison to Experimental Data

The values of Q_i/Q computed by using Equations 5 and 6 for all runs are plotted against observed values of Q_i/Q in Figures 5 and 6. An enveloping line has been drawn around the sets of data that departed from the norm and were excluded from the analysis. The straight line indicates perfect agreement between computed and observed results.

Figure 7 shows another comparison, that of Q_i as a function of S where S_x = 0.015, n = 0.016, and W = 0.6 m (2 ft). The experimental runs were for n = approximately 0.095 and are plotted at the value of S that has the same value of F_WT as does n = 0.016. This is acceptable because Q_i/Q is a direct function of F_WT for a given L_i. The agreement is reasonably good up to L_i = 10.7 m (35 ft).

Figure 7 also shows values of Q_i that were obtained by the use of the charts given in the Hydraulic Engineering Circular (2). A similar graph was plotted for S_x = 0.06, but is not included here. It also showed good agreement with experimental data, but the Q_i values derived from the charts averaged about 20 percent lower than the computed values.

The experimental data available for subcritical slopes are limited, and until such time as more complete data are available, the equations in this paper may be used.

Change of Q_i With Longitudinal Slope

For a given L_i, Q_i increases as the 0.3 rd power of S until $Q_i/Q = L_2/L_1$, the breakpoint in Figures 3 and 4. For steeper slopes, Q_i is constant for a given length as required by Equation 4. This is supported by the experimental data for L_i = 5.

The incremental change in Q_i/L_i can be calculated by differentiating Equation 8.

Figure 3. Experimental data (S_x = 0.015).

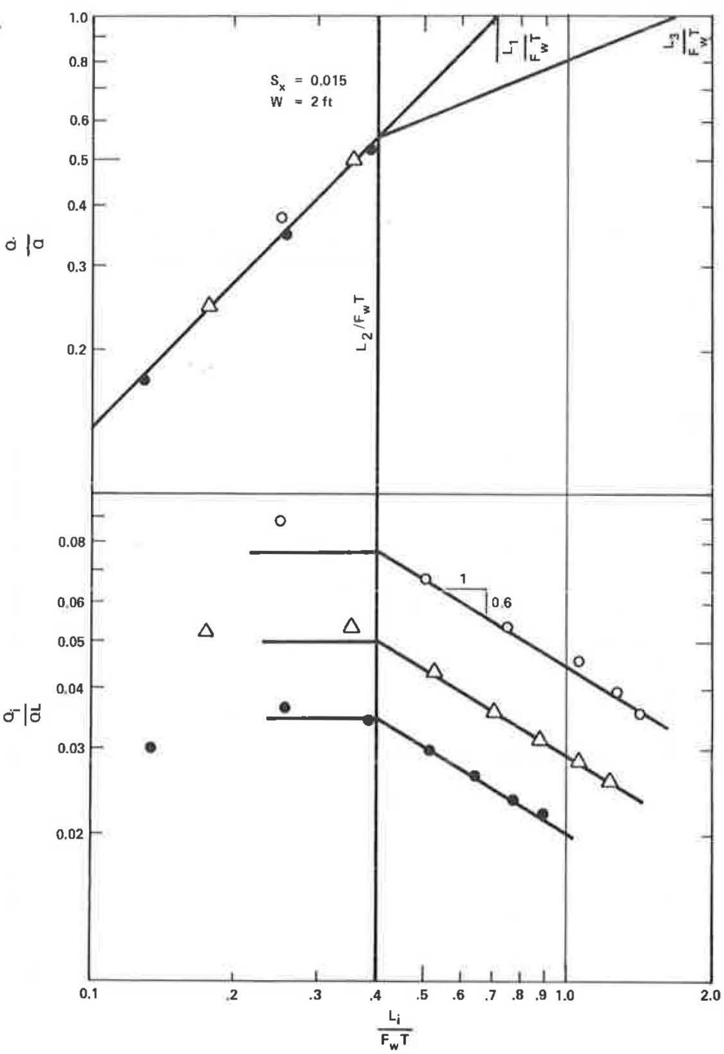
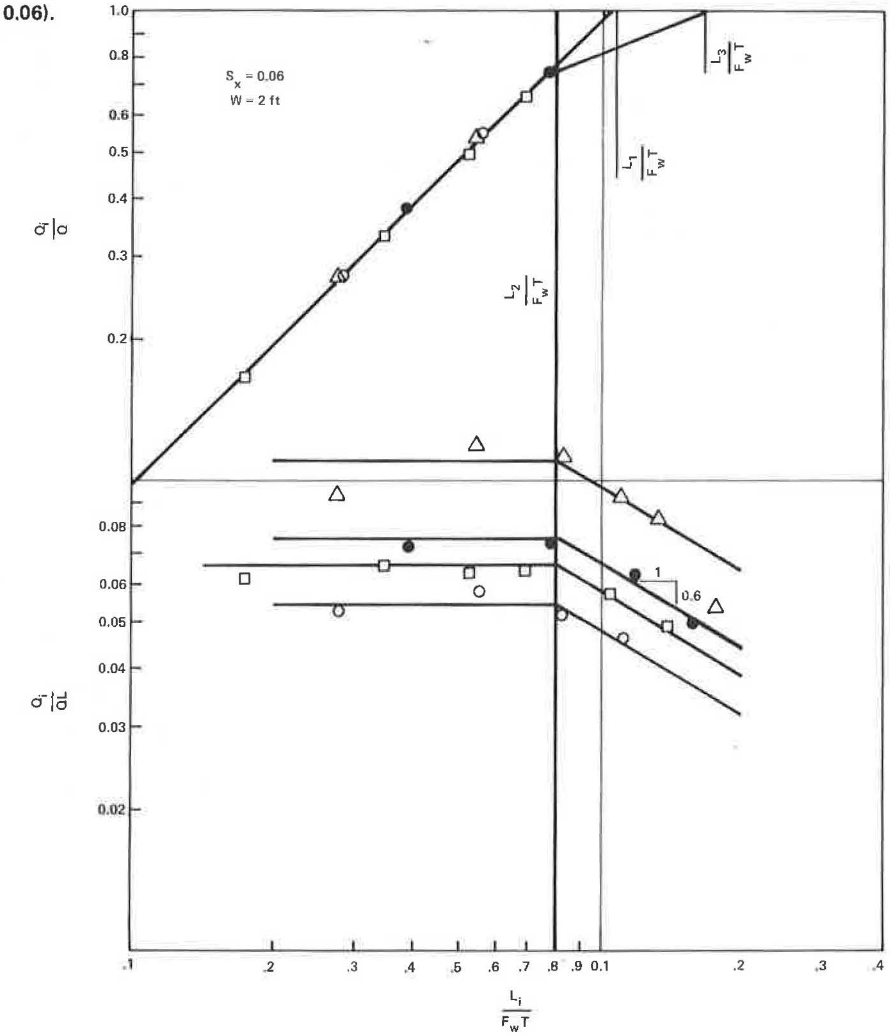


Figure 4. Experimental data (S_x = 0.06).



$$dQ_i/dL_i = (Q/L_3{}^{0.4})0.4L_i^{-0.6} \quad (10)$$

This reduces to $dQ/dL_t = 0.4Q/L_3$ when $L_t = L_3$ and $Q_t/Q = 1$. Thus, the last increment of inlet needed to intercept all of the flow becomes a very small quantity and, if the length of an inlet could be limited to that which would intercept 90 percent of the flow, L_t/L_3 would become $0.9^{2.5} = 0.77$, which means that the inlet could be shortened by 23 percent.

Extrapolation to Steeper Slopes

The design charts (2) are limited to a maximum grade of 4 percent, which was the limit of the experiments. Figure 7 shows that the data can safely be extrapolated to an 11 percent grade. This is because the set of runs on a smooth surface at 4 percent had $F_wT = 39$, and this plots at S = 0.11 for a rough surface having n = 0.016. At some slope, there will be a possibility of the generation of roll waves, which are a function of F_w, but apparently this did not occur within the range of Froude numbers tested.

Inlet on Grade Compared to Same Inlet at Sump

At the point of zero grade (sump), the curb-opening inlet performs according to a modified weir formula (1) that has the following equation when W = 0.6 m (2 ft) (2):

$$Q_i = 1.7(L_i + 1.8W)[d_{max} + (W/12)]^{1.85} \quad (11)$$

in which $d_{max} = S_xT$ and $(L_t + 1.8W)$ is the effective weir length.

In the following analysis, the unit discharge $[Q_t/(L_t + 1.8W)]$ for the weir is compared to the unit discharge (Q_2/L_2) for the inlet on a supercritical slope. Q_2 is the value of Q_t at L_2, the point at which the Q_t curve in Figure 7 levels out. The results are plotted in Figure 8 for the same data sets used in Figures 3 and 4. The line of equality shows that, as an average, Q_2/L_2 is about 0.06 $ft^3/s/ft$ less than that for the inlet at the sump.

Validity of Equations in Intermediate Cross-Slope Range

The relation established above provides a way of checking the probable validity of Equations 4 and 5 for intermediate cross slopes. Computations using W = 0.6 m (2 ft), n = 0.016, S = 0.01, and T = 3.05 m (10 ft) were carried out for cross slopes of 0.03, 0.04 and 0.05. Equations 1, 3, 4, and 5 were combined so that Q_2/L_2 reduces to $Q_2/L_2 = 28.3\, S_x^{1.2}$. The values computed from this equation are plotted against $Q_2/(L_2 + 3.6)$ in Figure 8. These values are slightly higher than those found for the experimental data in the middle range and tend toward divergence beyond $S_x = 0.06$. (Since this value is probably approaching the maximum cross slope, the divergence is not important.) Thus, Equations 4 and 5 can safely be used for interpolation between $S_x = 0.015$ and $S_x = 0.06$.

Composite Cross Section

The street cross section commonly used has a steeper cross slope within the gutter width than in the pavement. Consequently, the equations derived in this paper do not apply strictly.

The composite section is advantageous hydraulically and from the point of view of traffic because it concentrates more of the flow near the curb. Figure 9 shows the capacity of the composite section relative to that of the straight section for the case where the gutter width is 0.6 m (2 ft) and the cross slope is $^1/_{12}$. The computations for these curves are based on the method suggested in the Hydraulic Engineering Circular (2). The differences are negligible for steep cross slopes and large values of T.

No experiments have been run for curb-opening inlets on composite cross sections. However, it is reasonable

Figure 5. Comparison of computed and observed values of Q_i/Q ($S_x = 0.015$).

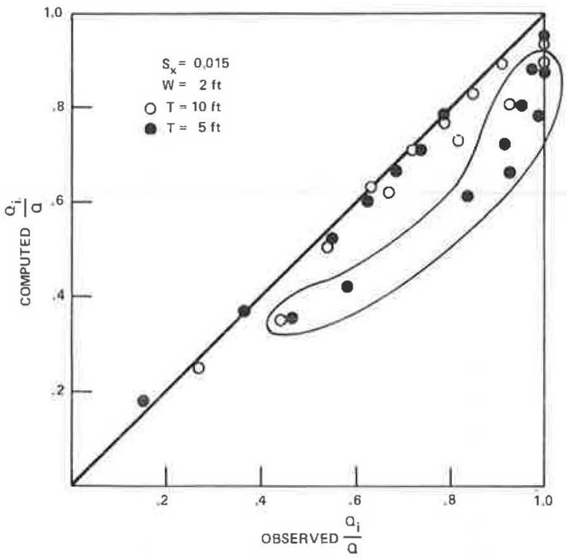


Figure 6. Comparison of computed and observed values of Q_i/Q ($S_x = 0.06$).

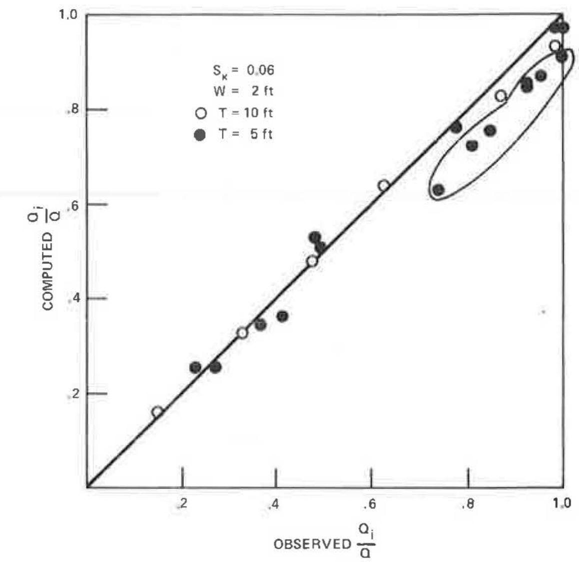
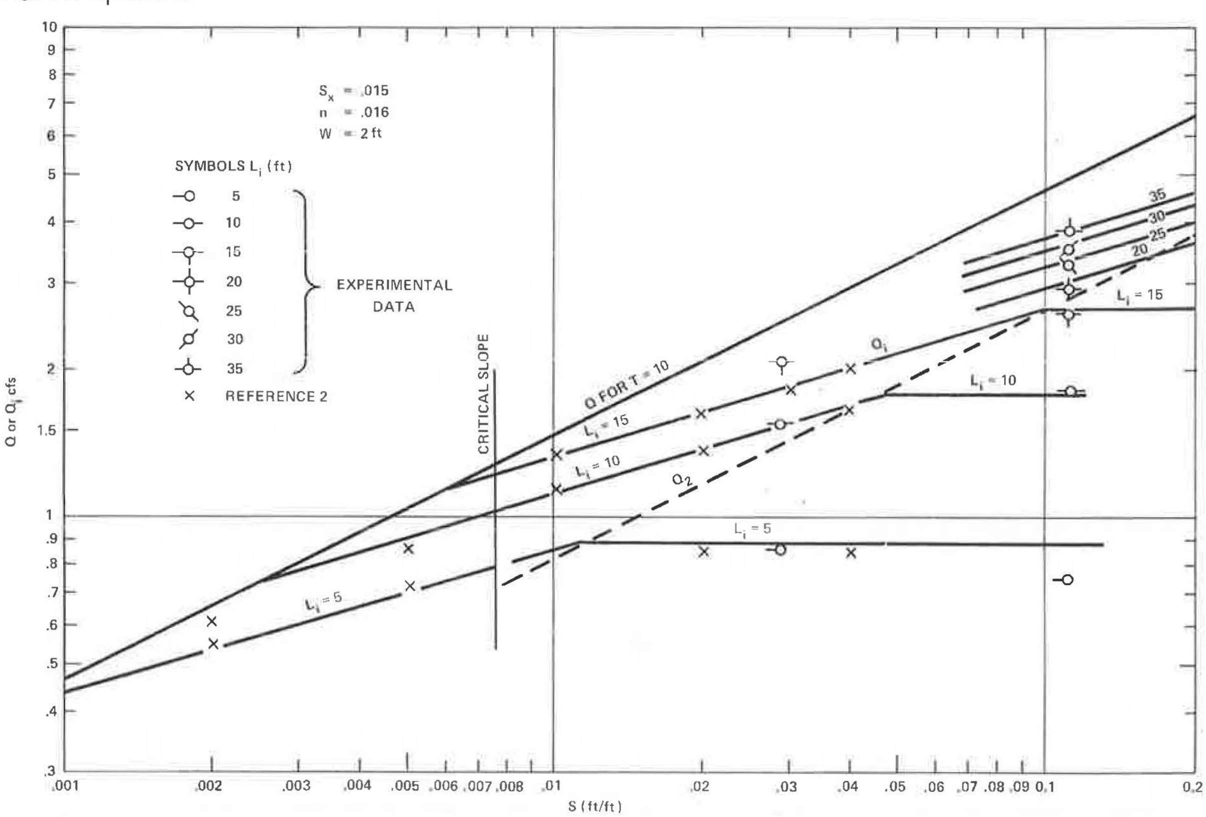
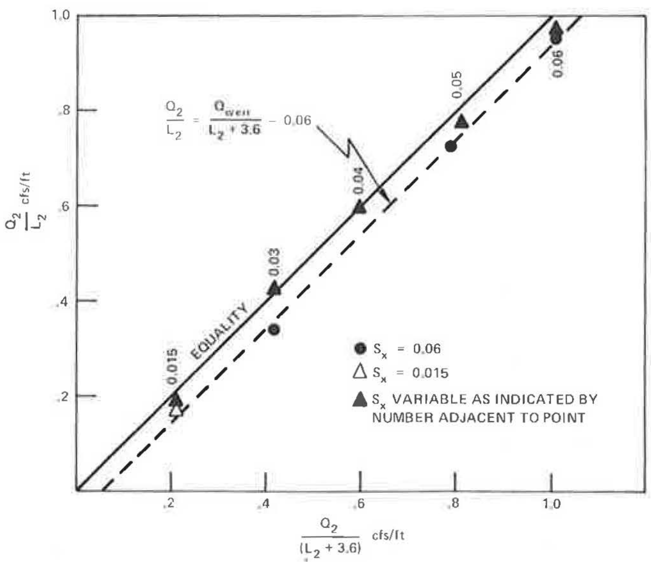


Figure 7. Q_i versus S.



to assume that all of the increment in flow in the deepened gutter will be intercepted if the width of the inlet depression is equal to or greater than the gutter width. Figure 9 shows a way of estimating the increment in flow that will be intercepted.

Figure 8. Comparison of unit discharge of inlet on grade to that of same inlet at sump.



APPLICATION OF SIMPLIFIED METHOD

The application of these equations to the design of curb-opening inlets or to the calculation of the capacity of existing inlets is illustrated in Tables 1 and 2. Fixed values of W = 2 and n = 0.016 are used, which reduce Equations 1, 2, 3, 5, and 6 to the forms shown below.

$$F_w = 16.4[(T - 2)S_x]^{1/6} S^{1/2} \quad (1a)$$

$$T = (Q/35S^{1/2})^{3/8} S_x^{-5/8} \quad (2a)$$

$$Q = 35S_x^{-5/3} T^{8/3} S^{1/2} \quad (3a)$$

$$L_1 = 2.49 S_x^{0.3} F_w T \quad (5a)$$

$$L_2 = 3.27 S_x^{0.5} F_w T \quad (6a)$$

In a design or evaluation, S_x, S, W, and n are generally known. Two other variables must also be known or be selected as design criteria. Usually the runoff rate (Q) at the point of design is known, or T may be given as the criterion for inlet spacing, which will determine Q. The other variable may be Q_i/Q by design criterion or a given L_i. Once the fixed variables are determined, the computations may be programmed for a digital computer, a handheld electronic calculator may be used, or the equations may be graphed for direct solution.

Example 1: The interception capacity of an inlet 2.4 m (8 ft) long is to be determined. Since $L_i < L_2$, Q_i/Q can be calculated by using Equation 4 and is 8/14.7 = 0.54, which makes Q_i = 2.0 ft³/s. The increment that

Figure 9. Ratio of discharge on composite section to discharge on straight section as a function of S_x and T.

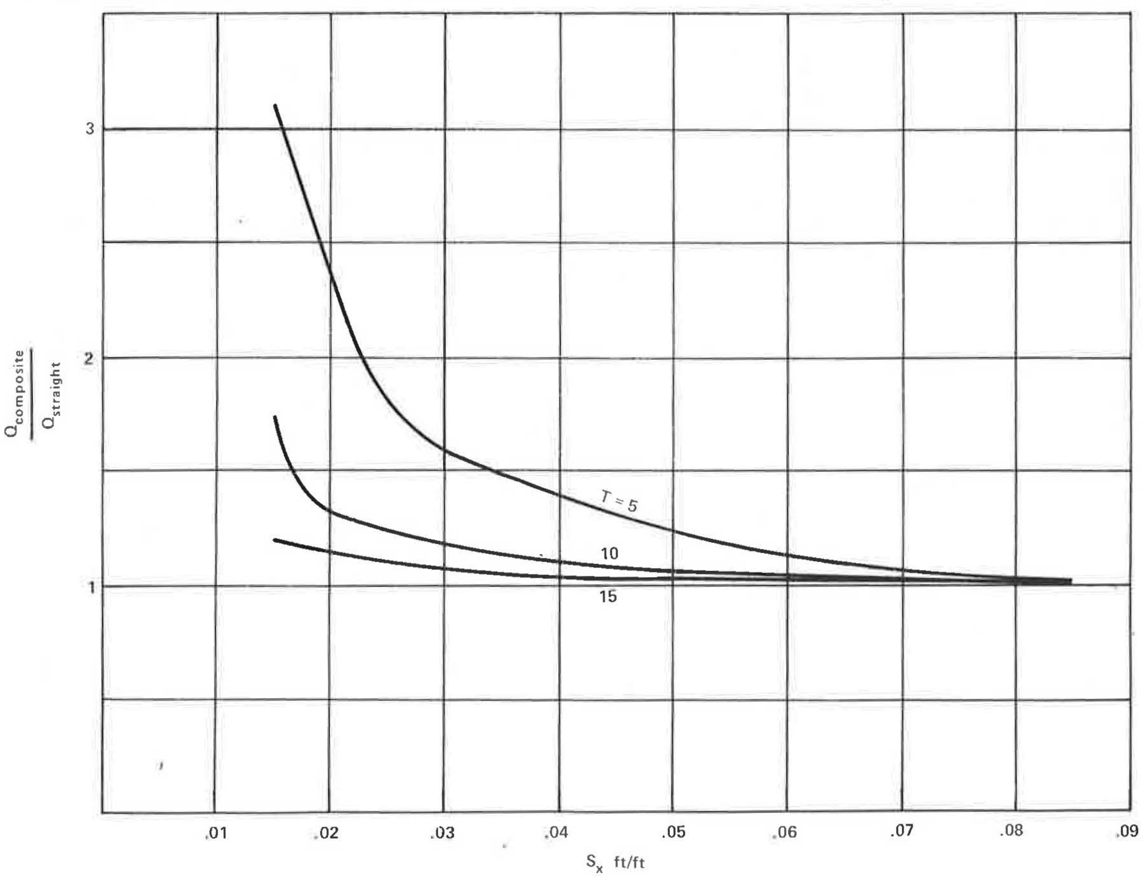


Table 1. Example of computations for design of curb-opening inlets: parameters.

Example	S_x	L_1/F_wT	L_2/F_wT	L_3/F_wT	S	T (ft)	$Q/S^{1/2}$ (ft^3/s)	Q (ft^3/s)	$F_w/S^{1/2}$	F_w	F_w T (ft)
1	0.02	0.770	0.462	1.65	0.025	10	23.6	3.73[a]	12.1	1.91	19.1
2	0.02	0.770	0.462	1.65	0.025	8.6	23.6	2.5[a]	11.7	1.85	15.9
3	0.02	0.770	0.462	1.65	0.025	8.6	23.6	2.5[a]	11.7	1.85	15.9
4	0.02	0.770	0.462	1.65	0.005	9.6	23.6	1.5[a]	12.0	0.847	8.47
5	0.04	0.948	0.654	1.65	0.005	8.0	41.9	3.0[a]	12.9	0.915	7.35

Note: 1 ft = 0.305 m; 1 ft^3/s = 0.028 m^3/s.
[a] Given values.

Table 2. Example of computations for design of curb-opening inlets: results.

Example	L_1 (ft)	L_2 (ft)	L_3 (ft)	L_2/L_1	Q_i/Q	Q_1 (ft^3/s)	Q_2 (ft^3/s)	L_i (ft)	Use L_i (ft)	Q_c (ft^3/s)	Q_{comp} (ft^3/s)
1	14.7	8.8	31.5	0.601	0.54	2.0	2.24	8[a,b]	8	1.7	3.2
2	12.2	7.3	26.2	0.601	0.90[a]	2.0	1.50	20.1[c]	20	0.3	2.7
3	12.2	7.3	26.2	0.601	0.80	2.0	1.50	15[a,c]	15	0.5	2.7
4	6.5	3.9	14.0	0.601	1.0[a]	1.5	1.00	14[c]	14	0	1.9
5	7.0	4.8	12.1	0.690	0.8[a]	2.4	1.80	5.6[c]	6	0.6	2.6

Notes: 1 ft = 0.305 m; 1 ft^3/s = 0.028 m^3/s.
[a] Given values. [b] $Q_i < Q_2$ and $L_i < L_2$. [c] $Q_i > Q_2$ and $L_i > L_2$.

would be added if the approach gutter of width W was sloped $^1/_{12}$ is read from Figure 9 and is 0.31 Q = 0.31(3.73) = 1.2 ft^3/s. As a check of the calculations L_2/L_1 should have the same value as computed by Equation 7.

Example 2: The length of an inlet to intercept 90 percent of the flow is to be calculated. In this case, $Q_1 > Q_2$ and Equation 8 is used. This gives L_i = $(0.9)^{2.5}L_3$ = 0.768(26.2) = 20.1 ft. If this is rounded to 20 ft the carry-over discharge (Q_c) will be 0.3 ft^3/s.

Example 3: The capacity of a 4.6-m (15-ft) inlet for the conditions given in example 2 is to be calculated. Here $L_i < L_2$ and Equation 8 again is used. This makes $Q_i/Q = (15/26.2)^{0.4}$ = 0.80 and Q_i = 0.8(2.5) = 2.0 ft^3/s.

Example 4: A subcritical slope is assumed, which makes F_w = 0.856, and 100 percent interception is required for Q = 1.5 ft^3/s. This could be an inlet at the end of a block where no carry-over is to be permitted. In this case $Q_1 > Q_2$ and Equation 8 is used: L_i = $1^{2.5}(14)$ = 14 ft. This result could have been obtained more simply by noting that, when Q_i/Q = 1, $L_i = L_3$.

Example 5: S_x is 0.04, and 80 percent of the flow is to be intercepted. The required length of inlet is 5.6 ft or a 6-ft standard inlet. (If 100 percent interception is required, the length will be 12 ft = L_3.)

A detailed program for future research on curb-opening inlets is beyond the scope of this paper, but the recognition of certain deficiencies in the data points to the need for evaluation of the following:

1. Performance on subcritical slopes,
2. Performance on supercritical slopes up to about 15 percent,
3. Inlets on street sections having a gutter cross slope that is steeper than that of the pavement and on parabolic cross sections,
4. Effects of the width of depression on inlet performance,
5. Performance on at least one cross slope between 0.015 and 0.06 and possibly up to 0.10 (inlets on superelevated curves), and
6. Effects of guide vanes cast into the inlet-depression surface.

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As chief of the hydraulic research division at the time the experimental research (3) was conducted, I must accept responsibility for deficiencies in the experimental data that show up in this paper. The current study leading to this paper has not been supported by any agency.

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