VOL. 30, NO. 10

OCTOBER 1959

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Partially completed Peoria (Ill.) Expressway. This interstate improvement provides a much needed north-south artery and bridge over the Illinois River joining Peoria with East Peoria.

PUBLISHED
BIMONTHLY BY THE
BUREAU OF
PUBLIC ROADS,
U.S. DEPARTMENT
OF COMMERCE,
WASHINGTON

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Published Bimonthly

Vol. 30, No. 10 *October 1959*

C. M. Billingsley, *Editor*

BUREAU OF PUBLIC ROADS

Washington 25, D.C.

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Use of funds for printing this publication has been approved by the Director of the Bureau of the Budget, March 28, 1958.

Contents of this publication may be reprinted. Mention of source is requested.

IN THIS ISSUE

U.S. DEPARTMENT OF COMMERCE

FREDERICK H. MUELLER, Secretary

BUREAU OF PUBLIC ROADS

BERTRAM D. TALLAMY, Administrator
ELLIS L. ARMSTRONG, Commissioner

The Use of Backwater in the Design of Bridge Waterways

BY THE DIVISION OF HYDRAULIC RESEARCH
BUREAU OF PUBLIC ROADS

Reported by JOSEPH N. BRADLEY,
Hydraulic Research Engineer

No generally accepted method has existed for the design of bridge waterways. The determination of the length of a bridge over a stream has been left to the bridge engineer's personal observation and experience. A comparison of the small number of bridge failures to the total number of bridges throughout the country attests to the commendable job bridge designers have performed with the limited design tools available. But, what proportion of these existing bridges are underdesigned or overdesigned from a standpoint of length and clearance? With many new bridges scheduled to be constructed under the accelerated highway program, this question deserves serious contemplation as to safety and economy. Given sufficient time, underdesigned bridges usually speak for themselves. In the case of overdesign, no reliable standards exist at the present time by which these structures can be judged impartially.

Since the subject of bridge waterways is too extensive even for a condensed treatment, the context of this article is confined to a discussion of only one phase of the problem, bridge backwater. It contains a brief account of the problem, the research results, the design information derived therefrom, and the application of bridge backwater to waterway design. The data presented are based on both experimental backwater studies using hydraulic models and field measurements.

IN 1954, a cooperative research project aimed at improving bridge waterway design methods was initiated by the Bureau of Public Roads at Colorado State University at Fort Collins. To date, the investigations have centered on the determination of backwater produced by bridges (*1, 2*),[1] scour at bridge abutments, scour around piers, and methods for alleviating such scour. Two research projects at the University of Iowa, sponsored by the Iowa State Highway Commission and the Bureau of Public Roads, have also contributed needed information on scour at bridge piers (*3*) and scour at bridge abutments (*4*).

Bridge waterway problems are diversified and complex, which account to some extent for the limited progress made in the past in understanding and resolving this phase of design. Because of the many variables involved, hydraulic models were used to serve as the principal research tool in all the work mentioned in the previous paragraph. It is possible with models to hold a certain number of variables constant while investigating the effect of others; then by systematically rotating the combination of variables in the test program, holding some constant and allowing others to vary, it is possible to isolate the part that certain principal variables play in the final result. In addition to aiding in a better understanding of the theory and mechanics involved, the models are indispensable since experimental coefficients are required which can be obtained in no other way.

Experimental Backwater Studies

A comprehensive record of the experimental data, test procedures, and analysis of results on bridge backwater studies appears in a report issued by Colorado State University (*1*). For those interested only in the design application, a booklet entitled *Computation of Backwater Caused by Bridges* (*2*) is recommended. This booklet contains design charts, an explanation of design procedures, and five practical examples. Since the above information is available, it will be necessary to draw from it only sufficiently to understand the contents of this article.

The manner in which flow is contracted in passing through a channel constricted by bridge embankments is illustrated in figure 1. The flow bounded by each pair of streamlines represents 1,000 c.f.s. It will be noted that channel constriction appears to produce very little alteration in the shape of the streamlines near the center of the channel, while a marked change is evident near the abutments where flow from the flood plains enters the constriction. As the discontinuity is greatest in this region, it is apparent that areas adjacent to the abutments can be most vulnerable to attack by scour during floods. Upon leaving the constriction, the flow, which is concentrated in the central portion of the channel, expands at an angle of 5 to 7 degrees on a side until normal conditions are reestablished downstream which may involve a considerable reach of the river.

Constricting the flow of a stream, of course, produces a loss of energy, the greater portion of this occurring in the reexpansion process downstream from the constriction. This loss of energy is reflected in a rise in both the water surface and the energy gradient upstream from the bridge as demonstrated by a profile of this same crossing taken along the centerline of the stream (fig. 2). The normal stage, or water surface existing for a given flood prior to construction of the bridge, is represented by a straight broken line. The water surface for the same flood, with constricting bridge embankments, is denoted by the solid line labeled water surface on centerline (W.S. on ℄). The water surface is above normal stage at section 1, passes through normal stage in the vicinity of section 2, reaches minimum depth near section 3, and returns to normal stage a considerable distance downstream at section 4, where the original regime of the river has not been disturbed. The energy at section 4 is thus the same with or without the bridge. The energy at section 1, on the other hand, must increase to provide head to overcome the loss introduced by the constriction. The major portion of this energy increase is reflected in the backwater, which is the vertical rise in water surface at section 1 (denoted by the symbol h^*_1 in fig. 2).

Note that the drop in water surface measured across the roadway embankment is not the backwater as is so often presupposed to be the case. The water surface as indicated in the central part of the channel at section 3, which is essentially the water surface along the downstream side of the embankments, is invariably lower than normal stage, so the difference in level across the embankments, Δh, is always larger than the backwater h^*_1.

It was found that the backwater to be expected at a bridge for a given discharge is dependent on a number of factors. The more prominent of these are: (1) the degree of constriction of the channel; (2) the number, size, shape, and orientation of piers in the constriction; (3) eccentricity of the bridge with the low-water channel or flood plain; (4) the angle or skew of the bridge with the stream; (5) the type and slope of bridge abutments (important only for the shorter bridges); (6) the amount of scour experienced in the constriction; and (7) the type of crossing; i.e., whether a single bridge or two or more parallel bridges on a divided highway. Contrary to expectations, the

[1] Italic numbers in parentheses refer to the references on p. 231.

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width of the abutment or roadway fill had no significant effect on the backwater.

Without a reliable stage-discharge curve for the bridge site, a backwater study can have but limited value. Also, a knowledge of the flood frequency and magnitude is required in order to determine the design discharge for a bridge and the necessary clearance (2).

In spite of the number of principal variables just enumerated, the backwater expression and the procedure for computing backwater, as developed from the experimental studies, are very realistic. A person with some training in hydraulics should have no particular difficulty in mastering this phase of waterway design.

An abbreviated form of the expression for computing bridge backwater follows:

$$h^*_1 = K^* \frac{V_{n2}^2}{2g} + (- - - - - \quad \text{Equation (1)}$$

In this expression, K^*, which consists of a combination of experimental backwater coefficients, is multiplied by a velocity head. The overall coefficient K^* varies with the seven geometric factors previously mentioned, while the velocity is computed with respect to the average water cross section under the bridge relative to normal stage. The remainder of the expression, which has been omitted for the sake of simplicity, consists of the change in kinetic energy between sections 1 and 4 (fig. 2) produced by alteration of the stream by the bridge. In many, but not all, of the cases this factor represents a small portion of the total backwater. Guides are provided whereby the importance of this factor can be readily recognized and omitted from the computations where permissible (2).

To present a general idea of the manner in which the expression for computing bridge backwater (equation 1) operates, the backwater coefficient for a symmetrical normal stream crossing, having wingwall abutments without piers or other complicating features, may be obtained directly from figure 3. The coefficient K_b (base curve value) varies with the degree of constriction of the channel M and the type of abutment. The parameter M is the ratio of the quantity of flow which can pass through the constriction unimpeded to the total discharge of the river. For no constriction of the stream, $M=1$ and the coefficient is zero. As the degree of constriction increases, M becomes less than unity and the coefficient K_b increases in value. To illustrate, the contraction ratio for the condition shown in figure 1 would be M=8,400/14,000=0.60. If piers, eccentricity, or skew are involved, the effect of these factors is accounted for by adding incremental coefficients to the value obtained from the base curve which results in an overall coefficient

$$K^* = K_{b\,(\text{base})} + \Delta K_{p\,(\text{piers})} + \Delta K_{e\,(\text{eccentricity})} + \Delta K_{s\,(\text{skew})}.$$

The values of the incremental coefficients for the effect of piers, eccentricity, and skew are obtained from charts prepared for that purpose. A detailed description of the procedure and the charts can be found in reference (2).

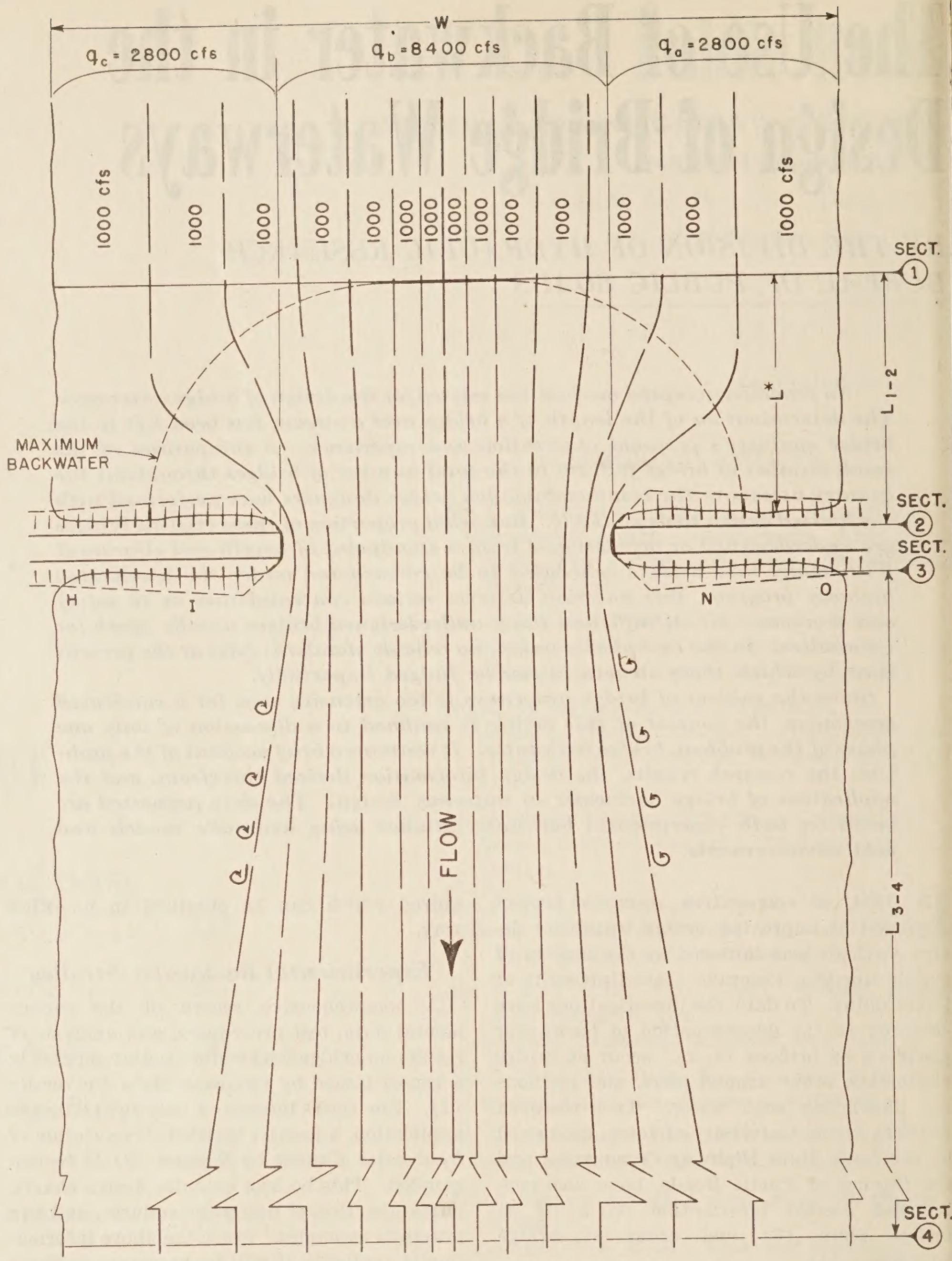

Figure 1.—Flow lines of typical normal crossing.

Reliability of Model Results

Since hydraulic models of bridges have definite limitations, it was imperative that some means be devised to verify or disprove the validity of the experimental information. This was accomplished by applying the computational procedure, developed from the model studies, to existing bridges for which the U.S. Geological Survey had furnished field measurements obtained during floods. Reliable measurements on bridge backwater are extremely difficult to make in the field, but the drop in water surface across embankments, Δh, is readily measurable (fig. 2). Model results showed a very definite relation existing between the drop in water surface across the embankments, Δh, and the backwater, h^*_1, so model computations and prototype measurements are compared on the basis of Δh.

A comparison of measured and computed values for several bridges varying from 20 to 340 feet in length is presented in table 1. Columns 2 through 6 give the bridge length, flood discharge, average velocity under the bridge, the contraction ratio, and the computed backwater, respectively. The computed and measured values of Δh are shown in columns 7 and 8, while the percentage difference in each case is shown in column 9. The differences range from −8.5 to +13 percent, the deviation being positive in six instances and negative for six with an average deviation of +2 percent. The deviation in the majority of the cases is well within the error of field measurement. The experimental error of the model experiments is estimated as

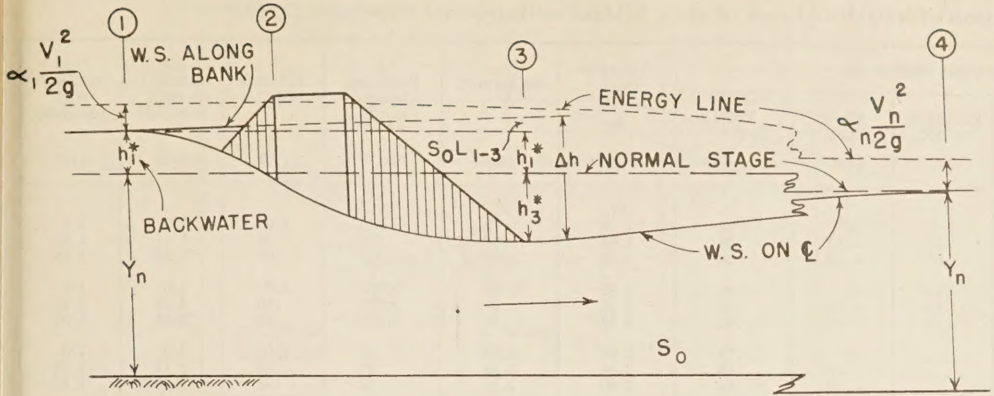

Figure 2.—Profile on centerline of stream.

comparable to the average deviation. Thus, the comparison affords a satisfactory verification.

Application of Backwater to Design

Now that it is possible to compute bridge backwater with a fair degree of confidence, to what practical purpose can this information be used in design?

1. It makes it possible to proportion bridges to operate during floodflows at a limited specified backwater.
2. It offers a fair means of settling claims involved in backwater damage suits instigated by upstream property owners.
3. It makes it possible to understand and compute the hydraulics involved in cases where approach roadways can be overtopped during infrequent floods.
4. It provides a large share of the necessary hydraulic information for a proposed economic analysis to determine the optimum design discharge and the most economical length of bridge.

In the case of item 2, no acceptable method has existed for computing backwater produced by bridges. Backwater based on field measurements made by the novice was also justifiably questionable. Thus, damage suits of this nature have resulted in indefinite delays or settlements have been made on considerations other than fact. The attainment of a sound method of procedure for determining the optimum design discharge and the most economical length of bridge (item 4) constitutes the ultimate goal in the present research program.

Application of Backwater to Length of Skew Crossing

A practical application to which the bridge backwater information may be used to advantage can be demonstrated by comparing the length and cost of skew bridges with the length and cost of equivalent normal crossings, on the basis of backwater. The procedure consists of choosing an existing normal stream crossing and computing the backwater which the bridge will produce for a given flood condition; then, holding stream conditions constant, compute the length of equivalent skew bridges which have the same effective waterway; i.e., produce the same backwater. This course of computation was followed for seven existing crossings and the results are shown in table 2. The normal length of these bridges varied from 75 to 2,000 feet and included both wingwall and spillthrough type abutments. The faces of the abutments under the bridge were oriented parallel with the flow, as shown in figure 4. This is the most efficient skew abutment shape. Abutments with faces at an angle to the flow require more length of bridge.

The ordinate in figure 4 is the ratio of skew length to normal length of crossing in percent, which is plotted with respect to the skew angle as abscissa and the contraction ratio M as a third variable. In the case of $M=1.0$ (no constriction of the stream), the skew length is simply $L_n/\cos \phi$. With constriction of the stream, the ratio L_s/L_n decreases with the value of M.

What is occurring can be better understood by referring to figure 5. The ordinate is the ratio of the projected skew length to the

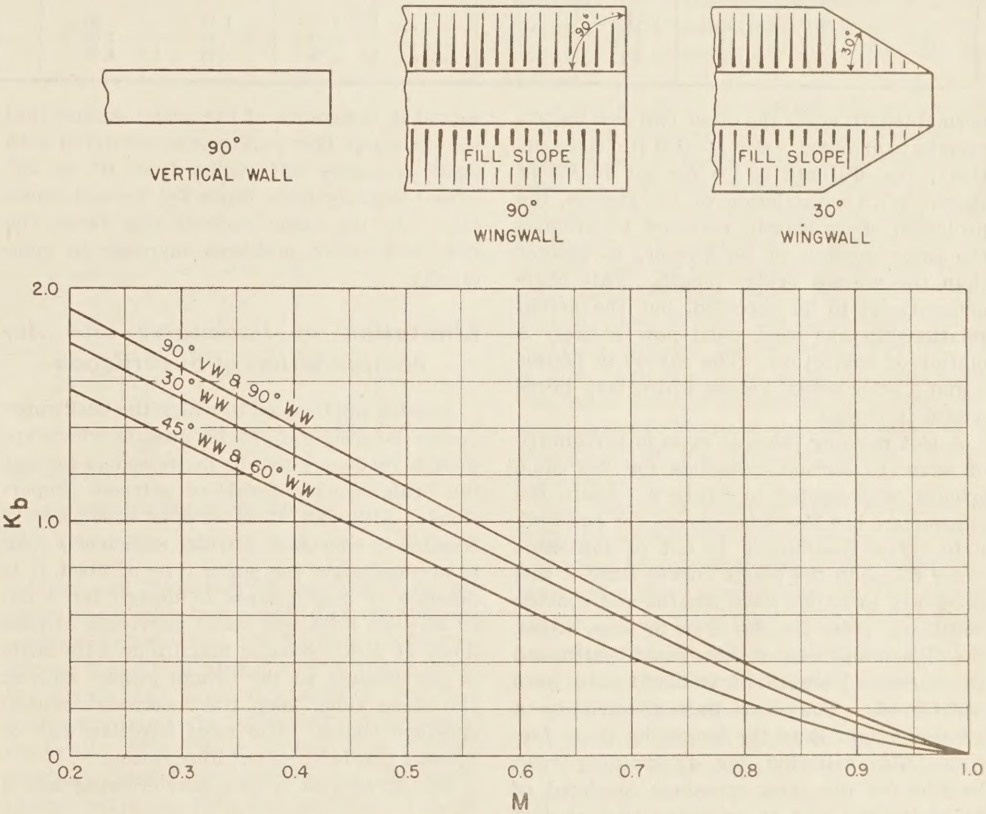

Figure 3.—Backwater coefficient K_b for wingwall abutments (base curve).

Table 1.—Comparison of computed Δh values with field measurements

Bridge number	Bridge length	Flood discharge	Average velocity under bridge	Contraction ratio, M	Computed backwater, h^*_1	Drop across embankments		Percentage difference, Δh
						Computed Δh	Measured Δh	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
	Ft.	*C.f.s.*	*F.p.s.*		*Ft.*	*Ft.*	*Ft.*	
1	20	1,370	9.1	0.57	1.07	1.90	1.99	−4.5
2	84	4,340	6.8	.85	.21	.65	.70	−7.2
3	220	27,500	7.5	.90	.28	.76	.83	−8.5
4	83	5,240	8.6	.60	1.03	1.81	1.60	13
5	72	12,000	10.2	.83	.57	1.94	1.95	−.5
6	58	3,400	7.1	.82	.18	.61	.55	10.9
7a	44	2,620	7.8	.66	.63	1.23	1.15	6.9
7b	44	1,450	5.4	.70	.30	.66	.69	−4.4
8	112	9,640	9.0	.33	1.80	2.53	2.24	12.9
9	340	70,000	10.5	.90	.77	2.57	2.70	−5.0
10	68	7,230	(1)	(1)	----------	1.53	1.48	3.4
11	120	2,600	(1)	(1)	----------	1.70	1.61	5.6

[1] Deck girder immersed.

Table 2.—Comparison of length and cost of skew bridges with normal crossings

Bridge identification	Skew angle, degrees	Contraction ratio, M	Backwater coefficients: Base k_b	Piers Δk_p	Eccentricity ΔK_e	Skew ΔK_s	Total K^*	Velocity head, $\frac{V_{n2}^2}{2g}$	Backwater, h^*_1	Projected bridge length	$\frac{L_s \cos \phi}{L_n}$	Length (skew)/length (normal)	Cost (skew)/cost (normal)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
								Ft.	Ft.	Ft.			
A	0	0.90	0.12	0.09	0.07	0	0.28	1.70	0.76	340	1.0	1.0	1.0
	30	.90	.12	.09	.07	−.02	.26	1.84	.76	335	.98	1.14	1.22
	45	.90	.12	.09	.07	−.03	.25	1.89	.76	330	.97	1.37	1.54
B	0	.67	.48	.04	.15	0	.67	1.22	.89	2,000	1.0	1.0	1.0
	30	.665	.50	.05	.15	−.05	.65	1.26	.89	1,925	.96	1.12	1.18
	45	.66	.51	.06	.15	−.08	.64	1.30	.89	1,900	.95	1.34	1.50
C	0	.64	.55	.19	--------	0	.74	2.66	2.18	87	1.0	1.0	1.0
	30	.635	.55	.19	--------	−.06	.68	2.84	2.18	84	.96	1.12	1.22
	45	.63	.56	.19	--------	−.09	.66	2.90	2.18	80	.92	1.30	1.50
D	0	.62	.60	.03	.16	0	.79	1.65	1.41	1,100	1.0	1.0	1.0
	30	.62	.60	.04	.16	−.07	.73	1.79	1.41	1,025	.93	1.08	1.15
	45	.61	.62	.05	.16	−.11	.72	1.82	1.41	1,010	.92	1.30	1.45
E	0	.53	.92	.08	--------	0	1.00	1.11	1.19	630	1.0	1.0	1.0
	30	.52	.96	.09	--------	−.19	.86	1.27	1.19	600	.96	1.10	1.16
	45	.51	1.00	.12	--------	−.37	.75	1.46	1.19	575	.91	1.29	1.42
F	0	.46	1.13	.15	.04	0	1.32	.67	.93	1,075	1.0	1.0	1.0
	30	.43	1.16	.16	.04	−.26	1.10	.82	.93	990	.92	1.06	1.08
	45	.42	1.21	.19	.04	−.49	.95	.90	.93	925	.86	1.22	1.31
G	0	.46	1.06	.06	--------	0	1.12	.90	1.05	75	1.0	1.0	1.0
	30	.44	1.08	.08	--------	−.25	.91	1.09	1.05	69	.92	1.06	1.12
	45	.42	1.14	.09	--------	−.48	.75	1.35	1.05	64	.86	1.20	1.34

normal length while the other two parameters remain unchanged. For M=1.0 (no constriction), the ordinate is 1.0 for all angles of skew. With constriction of the stream, the projected skew length, required to produce the same amount of backwater, is shorter than the normal bridge length. This characteristic is to be expected, but the actual relationship has been until now entirely a matter of conjecture. The curves in figures 4 and 5 offer actual values which may prove useful in design.

A plot relating the cost ratio in percentage of skew to normal crossings for the same bridges is presented in figure 6. Again the parameters are the same except for the ordinate. The consistency is not of the same order found in the length curves since it was necessary to adjust span lengths and provide additional piers for the skew bridges. Also, the higher unit cost of skew construction and the increased length of embankments were considered. The cost was affected to a greater extent than the length by these factors. The criterion for determining span lengths for the skew crossings consisted of balancing the cost of superstructure against the cost of piers on an equal basis. The increase in cost of superstructures per square foot was assumed to be 5 percent for the 30° skew and 10 percent for the 45° skew.

For the purpose of comparison, the length varies from 107 to 115.5 percent of normal for the 30° skew, while the cost variation for the same range of contraction ratios is from 110 to 127 percent. In the case of the 45° skew, the length varies from 120 to 141 percent of normal compared with a cost variation of 130 to 158 percent for the same values of M.

Skew angles from 0° to 20° produce only a small increase in both length and cost over the normal crossing (figs. 4–6). As the angle increases above this value the curves get steeper and the length and cost rise more rapidly. In this same connection, it was observed in the course of the model studies that the hydraulic flow problems encountered with skew crossings for angles from 0° to 20° varied slightly from those for normal crossings. As the angle exceeds this value, the flow and scour problems increase in complexity.

Limitation of Backwater and Accommodation of Superfloods

Another application in which the backwater design information can be used is where approach roadways can be depressed to protect the bridge during floods of extreme proportions. Although it is seldom economically feasible to construct a bridge sufficiently long to accommodate the super type of flood, it is possible in many cases to design for a 35- or 50-year flood but make provision to pass flows of much greater magnitude with little or no damage to the bridge proper and, at the same time, keep the backwater within specified limits. The most effective way to present this is by actual illustration.

The stream at a proposed crossing has a low-water channel about 700 feet across, while in flood the stream may be a mile wide. Records show that within the past 50 years two floods approximating frequencies of 100 years have occurred on this stream, the last one destroying a bridge at the site. This is on a State route carrying a moderately heavy volume of traffic. A considerable amount of residential and business development occupying portions of the flood plain has sprung up within the last decade. It is therefore important from the traffic viewpoint that the bridge structure not fail or be out of service for an extended length of time during its expected life; and from the standpoint of life and property damage, it is desirable that the bridge backwater be limited to a definite figure for all flows. For the purpose of illustration the bridge is reconstructed to satisfy the above requirements and limit the backwater to 0.5 foot for any discharge likely to occur during the life of the bridge.

There is a choice here of designing a long bridge to take the full flow of the river for a 100-year frequency flood, keeping the embankments above high water at all times, or the

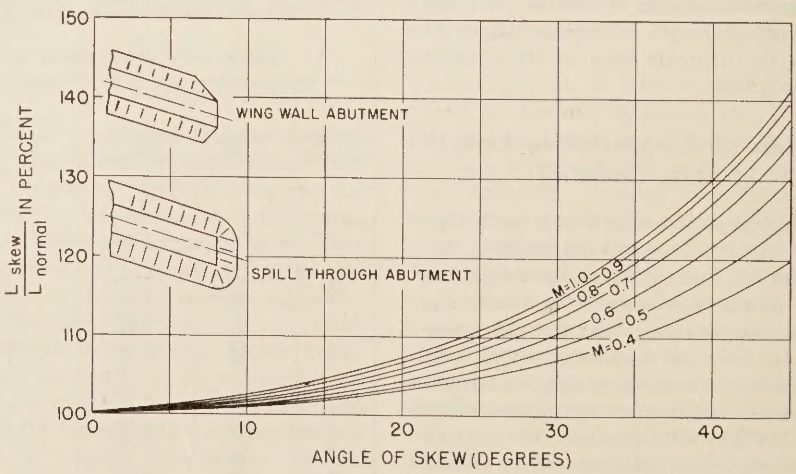

Figure 4.—Length ratio of skew to normal crossings for equivalent backwater.

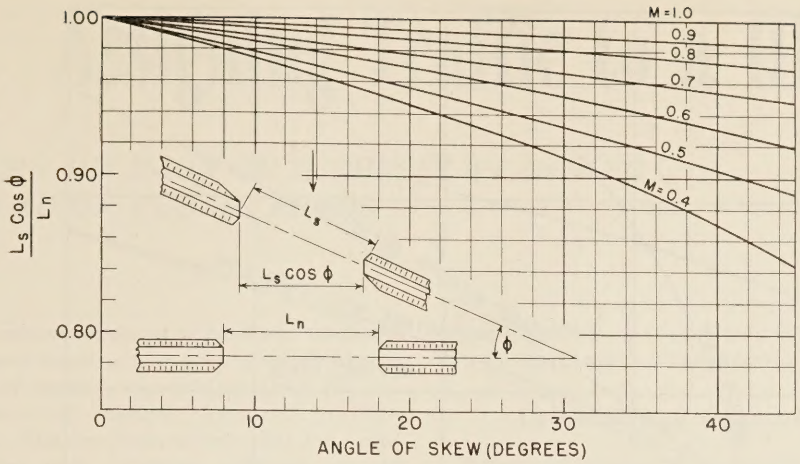

Figure 5.—Ratio of projected skew length to normal bridge length for equivalent backwater.

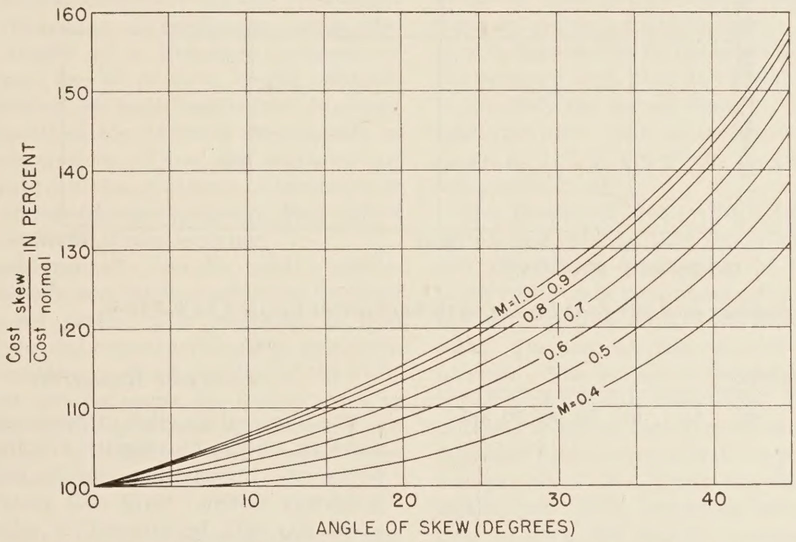

Figure 6.—Cost ratio of skew to normal crossings for equivalent backwater.

alternative of choosing a shorter bridge and using the one-half to three-fourth mile of roadway transversing the flood plain as a spillway during extreme high water. In either case the superstructure will be located above extreme high water at all times.

The case where the embankments are located above high water and the bridge is required to accommodate the entire flow will be investigated first. Figure 7 shows the backwater relative to length of bridge and discharge for this type of operation. In addition, scales have been superimposed showing flow recurrence interval at the top and bridge cost at the right. Were there no restriction on backwater, a bridge 1,500 feet long, producing 1.5 feet of backwater, might be a reasonable choice. But with backwater limited to 0.5 foot, it is observed that the bridge should be 2,250 feet long for a 50-year flood, or 2,600 feet long for a 100-year flood. From the scale on the right, the cost involved in reducing the backwater from 1.5 feet to 0.5 foot approximates $400,000 for the 100-year flood or 158 percent of the cost of the former. This comparison demonstrates how limitation of backwater can increase the initial cost.

How can limitation of backwater be accomplished with less cost? Let us now examine the alternative, the depressed roadway. Figure 8 demonstrates a very extreme case; the bridge has been shortened to 800 feet with approximately 3,500 feet of depressed roadway. The lower broken line, labeled "normal stage," represents the stage-discharge curve for the river prior to construction of the bridge. The upper broken line, labeled "stage without overflow," represents the stage discharge to be expected upstream from the 800-foot bridge without overflow. The difference between the two curves represents the backwater. For a discharge of 250,000 c.f.s. (50-year flood) the backwater is 2.5 feet, and for 300,000 c.f.s. (100-year frequency flood) the backwater approximates 4 feet. The limitation of 0.5 foot for backwater is reached at a discharge of 121,000 c.f.s. If the approach embankment is placed at elevation 87.5 so water will spill over the roadway for flows greater than 121,000 c.f.s., the backwater will decrease with further increase in discharge, falling off to about 0.1 foot for a discharge of 300,000 c.f.s. The backwater with overflow is represented by the difference between the lines labeled "normal stage" and "stage with overflow." As the roadway overflows, the discharge under the bridge increases slowly with upstream stage, while the roadway flow increases rapidly with stage. At stage 93.8 these lines intersect and the roadway is carrying as much flow as the bridge.

As the roadway is elevated, the backwater and flow characteristics remain similar to those shown in figure 8 but the bridge length must be increased if the backwater is to be limited to 0.5 foot for upstream stage level with the new roadway. Figure 9 demonstrates how the backwater varies with roadway elevation and length of bridge. The dashed lines denote the backwater which could be expected for several bridge lengths if flow over the road were not permitted. The solid lines demonstrate how flow over the roadway restricts the backwater to a maximum of 0.5 foot regardless of the discharge.

The depressed roadway not only serves to hold the backwater within limits but offers a means of accommodating the superflood without undue overloading of the bridge proper. It is true that the higher the approaches and the shorter the length of embankment, the longer the bridge must be for a given amount of backwater; nevertheless it is usually possible to set embankments for the 50-year flood stage and still retain this desirable safety valve feature.

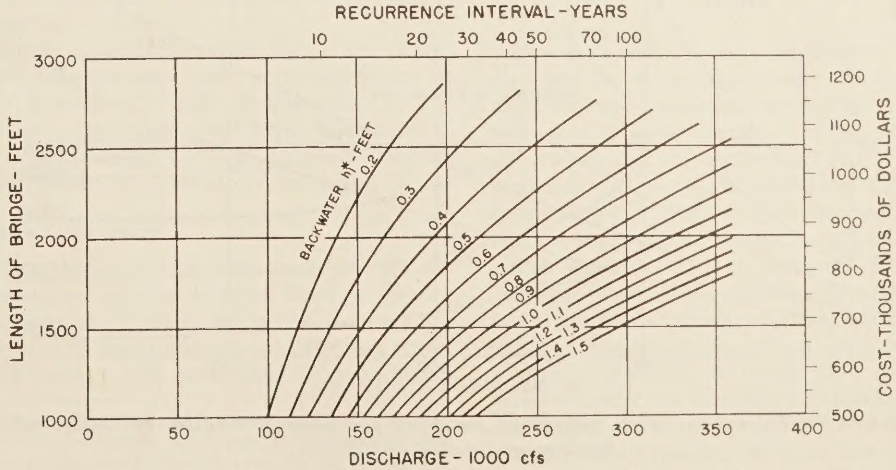

Figure 7.—Variation of backwater with length of bridge and discharge.

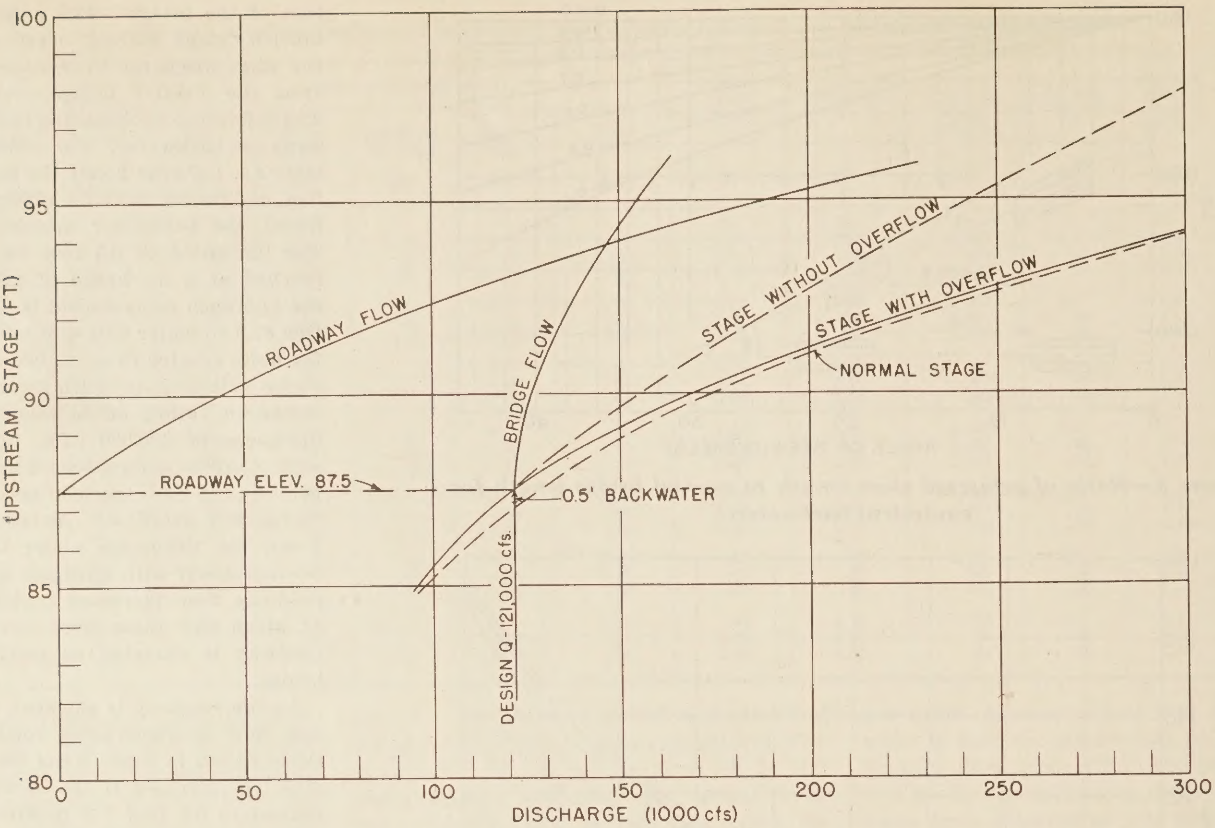

Figure 8.—Operation with depressed roadway and 800-foot bridge with backwater limited to 0.5 foot.

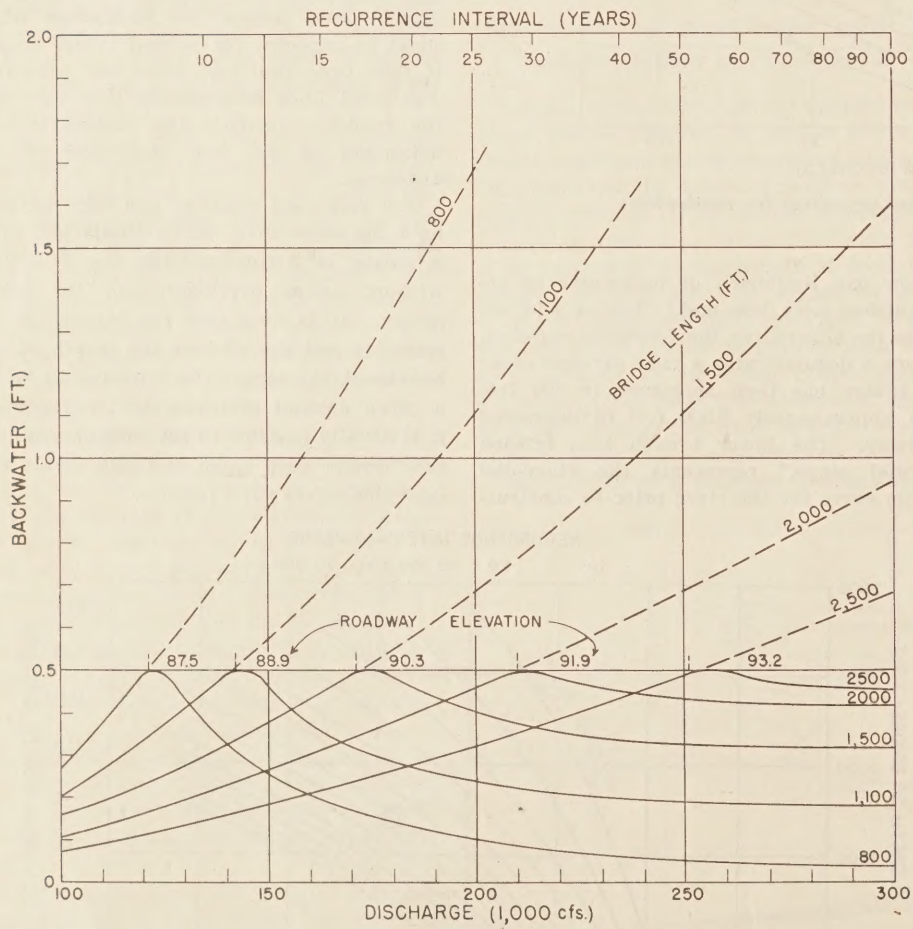

Figure 9.—Operation with depressed roadway for several lengths of bridge with backwater limited to 0.5 foot.

Current Research

The illustrations cited represent only a few ways in which the recently acquired bridge backwater information can be applied to waterway design. Gaps still remain which eventually will be plugged as reliable field data become available.

An equally important or second phase involving the hydraulics of waterways is the reasonable prediction of maximum scour depths at abutments and piers. Some information is already available for streams with alluvial beds (*3*, *4*), and additional information will be forthcoming.

It will be found that the hydraulic analysis offers many variations of supposedly good waterway proportions. How then is an unbiased choice to be made? It is believed that this can best be accomplished through development of a generally acceptable type of economic analysis—which at the present does not exist—taking into account all tangible and certain intangible costs over the useful life of a bridge which may be incurred by highway agencies in building, operating, and maintaining a bridge and by highway users who travel over the bridge. In this way a fair monetary value can be assigned to each design, whereby comparisons can be made on a basis familiar to all parties concerned. Determination of the fundamental concepts on which such an economic analysis should be founded constitutes the third major phase of this research program now under consideration.

(*Continued on page 231*)

Assigning Traffic to a Highway Network

BY THE DIVISION OF HIGHWAY PLANNING
BUREAU OF PUBLIC ROADS

Reported by GLENN E. BROKKE, Highway Research Engineer

The construction of a highway network to serve a large urban area extends over a period of many years and involves the expenditure of several hundred million dollars. Before undertaking this long and expensive task, highway officials should know whether the network will adequately and efficiently accommodate future traffic.

This article describes the use of a high-speed computer in assigning estimated future traffic to a complete network of highways. In the process, traffic volumes are recorded on each individual highway link as well as the turning movements at each intersection. Thus, the engineer can investigate a series of alternate locations or design standards and measure their effect on the entire highway system.

The assignments can be made rapidly, inexpensively, and with a minimum amount of manual work. The program is also consistent and reproducible from one location to another. At the option of the user, provision can be made in the program to allow for one-way street operation, turn restrictions or delays, and peak hour traffic volumes by direction.

FOR many years highway engineers have been faced with the problem of estimating the volume of traffic that would use a proposed new facility. In addition, they would like to know how well this facility will continue to serve traffic in the future. A realistic solution to this problem is not simple. All zones in an entire metropolitan area have the potential of affecting any highway facility. The degree of this effect is governed by the location and design standards of the highway system as it now exists or as it may be subsequently improved. Thus, individual highway improvements can cause a realinement of traffic throughout the area.

Much research work has been done to provide a method for solving these problems. While the future will probably bring added refinements and even major changes in concepts, present knowledge is sufficient to warrant establishing at least an interim procedure for assigning present and future traffic to a network.

The necessity of handling a mass of information in a relatively simple but coherent manner is a characteristic of traffic studies. To process these data in a reasonable time, it has heretofore usually been necessary to use a series of short cuts, approximations, or judgment estimates, each exacting its toll in deviations from accuracy. It is now possible to employ electronic computers to provide a consistent solution, subject only to the limitation of knowledge of the behavior of traffic.

Utilization of Computers

The use of computers does involve the time-consuming operation of preparing a program. While it is desirable to allow flexibility in the program, this is not easily accomplished. Thus, to use an existing program, it is necessary to have access to the machine for which the program is written, to furnish input data in precisely the correct form, to willingly accept the logic that is incorporated in the program, and to accept the results in predetermined format.

The Bureau of Public Roads has written, tested, and used an IBM 705 program to predict the future distribution of trips.[1] The input required is the number of existing zone-to-zone trips and the growth factor for each zone. The program logic follows the Fratar formula. The output is the estimated number of future zone-to-zone trips. Either three different modes of travel or one mode to three different future periods can be processed simultaneously. For a city the size of Washington, D.C., with 500 zones, about one-half hour of IBM 705 time is required for each iteration. Two or three iterations are usually sufficient, thereby making the computer cost of obtaining future zone-to-zone trips somewhat less than $600 for a city the size of Washington.

Procedures for Assigning Traffic

With the present and future zone-to-zone trips available, the next problem is to estimate the loading of these trips on a highway network. To do this, some method of predicting the distribution of traffic between routes is required.

Three types of diversion curves are in current use: the time ratio curve developed by the Bureau of Public Roads (fig. 1), the distance ratio and speed ratio curve used in a Detroit study, and the time and distance differential curve used in California.

Time ratio curve

The Bureau's time ratio curve relates the percentage of trips using a freeway facility based on the ratio of the travel time via the freeway to the travel time via the best alternate route. The percentage of trips using the freeway varies as an S-shaped curve from 100 percent at a time ratio of 0.5 or less to zero percent at a time ratio of 1.5 or more. If the travel time via the freeway is equal to the travel time via the alternate route (time ratio=1.0), approximately 42 percent of the trips are assigned to the freeway.

Speed ratio curve

The speed ratio curves developed for the Detroit area transportation study consist of a family of curves where the percentage of freeway usage is related to speed and distance ratios. These curves are also S-shaped for normal conditions; and with a speed ratio of 1.0 and a distance ratio of 1.0 approximately 45 percent of the trips are assigned to the freeway. Since these curves represent a three-dimensional surface with an undefined mathematical relationship, they are difficult to use in a computer application.

Time and distance curve

The California time and distance curve consists of a family of hyperbolas; and with equal time and distance on the freeway as compared with the best alternate route, 50 percent of the trips are assigned to the freeway. These curves can be expressed in the equation,

$$P=50+50\ (d+\tfrac{1}{2}t)\ [(d-\tfrac{1}{2}t)^2+4.5]^{-\frac{1}{2}}$$

Where:

P=Percentage of freeway usage.
d=Distance saved in miles via the freeway.
t=Time saved in minutes via the freeway.

Problems Encountered in Assigning Traffic

The development of an assignment procedure has two major difficulties: the measurement of the minimum traveltime between each pair of zones over the arterial network and then over the entire highway network including the contemplated freeways, and the accumulation of the assigned volumes on the various segments of the highway system.

The Washington, D.C., Regional Highway Planning Committee, having recently completed an origin-destination study and having predicted the zone-to-zone movements to 1980, wished to assign this traffic to a highway network using the Bureau's time-ratio diversion curve. The effort involved in accomplishing this task for an area the size of Washington was clearly beyond the range of

[1] *Evaluating trip forecasting methods with an electronic computer,* by Glenn E. Brokke and William L. Mertz. PUBLIC ROADS, vol. 30, No. 4, Oct. 1958, pp. 77–87.

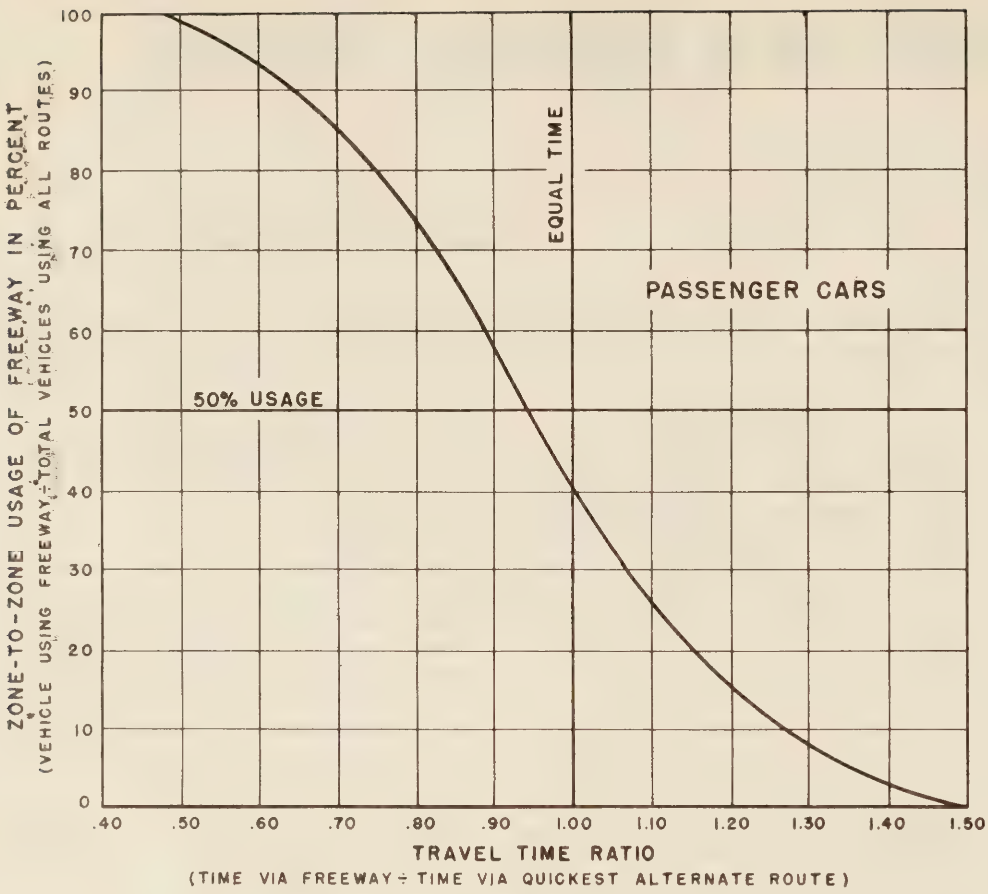

Figure 1.—Time-ratio diversion curve.

practicability, unless an electronic computer could be used for a major portion of the work. The Bureau of Public Roads offered technical assistance. It was soon discovered that no suitable program was available which would handle the complexities of Washington traffic movements. Hence, it became necessary to develop a Washington assignment program.

Development of Minimum Path Principle

At about this time Dr. J. D. Carroll, Jr., director of the Chicago Area Transportation Study, and his staff discovered a method of determining the minimum path through a highway network. Also Dr. A. J. Mayer, director of the Detroit Area Traffic Study, and his staff carried this minimum path principle along somewhat different lines toward an assignment procedure. Both organizations were visited and each was entirely cooperative and responsive in outlining their procedures and ideas on the problem.

Since all diversion curves being considered are based on the relationship between the traveltime (and distance) on the most favorable freeway route and the traveltime (and distance) on the most favorable alternate route, the initial problem is to determine which of the freeway routes and which of the alternate routes are truly the most favorable.

The difficulty of this problem can be appreciated most easily if a rectangular street network is considered. To arrive at a point 4 blocks east and 4 blocks south of an origin, there are over 40 different routes or paths that appear approximately equal to the eye as far as traveltime (or distance) is concerned. However, by accurately adding the time (or distance) values on each of the segments involved for each route, the route with the least overall traveltime can be selected. This selected route is the minimum traveltime route or minimum path.

It is true and probably apparent that the longer the trip, the more alternate routes there are available between two points. For travel across an entire city, there may be literally thousands of alternate paths or routes and the initial problem of determining which path is the minimum appears rather difficult.

Fortunately, Dr. Carroll and his staff were able to use a procedure developed by Edward F. Moore [2] of Bell Telephone Laboratories, Inc. The same basic method is used in this program and essentially consists of accumulating the minimum time and path from a central point to an ever-increasing circle of points surrounding this central point.

Advantages of Minimum Path Principle

The value of the minimum path principle can hardly be overestimated in the solution of the assignment problem. The distance and traveltime on each segment of the highway network are determined and fed into the computer. These initial time and distance measurements are required for any of the methods being used, and are neither easier nor more difficult to obtain for the minimum path procedure. Once the traveltimes are in the computer, however, very substantial advantages begin to accrue.

The most obvious advantage is the savings in man-hours. On the optimistic basis that the best route of travel between a pair of zones can be located and measured in 3 minutes by manual methods, approximately 2 man-years of labor would be required to find the traveltime via the arterial network and via the freeway network for the 40,000 zone-to-zone volumes occurring in the Washington area. The computer can absorb all of this manual work at the rate of about 2 computer-hours being equivalent to 1 man-year of manual computation.

A second advantage is the increase in accuracy and consistency. A manual determination of the best route has two sources of frequent errors. The routing selected as the minimum path may actually be longer than some other path. Secondly, an error may be made in adding the time intervals that make up the selected path. The minimum path program, however, tests all possible paths, selects the minimum, and adds the time intervals unerringly.

A third advantage, which is somewhat more obscure, is the ability of the computer to take additional factors into account. For example, the computer can be rather easily instructed to test a routing and insert a turn penalty whenever a right- or left-hand turn occurs. To add this or a similar complication into a manual procedure would be entirely impractical.

Washington Traffic Assignment Program

The Washington traffic assignment program is in reality a library of programs that can be selected in any desired order through the use of a master control program. To use this library of programs certain conventions must be observed, and to understand these conventions a few definitions are required.

Definitions

Node.—A node is any specific point on the highway system that is needed for identification purposes. Primarily, nodes are used to designate zone centroids and highway intersections. They are identified by number.

Link.—The portion of the highway system between two nodes is a link. To avoid needless complication only the "through" or more important highways are identified by actual location. Links are identified by the two node numbers which terminate the link.

Route.—A group of connecting links between a pair of zones is the route of travel between these zones. If a particular route has a shorter traveltime than any other

[2] *The shortest path through a maze,* by Edward F. Moore. Paper presented at the International Symposium on the Theory of Switching, Harvard University, 1957.

route, it is called the minimum time path. If distance instead of time were the criterion, a minimum distance path could be similarly described.

Tree.—All minimum path routes from one particular node to all other nodes in the system constitute the tree for that particular node. In practice, trees need to be built only for the zone centroid nodes.

Program conventions and limitations

1. No more than four links may meet at any node. To accommodate five or more links which would otherwise intersect at a single node, it is necessary to separate the one node into two (or more) nodes with zero time and distance between them.

2. No node may be numbered more than 4,000.

3. The node numbers must be arranged in sequence in four separate groups in the following order: *group A*, zone centroids starting with number 1; *group B*, four-way arterial nodes; *group C*, two- or three-way arterial nodes; and *group D*, freeway nodes.

4. From each four-way node there must be at least one link to a numerically larger node number.

5. The zone-to-zone trip cards must be in sort by the first zone before being placed on tape.

6. To be able to insert a turn penalty for right and left turns, it is necessary to designate each link as positive or negative. Movements between links of the same sign involve no turn. To accommodate diagonal or curving streets, a flag position is also available to change signs as needed.

The Washington traffic assignment program library consists of the following individual programs:

0—Master control.
1—Build trees.
2—Load arterial network (all or nothing).
3—Load entire network (time ratio curve).
4—Sum vehicle-miles and vehicle-hours.
5—Convert link data from decimal to binary.
6—Make freeway corrections to link data.
7—Convert trip volumes from decimal to binary.
8—Correct trip data.
9—Prepare time-ratio diversion table.

These programs are on magnetic tape and may be used in any desired order through the master control program.

Input Data Requirements

Zone-to-zone trips

Each of the zone-to-zone trip volumes must be represented by a trip card identifying the two zones (or stations) involved and the number of trips between them. Zero volumes need not be represented by a card except that each zone other than the last one must be represented by at least one card (zero volume if necessary) when arranged in sort by the first identifying zone number. If not already accomplished in a previous stage, the zones and stations must be renumbered to form an unbroken sequence starting with number 1. When placed on tape, the zone-to-zone cards must be in sort by the first identifying zone number.

Highway link data

The highway network must be described properly for computer application. This is accomplished by listing each link of the highway network on a coding sheet. The listing consists of the two identifying nodes together with the sign, a flag if needed, the traveltime in minutes and hundredths, and the distance in miles and hundredths. These data are then key punched with one link to a card. The cards are then duplicated, reversing the two identifying nodes and adjusting the sign and flag, so that in the final deck each link is actually listed on two cards. The data on the cards are then transferred to magnetic tape.

Program Operation

Program 0, master control

This control sets the parameters for the specific area in which traffic is being assigned and permits the choice of any of the subroutines included in the program library. As input, it requires the number of zone centroids, the number of four-way intersections, the number of two- or three-way intersections, the number of freeway intersections, and the amount of turn penalty in minutes and hundredths.

In addition, it has been necessary to scale the time and distance values into units which will economically use the memory availability of the computer. Therefore, the maximum traveltime on any link plus the turn penalty is equated to 63. The time values on all other links are converted to sixty-thirds of this maximum traveltime link. For the same reason the maximum distance link is likewise equated to 63, and all distances converted to sixty-thirds of this maximum distance link. It should be noted that the maximum traveltime link and the maximum distance link need not be the same link.

The maximum traveltime link and the maximum distance link are also necessary inputs to the master control program. All of the other programs are then set with the specific characteristics entered through this master control program.

Program 5, convert link data from decimal to binary

In program 5 the computer edits the link data for impossible codes, scales the time and distance values to appropriate units, converts all data from decimal to binary, and packs the information to fit exactly into a block of computer memory. The output is a binary coded tape containing this large block of information.

Program 6, make freeway corrections to link data

The most difficult problem in determining the freeway route is to arrive at some method which will compute a minimum freeway path even though it is longer than an arterial street path. This is necessary because some diversion to a freeway exists even if the time ratio is more than 1.0. To retain the advantage of the minimum path method and still obtain a freeway time longer than arterial time, it was decided to temporarily halve the time on the freeway links. Once the tree has been established, the time values are corrected.

In program 6 the previous arterial links are modified as needed by the addition of the freeway nodes; the freeway links are inserted into the system with their time values cut in half; and the information is converted to binary and packed into a block of memory. The memory is then written out on tape in binary code.

Program 7, convert trip volumes from decimal to binary

The tape containing the zone-to-zone trip volumes is edited in program 7. The numbers are converted from decimal into binary, and all of the trips from the first-listed (or origin) zone to all other zones are packed into one record block which is written out on tape. There must be a trip record block for each zone except the last (highest numbered) zone.

Program 8, correct trip data

If in the process of editing or through subsequent checking it is found that some of the zone-to-zone trips are in error, the values may be corrected in program 8 without rerunning the entire program.

Program 9, prepare time-ratio diversion table

Program 9 builds the diversion curve table for converting time ratio to percentage diversion. At the present time the traffic diversion curve plotted in figure 1 is incorporated in the program.

Program 1, build trees

Program 1 determines the minimum path from each zone to all nodes in the highway network. If only the arterial links are used, the program builds arterial trees. If the freeway links are also included, the program builds freeway trees. Thus, the previously prepared link data are the input for this program.

The program instructs the computer to set aside a block of memory for the tree with each memory word of the block corresponding to an actual node on the highway system. The memory words in the block are in the same sequence as the node numbers. Thus the position of the memory word identifies the node number.

In addition, each word in the tree memory block will contain two major items of information: (1) the preceding node through which the route has passed in building the tree, which is called the back node, and (2) the total elapsed time from the tree centroid to the node represented by this memory word.

Each memory word is initially set to the largest possible value. The computer then starts building the tree from zone centroid 1 in the following manner:

A. Since the tree is being built from node 1, the first step is to set the back node and the

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elapsed time in memory word No. 1 to zero. At the same time this node is listed in an elapsed time sequence table in the zero time slot.

B. The computer then takes the minimum entry in the elapsed time table, erases this entry, and from the link memory block, finds all links that emanate from this node, which can be called node A.

C. At the end of each of these links there is a second node which can be called node B. The link time from node A to node B, plus a turn penalty if required, is added to the total elapsed time at node A to give the total elapsed time from the tree centroid to node B. The machine compares the computed elapsed time at node B with the previously established elapsed time stored in the word represented by node B in the tree memory. If the new time is less than the stored time, it replaces the stored time and node A replaces the previously stored back node. At the same time node B is stored in the elapsed time sequence table in the appropriate time slot. If, however, the new time is equal to or greater than the previously stored elapsed time, the route is not a minimum path and the computer discards this value.

D. When all of the links emanating from node A have been completed in this manner, the computer again selects the minimum entry in the elapsed time sequence table and repeats the process.

E. When all values in the elapsed time sequence table have been used, the tree from zone centroid No. 1 has been completed and the tree memory is written out on tape.

F. The computer then proceeds to zone 2 and builds the tree from this zone in exactly the same manner.

G. When trees have been built from all zone centroids, the arterial tree routine has been completed. For the Washington, D.C., area, about 450 trees are needed.

The program has been written so that any freeway pattern may be superimposed on the arterial street network without destroying the arterial trees already developed. The freeway trees are built in exactly the same manner as the arterial trees, except that the freeway links as well as the arterial links are included in the input data.

Program 3, load entire network (time ratio curve)

The previously completed freeway and arterial tree tapes and the zone-to-zone trip tape become input for program 3. In addition, the relationship between time ratio and percentage diversion is placed in the computer memory. The program then performs the following operations:

A. The arterial tree for node 1 (also zone centroid 1) is read into a block of memory.

B. The freeway tree for node 1 is read into a separate block of memory.

C. The zone-to-zone trips from zone 1 are read into a third block of memory.

D. The trips between the first pair of zones initiate the following action: (1) The destination zone of the trip becomes the first entry in the arterial route. (2) From the arterial tree, the back node of the destination zone becomes the second entry in the arterial route. (3) The back node of the second entry becomes the third entry for the route and so on until the route reaches zone centroid 1. (4) The freeway route is established in the same manner with the corrections to travel-time on the freeway links being made. (5) The arterial route is compared with the freeway route. (*a*) If the routes are identical, all of the trips are accumulated on the arterial routing in a block of memory where each word represents a corresponding highway link. (*b*) If the routes are different, the two points of choice are determined. The travel-time via the freeway and via the arterial system between points of choice is computed and converted to time ratio and then to percentage diversion. The freeway traffic is accumulated in memory via the freeway route, and the arterial traffic is accumulated in memory via the arterial route. (6) At all four-way intersections, two of the turning movements are recorded separately in a turn table so that in the final analysis all turning movements are available.

E. The remaining trips from zone 1 are handled in the same manner after which the trees and trips from zone 2 replace those of zone 1 and the process is repeated.

F. This procedure is continued until all zone-to-zone trips have been read by the computer, at which time the accumulated volumes are written on tape in decimal form. The decimal tape is printed on peripheral equipment. The printed output is the traffic load on all segments of the entire network including all turning movements.

Table 1.—Computer time required to run through the various programs in the library

Program	Units: Nodes	Units: Zone-to-zone cards	Total computer time	Rate
			Sec.	
Convert links to binary	543	-----	25	¾ minute per 1,000 nodes.
Convert volumes to binary	---	3,488	60	3 minutes per 10,000 cards.
Build trees (102 zones)	543	-----	280	8½ minutes/100 zones/1,000 nodes.
Load network (time ratio)	---	3,488	275	13 minutes per 10,000 cards $\times\left[\frac{\text{Number of nodes}}{500}\right]^{\frac{1}{4}}$
Load network (all or nothing)	---	3,488	97	4⅔ minutes per 10,000 cards $\times\left[\frac{\text{Number of nodes}}{500}\right]^{\frac{1}{4}}$
Compute vehicle-miles and vehicle hours [1]	543	-----	35	1 minute per 1,000 nodes.

[1] Includes conversion of link volumes and turning movements from binary back to decimal, written out on tape in an appropriate format.

Program 2, load arterial network (all or nothing)

The library also includes a program for loading all trips on the shortest route. This is accomplished by reading only the arterial trees into memory and loading all trips on the routing established by these trees.

Program 4, sum vehicle-miles and vehicle-hours

The vehicle-miles and vehicle-hours on the freeway network, on the arterial system, and on the local system are then computed and printed.

Computer running time

The entire program library has been completed and a major portion tested in the Virginia portion of the Washington metropolitan area. This area consists of 102 zones, 543 nodes, and 3,488 zone-to-zone movements. The time required to run through the various programs in the library is shown in table 1.

Using the rates given in table 1, it is possible to estimate the computer time required for one complete assignment for any area. For example, it is expected that the Washington, D.C., metropolitan area will consist of about 450 zones, 3,100 nodes, and 40,000 zone-to-zone trip cards. The estimated computer time for this area is shown in table 2. An

Table 2.—Estimated computer time required for 1 complete assignment of traffic in the Washington, D.C., metropolitan area

Program	Units: Nodes	Units: Zone-to-zone cards	Total computer time	Rate
Convert arterial links to binary	3,100	----	3 min	¾ minute per 1,000 nodes.
Convert freeway links to binary	3,100	----	3 min	¾ minute per 1,000 nodes.
Convert volumes to binary	---	40,000	12 min	3 minutes per 10,000 cards.
Build arterial trees (450 zones)	3,100	----	120 min	8½ minutes/100 zones/1,000 nodes.
Build freeway trees (450 zones)	3,100	----	120 min	8½ minutes/100 zones/1,000 nodes.
Load network (time ratio)	---	40,000	90 min	13 minutes per 10,000 cards $\times\left[\frac{\text{Number of nodes}}{500}\right]^{\frac{1}{4}}$
Compute vehicle-miles and vehicle-hours.	3,100		3 min	1 minute per 1,000 nodes.
Total computer time	-	----	5 hr. 51 min.	
Set up and control time	-	----	10 min	
Contingency time (10 percent)	-	----	34 min	
Grand total computer time	-	----	6 hr. 35 min	

IBM 704 computer with 32,000 words of memory will cost from $350 to $400 per hour. Thus the machine cost for one assignment for the Washington area will range from $2,200 to $2,600.

Characteristics of Traffic Assignment Program

The accuracy of the assignment program rests basically upon the accuracy of the assumption that traffic divides between routes in accordance with the time-ratio diversion curve. In any particular city, the accuracy of this assumption can be checked by assigning present trips to the existing highway system and checking against current traffic counts. If better criteria are subsequently developed which improve the distribution of traffic among routes, they will be incorporated into the program.

The program as written has a considerable amount of flexibility. Changes in the extent or location of the proposed freeways can be tested by merely altering the freeway network and rerunning the program. If any of the proposed highway segments are loaded beyond capacity, the traveltimes on these sections can be adjusted and the program rerun until there is a balance between capacity and traveltime on each segment. If directional zone-to-zone trips are available, the program can give directional assignments. Thus, traffic on one-way streets or ramps can be directly computed.

It is likely that the future use of the program will develop additional subroutines which will be useful in designing highways. By way of illustration, consider the case of a ramp connection immediately before an interchange between freeways. If the predominant flow of traffic from the ramp turns left at the freeway interchange, it may be advantageous to bring the ramp into the freeway from the left to avoid the confusion of weaving this traffic across the freeway. By suitable instructions, the computer can develop these or similar data.

The program is written for an IBM 704 computer with 32,000 words of memory. If there are less than 900 nodes in the highway system, an 8,000-word memory will be sufficient. In addition to memory capacity, it is essential that the computer have extremely fast access time to all memory positions. This consideration, at least for the present, precludes the use of computers which rely on a magnetic drum memory.

Acknowledgment

The development of this traffic assignment program represents the efforts of a number of specialists in the field. Besides Dr. Carroll and Dr. Mayer, who are mentioned earlier in the article, valuable contributions were made by the staff of the General Electric Computer Laboratory, Tempe, Ariz.; Mr. W. F. Boardman, Acting Technical Director, Regional Highway Planning Committee, Washington, D.C.; and Mr. W. L. Mertz, Bureau of Public Roads.

Use of Backwater in Bridge Design

(*Continued from page 226*)

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Vehicle Acceptance Rates of Parking Areas

BY THE DIVISION OF TRAFFIC OPERATIONS
BUREAU OF PUBLIC ROADS

Reported by ARTHUR A. CARTER, Jr.,
Highway Research Engineer,
and BESSIE A. HAYES,
Statistical Assistant

VEHICLE acceptance rates of parking facilities, or more specifically, the number of entering vehicles that can be accommodated in a given period of time by a single entrance, are of primary importance in parking lot operation. In recent years there has been considerable research on this subject in downtown areas, including outdoor parking lots, and parking structures of various types, as well as studies of curb parking usage. Consequently, curb parking habits and the vehicle acceptance rates of city parking lots and structures are quite well known.

Considerably less research effort, however, has been devoted to vehicle acceptance rates at parking facilities in suburban and rural areas. In these outlying areas parking facilities are quite large and, except for those at suburban shopping centers, are used primarily for long-term parking. They include parking areas at industrial plants, racetracks, military installations, stadiums, and similar places. Generally, there is little turnover of vehicles at these parking areas. Instead, a concentrated influx of vehicles occurs over a relatively short period, with a similar discharge surge at a later time. Few detailed studies have been made of the traffic flows into such parking areas, although many overall counts have been made of the total number of vehicles accommodated during the heavy influx period.

Early in 1958, as one of its responsibilities in connection with civil defense highway planning, the Bureau of Public Roads was requested to determine the vehicle acceptance rates of large parking areas comparable to those which might be provided at refugee reception centers. This presented an ideal opportunity for the Bureau to accomplish two purposes with one study. The results would fill the existing void in available information regarding vehicle acceptance rates of outlying parking areas, and provide the information needed for emergency planning. For the first purpose, field studies of the operation of large improved parking areas were essential. Secondly, operations at these improved lots could be accepted as comparable to the best operation expected at refugee centers, since most of these centers would have relatively unimproved parking areas such as large open fields.

Accordingly, after Bureau of Public Roads personnel had made pilot studies near Washington, D.C., the cooperation of the State highway departments was enlisted in conducting studies at major parking areas throughout the country. The response was excellent, with 24 States supplying data obtained at a total of 74 parking areas. Included were studies at 125 entrances, of which 71 were found to include capacity conditions suitable for analysis.

Scope of Study

The data received were classified according to five types of entrances, depending upon the driving maneuver involved in the approach to the parking lot. The types were as follows: (1) straight approach (no turning movement); (2) 90° right turn; (3) 90° left turn; (4) oblique angle turn, right; and (5) oblique angle turn, left. Within each category, the lot entrances were further identified either as familiar to the drivers using them, such as those at industrial plants, or as unfamiliar lot entrances, which were infrequently visited by individuals, such as those provided for various special events.

Except for a few special reports, the analyses were based entirely upon the determination of approach volumes per hour per lane which actually entered the parking facility. Where over 50 nonentering vehicles per hour were intermingled in an approach traffic flow, or where opposing traffic was significant, the study was rejected. From each individual lane study, the peak-hour and the peak 10-minute volumes were recorded, and the peak 10-minute rate was expanded to a full-hour rate.

Observations made during the pilot study, as well as comments made by several of the observers in the field, indicate that an acceptance rate attainable for a full 10 minutes can be maintained for a full hour, provided that adequate maneuvering and parking space remains available within the lot and that a continuing capacity flow of vehicles arrives. This presumably would be the situation at a refugee reception center, although at the facilities studied, peak arrival rates seldom lasted for more than a half hour. Results are therefore given in terms of vehicles per hour per lane, as obtained by expanding the peak 10-minute rates (table 1). They are average values considered to be typical for the conditions shown.

Results of Study

Straight approach entrances

In the 11 studies involving unfamiliar, straight approach entrances, vehicle acceptance rates ranged from 770 to 940 vehicles per hour per lane. The average rate was 850 vehicles per hour per lane. Most drivers at this type of entrance, as well as most other unfamiliar entrances, came to a near stop in their approach.

Studies at familiar, straight approach entrances showed an average acceptance rate of 1,100 vehicles per hour. However, extremes to this average were reported. A rate of only 600 vehicles per hour was found at one industrial plant parking location. At another plant location, where an estimated speed of 20 m.p.h. was maintained through the entrance and vehicles could spread out into several lanes beyond the entrance, a rate of 1,900 vehicles per hour was recorded. This rate is somewhat higher than the predicted possible capacity of a single traffic lane at 20 m.p.h. The study, however, was made at an unusually well-operated industrial plant parking lot where roadway speed could be maintained through the gates and where parking had become a daily routine to all drivers.

90° turns

The average entrance rate at unfamiliar parking areas involving a 90° right turn was about 750 vehicles per hour. A one-third

Table 1.—Vehicle acceptance rates of large parking areas in 24 States

Approach to entrance	Number of studies	Average acceptance rates—vehicles per hour per lane	
		Unfamiliar entrance [1]	Familiar entrance [2]
Straight approach (no turning movement)	20	850	1,100
90° right turn	15	750	1,000
90° left turn	24	830	900
Oblique angle, right	8	650	1,000
Oblique angle, left	4	720	(3)

[1] Includes racetracks, stadiums, and other facilities not frequently visited by the same individuals.
[2] Includes industrial plants, military bases, and other facilities where the same drivers enter daily.
[3] No data available.

increase in the flow, to 1,000 vehicles per hour, resulted where drivers were familiar with the parking location.

At entrances involving 90° left turns, the results were surprisingly high in several studies at both familiar and unfamiliar locations. While volumes at several entrances were in the 600–700-vehicle-per-hour range, observers at other parking areas reported rates from 1,100 to 1,300 vehicles per hour, even though speeds were estimated as 10 m.p.h. or lower. One study location was particularly comparable to refugee center conditions. At this location, where vehicles from one lane rather promiscuously crossed a shallow ditch at various points to enter a grass lot, a rate of 850 vehicles per hour was attained. This was approximately equal to the average of 830 vehicles per hour for all of the unfamiliar lots, which indicates that unimproved lots can operate just as well as improved lots. The rate at familiar entrances showed only slightly better operation, averaging 900 vehicles per hour.

The results of the study show that the average left-turn rate was somewhat higher than the average right-turn rate at unfamiliar lots. This is contrary to the usual finding in intersection capacity studies that left turns involve more delay than do right turns. At the average intersection, drivers in making left turns are often confronted with opposing through traffic. In the parking studies, drivers were not faced with opposing traffic in negotiating left turns. Under these circumstances, left turning movements might be made from either lane of a two-lane highway, depending upon whether the approach road was operating one-way or two-way with practically no opposing traffic. When left turns are made from the left lane, the turn is just as sharp as the usual right turn, and thus the acceptance rates for either right or left turns should be similar. When left turns are made from the normal right-hand approach lane, a wider turn is involved which can be made much more easily. This very likely explains the more favorable rate for 90° left turns reported in table 1 for unfamiliar entrances as compared with 90° right turns.

Oblique angle entrances

Limited data obtained for oblique angle entrances indicated that left-turn rates again were somewhat higher than those for right turns. At unfamiliar locations, left turns averaged 720 vehicles per hour, while right turns averaged only 650 vehicles. At familiar locations, only data for right oblique entrances were available. These showed an average flow of 1,000 vehicles per hour. It can be assumed that left oblique entrances would approximately equal this capacity.

Effect of Gatemen on Traffic Flow

When these studies were being planned, it was assumed that collection of fees or inspection of passes would have an adverse effect on the traffic flow. The results, however, showed no consistent difference between lots where fees were collected or passes checked (applicable to about one-third of the parking areas studied) and those where access was unrestricted. At the average lot, entrance speeds were very low regardless of whether or not gatemen were present. Similarly, at these low speeds, it was found that lane width was not significant. All approach and entrance lanes studied were at least 10 feet wide, so it is possible that narrower rural roads might yield lower rates.

Practical Acceptance Rate

As a result of these studies, it appears that 800 vehicles per hour per entrance lane could be considered the practical maximum vehicle acceptance rate for large unfamiliar parking areas, regardless of the turning movement involved. This rate is possible if adequate parking areas exist to absorb all entering vehicles, if attendants prevent congestion within the lot, if the feeder roadway has a continuous backlog of vehicles, and if disabled vehicles can be rapidly cleared from the highway. It should be emphasized that dry weather conditions must prevail to accommodate 800 vehicles per hour per lane where unpaved entrances and parking areas are involved. Rain or snow could easily make such areas totally unusable.

If all drivers are familiar with an entrance, a rate of at least 1,000 vehicles per hour, and frequently more, may be possible. In those unusual situations where well designed straight-through approaches permit traffic to maintain full highway speed until distribution to specific sections within the lot begins, it may be possible to attain the maximum possible capacity of a moving traffic lane, 2,000 vehicles per hour. Certainly, however, this figure should not be used in planning.

It should be noted that studies were made only of traffic entering the various facilities before events, since this was the movement considered applicable to civil defense planning. Only limited data were received regarding exit flow from these same facilities, but it was evident that exit rates were somewhat higher than approach rates.

New Publication

The *Catalog of Highway Bridge Plans* has been assembled by the Bureau of Public Roads primarily for use by highway departments to facilitate the exchange of designs on a national basis. The program for the exchange of bridge designs was suggested by many State highway departments and by committees and members of the American Association of State Highway Officials.

The States were requested to submit their available bridge designs dating from 1950 for H20–S16 loading or heavier. The publication is divided into 13 sections and lists in tabular form over 4,500 designs. The data are assembled by States according to span types, simple, continuous, or cantilever, in their relevant sections and cover rolled beam spans with and without composite action, steel deck girder spans riveted or welded, with and without composite action, steel truss spans, reinforced concrete slab, deck girder, box girder, and rigid frame spans, prestressed concrete slab, girder and box girder spans, steel arch spans, reinforced concrete arch spans, movable bridges of lift span and bascule span types, suspension bridges, pedestrian overcrossing structures, traffic sign structures, and tunnels.

The last section (XIII) contains quantity charts which may prove useful in making design comparisons. These charts show structural steel quantities for rolled beam, deck plate girder and truss spans, and concrete quantities for reinforced concrete spans. Insufficient data were available to prepare charts covering reinforcing steel.

The *Catalog of Highway Bridge Plans*, a 167-page publication, may be purchased from the U.S. Government Printing Office, Washington 25, D.C., at $1 per copy.

Traffic Signals and Accidents in Michigan

BY THE TRAFFIC OPERATIONS DIVISION
BUREAU OF PUBLIC ROADS

Reported[1] by DAVID SOLOMON
Highway Research Engineer

TO the general public, the installation of a traffic signal often implies that accidents will be substantially reduced. Studies have frequently shown, however, that this is not always true. One such study was recently conducted in Michigan. From 1946 to 1957 the Michigan State Highway Department made a detailed accident study at each of 89 intersections where traffic signals were installed. The intersections, distributed throughout the southern half of the State, were located in and near cities as well as on rural highways. Data were collected at each intersection for 1 or 2 years prior to installation of the signals and for an equivalent period afterwards.

The types of intersections included in the study ranged from simple three-leg to five- and six-leg and other complex intersections. As might be expected, more than half of the intersections had four approach legs without medians or islands to separate opposing streams of traffic. The four-leg divided intersections were, in most cases, divided on the main highway only. Of the 89 intersections studied, 39 were signalized with standard stop-and-go controls and 50 with flashing beacons. These beacons flashed yellow on the main street or major highway, and red on the side street or minor highway.

At 38 of the 89 intersections, traffic data were available. The average daily traffic volumes entering from all approaches at these intersections averaged 20,200 vehicles a day for stop-and-go signals and 8,000 vehicles a day for flashing beacons.

[1] Acknowledgment is made to the Michigan State Highway Department for furnishing the data for this article.

Summary of Findings

At intersections where stop-and-go signals were installed, the number of accidents increased nearly one-fourth. It was found that the simpler the intersection, the greater the increase in accidents. For example, simple three-leg intersections showed a gain in accidents of 78 percent, whereas five- and six-leg and other complex types showed a decline of 47 percent. In the case of flashing beacon installations, the number of accidents decreased regardless of the type of intersection, the range being 21 to 37 percent and averaging 26 percent.

The number of persons injured decreased by one-fifth at intersections where stop-and-go signals were installed and by one-half in the case of flashing beacon installations. The number of fatalities also decreased after either of the two types of signals were installed.

After installation of stop-and-go signals, the number of rear-end, head-on, and side-swipe collisions increased 200, 157, and 74 percent, respectively. Only angle and miscellaneous types of collisions decreased. The installation of flashing beacons resulted in a nearly uniform reduction of about one-fourth for each type of collision.

In the investigation of the effect of light and weather conditions on the accident rate following stop-and-go signal installation, it was found that the number of accidents during inclement weather increased fourfold as compared with accidents during all-weather conditions. This relation applied to both daytime and nighttime driving. After installation of flashing beacons, daytime accidents during inclement weather as well as during all-weather conditions decreased by one-fift[h]. On the other hand, nighttime accidents durin[g] inclement weather decreased by one-sixth a[s] compared with a decrease of one-third f[or] all-weather conditions.

At 38 intersections where traffic volum[e] data were available, it was found that th[e] greatest reduction in accident rates occurre[d] at the higher volume intersections for sto[p]-and-go signals and at the lower volume inte[r]sections for flashing beacons.

Accident Experience

At 39 intersections where stop-and-go sig[nals] were installed, the number of accident[s] increased 23 percent (table 1 and fig. 1). On the other hand, at 50 intersections wher[e] flashing beacons were installed, a 26-percen[t] reduction in accidents was recorded.

The greatest increase in the number o[f] accidents after stop-and-go signals were in[]stalled was observed at three- and four-le[g] undivided intersections (table 1 and fig. 2). The four-leg divided intersections showe[d] practically no change, and at the more com[]plex five- and six-leg intersections the num[]ber of accidents declined by 47 percent.

Flashing beacon installations resulted in [a] reduction in the number of accidents rangin[g] from 21 percent at four-leg undivided inter[]sections to 37 percent at five- and six-leg inter[]sections. Compared with stop-and-go signals this is a relatively narrow range and n[o] general pattern was evident among the fou[r] types of intersections.

Both types of signal installations brough[t] about a reduction in the number of person[s] injured: 20 percent for stop-and-go signal[s] and 50 percent for flashing beacons. Fatal[]

Table 1.—Number of accidents, injuries, and fatalities before and after traffic signal installation at 89 intersections

Number of intersections	Type of intersection	Number of accidents				Number of injuries				Number of fatalities			
		Before signal installation	After installation of—			Before signal installation	After installation of—			Before signal installation	After installation of—		
			Stop-and-go signal	Flashing beacon	Percentage change		Stop-and-go signal	Flashing beacon	Percentage change		Stop-and-go signal	Flashing beacon	Percentage change [1]
3	3-leg	37	66	-------	78	27	26	------	[2]−4	3	0	------	------
13	3-leg	114	-------	78	−32	78	-------	34	−56	3	------	4	------
23	4-leg, undivided	182	293	-------	61	145	174	------	20	7	4	------	[2]−43
29	4-leg, undivided	219	-------	174	−21	176	-------	94	−47	21	------	6	−71
9	4-leg, divided	158	165	-------	[2]4	136	91	------	−33	2	1	------	------
5	4-leg, divided	50	-------	35	−30	42	-------	16	−62	2	------	2	------
4	5- and 6-leg and other types	85	45	-------	−47	70	13	------	−81	2	2	------	------
3	5- and 6-leg and other types	19	-------	12	[2]−37	15	-------	11	[2]−27	2	------	1	------
39	All types	462	569	-------	23	378	304	------	−20	14	7	------	−50
50	All types	402	-------	299	−26	311	-------	155	−50	28	------	13	−54

[1] Too few data to compute percentage change, except for 4-leg undivided and all types of intersections.
[2] Percentage change is not statistically significant.

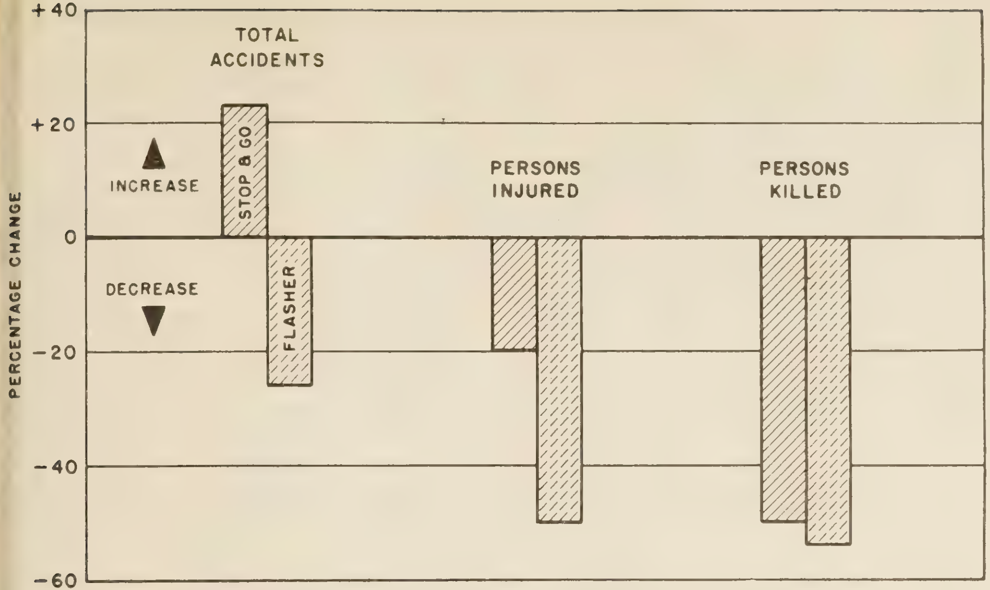

Figure 1.—Percentage change in the number of accidents, injuries, and fatalities after installation of traffic signals.

ities also decreased after installing either type of signal, but no definite conclusions should be drawn from this experience because of the limited sample.

After stop-and-go signals were installed, the number of persons injured remained fairly constant at three-leg intersections; increased slightly at four-leg undivided intersections; decreased moderately at four-leg divided intersections; and decreased substantially at five- and six-leg and other complex types of intersections. This trend was quite similar to that found for the number of accidents. At flashing beacon installations there was a general decrease in the number of persons injured at all types of intersections, but no definite pattern was evident.

The data developed in this study indicate that stop-and-go signals are less effective from the safety standpoint at simple types of intersections, but for the more complex types they may be desirable. It appears that greater consideration should be given to the use of flashing beacons at three- and four-leg intersections where some type of signal is required.

Accident Exposure Considered

Up to this point exposure to accidents at the various intersections has not been considered. At 38 intersections where traffic volume data were collected, it was found that volumes increased 11.1 percent after installing stop-and-go controls and 11.5 percent after installing flashing beacons. These percentage increases applied generally to the different types of intersections studied.

Accident, injury, and fatality rates based on 100 million vehicles entering the intersections are shown in table 2. In general, the trends observed in table 2 for the 38 intersections, after considering the rate of exposure to accidents, were similar to those shown in table 1 for all 89 intersections. There is one important exception, however. In the case of flashing beacon installations, the percentage decrease in the number of accidents remained fairly constant for the 50 intersections reported in table 1, regardless of the type of intersection studied, but for the 25 intersections reported in table 2 the percentage decrease became progressively greater with intersection complexity. For stop-and-go signal installations, the relative changes in the number of accidents followed a similar trend in both tables 1 and 2. Whether the traffic volume factor is considered or not, it is immediately evident that stop-and-go signals are very effective in reducing accidents at the most complex types of intersections.

Traffic volumes at stop-and-go signal installations were on the average two or three times greater than the volumes at intersections having flashing beacons. In order to provide a meaningful comparison of the effectiveness of stop-and-go signals and flashing beacons, intersections with like traffic volumes were selected. Since four-leg undivided intersections predominated in the study, the comparison has been narrowed to this type in table 3. By grouping the 4 lowest traffic volume intersections having stop-and-go signals and the 4 highest volume intersections having flashing beacons, it was possible to select 2 groups of 4 intersections for which the average daily volumes approximated 16,000 vehicles.

The percentage change in accident and injury rates was computed for these eight intersections and for all four-leg undivided intersections (table 3). It is seen that installation of stop-and-go signals resulted in a substantial increase in the accident rate and a somewhat smaller increase in the injury rate. Flashing beacons, however, brought about a slight decline in the accident rate and a substantial decline in the injury rate. The pattern is the same whether the 8 intersections or all 22 intersections are compared.

In a further attempt to determine the effect of traffic volumes on accident rates, the stop-and-go controlled intersections and those with flashing beacons were each divided into two groups: intersections with traffic volumes below the average for the group and those above the average (table 4). Averages for the two groups were 20,200 ADT for stop-and-go signals, and 8,000 ADT for flashing beacons.

At stop-and-go controlled intersections, the

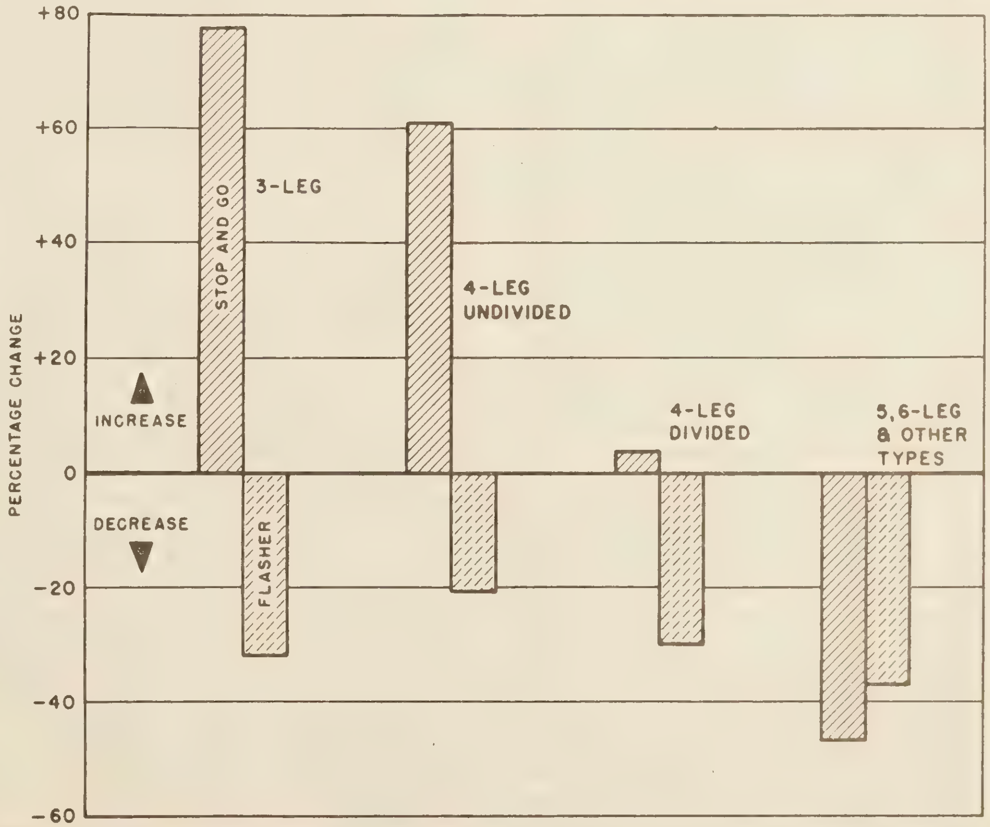

Figure 2.—Percentage change in the number of accidents, by type of intersection, after installation of traffic signals.

Table 2.—Number of accidents, injuries, and fatalities per 100 million vehicles entering before and after traffic signal installations at 38 intersections

Number of intersections	Average daily traffic	Type of intersection	Accidents: Before signal installation	Accidents: After installation of— Stop-and-go signal	Accidents: After installation of— Flashing beacon	Accidents: After installation of— Percentage change	Injuries: Before signal installation	Injuries: After installation of— Stop-and-go signal	Injuries: After installation of— Flashing beacon	Injuries: After installation of— Percentage change	Fatalities: Before signal installation	Fatalities: After installation of— Stop-and-go signal	Fatalities: After installation of— Flashing beacon	Fatalities: After installation of— Percentage change [1]
2	11,800	3-leg	172	297	------	73	179	116	------	−35	20.7	0	------	------
5	6,700	3-leg	190	------	178	−6	108	------	94	−13	6.3	------	5.6	------
7	20,000	4-leg, undivided	130	199	------	53	62	76	------	23	1.6	1.5	------	------
15	8,200	4-leg, undivided	172	------	150	−13	134	------	74	−45	22.6	------	6.1	------
3	27,200	4-leg, divided	135	114	------	−16	118	29	------	−75	2.8	0	------	------
2	7,800	4-leg, divided	373	------	290	−22	280	------	157	−44	0	------	24.1	------
1	16,900	5- and 6-leg and other types	412	129	------	−69	471	81	------	−83	29.4	16.1	------	------
3	9,000	5- and 6-leg and other types	188	------	101	−46	150	------	92	−39	19.8	------	8.4	------
13	20,200	All types	153	182	------	19	117	67	------	−43	5.9	1.5	------	−75
25	8,000	All types	193	------	160	−17	143	------	87	−39	17.6	------	7.7	−56

[1] Too few data to compute percentage change for the different types of intersections.

accident rate increased 32 percent at the 8 intersections with below-average traffic volumes, and decreased 2 percent at the 5 intersections with above-average volumes. A similar comparison for flashing beacon installations showed a 22-percent decrease in the accident rate for 17 intersections having below-average traffic volumes and a 10-percent decrease for the 8 intersections with above-average volumes. Injury rates followed a declining trend, irrespective of traffic volumes or type of signal. Accident data reported in table 4 indicate that flashing beacons should be employed at low traffic volume intersections and stop-and-go signals at high volume intersections.

Traffic Signals Affect Types of Collisions

Types of collisions and the percentage change before and after installing stop-and-go signals and flashing beacons are reported in table 5 and illustrated in figure 3. Following installation of stop-and-go signals at 39 intersections, 3 types of collisions increased substantially: Rear-end collisions increased 200 percent; head-on collisions, 157 percent; and side-swipe collisions, 74 percent. Only angle collisions and other miscellaneous accidents showed a decrease. The 50 intersections with flashing beacons presented an entirely different picture. All types of collisions declined, ranging from 18 percent for rear-end collisions to 32 percent for the head-on type.

In the before period, angle collisions were dominant, but after installing stop-and-go signals, rear-end collisions ranked highest. However, after flashing beacons were installed, angle collisions still predominated, but there was a 29-percent decline in the number.

Since stop-and-go signals tend to reduce angle collisions, it follows that where this type of accident is a substantial percentage of the total, stop-and-go signals are more likely to be effective. Traffic engineers in the past have used this criteria in deciding whether a traffic control signal should be installed at a particular intersection.

Light and Weather Conditions

Table 6 shows that after stop-and-go signals were installed, the number of accidents increased regardless of light or weather conditions, with the greatest increase occurring during periods of inclement weather. During daylight hours, such accidents increased 126 percent as compared with an increase of only 32 percent for all-weather conditions. Similarly, accidents occurring at night during inclement weather increased 41 percent, as compared with an increase of only 11 percent for all-weather conditions.

With flashing beacons, the number of accidents decreased under all-weather conditions, both day and night. During daylight hours and inclement weather, the number decreased 20 percent as compared with an identical decrease for all-weather conditions. At night and during inclement weather, the number of accidents decreased 16 percent or about half as much as during all-weather conditions.

These comparisons point to the accident reduction benefit that can result from converting stop-and-go signals to flashing signals during inclement weather. More consideration should be given to this type of operation where signal controls are flexible enough to permit the conversion.

Established Policy on Traffic Control Devices

The Manual on Uniform Traffic Control Devices for Streets and Highways [2] provides that the total vehicular volume entering an

[2] Published by the Public Roads Administration, Washington, 1948 (also 1954 revisions supplement).

Table 3.—Percentage change in the number of accidents and injuries per 100 million vehicles entering after installing traffic signals at 4-leg undivided intersections

Comparison	Average daily traffic	Type of signal installed	Percentage change after installing signal: Accident rate	Percentage change after installing signal: Injury rate
4 intersections with lowest ADT	15,800	Stop-and-go	81	52
4 intersections with highest ADT	16,600	Flashing beacon	−7	−66
All 7 intersections	20,000	Stop-and-go	53	23
All 15 intersections	8,200	Flashing beacon	−13	−45

Table 4.—Number of accidents and injuries per 100 million vehicles entering before and after traffic signal installations at 38 intersections, grouped according to below average and above average daily traffic volumes

Number of intersections	Average daily traffic	Accidents: Before signal installation	Accidents: After installation of— Stop-and-go signal	Accidents: After installation of— Flashing beacon	Accidents: After installation of— Percentage change	Injuries: Before signal installation	Injuries: After installation of— Stop-and-go signal	Injuries: After installation of— Flashing beacon	Injuries: After installation of— Percentage change
8	Below 20,200	172	227	------	32	151	95	------	−37
5	Above 20,200	131	129	------	−2	77	34	------	−56
17	Below 8,000	285	------	223	−22	203	------	126	−38
8	Above 8,000	125	------	112	−10	98	------	56	−43
13	20,200	153	182	------	19	117	67	------	−43
25	8,000	193	------	160	−17	143	------	87	−39

Table 5.—Number of accidents, by type of collision, before and after traffic signal installations at 89 intersections

Manner of collision	Accidents before signal installation	Accidents after installing **stop-and-go signal**	Percentage change	Accidents before signal installation	Accidents after installing **flashing beacon**	Percentage change
Rear-end	96	288	200	83	68	−18
Head-on	23	59	157	37	25	−32
Side-swipe	38	66	74	65	49	−25
Angle	259	128	−51	175	124	−29
Other	46	28	−39	42	33	[1] −21
All collisions	462	569	23	402	299	−26

[1] Percentage change is not statistically significant.

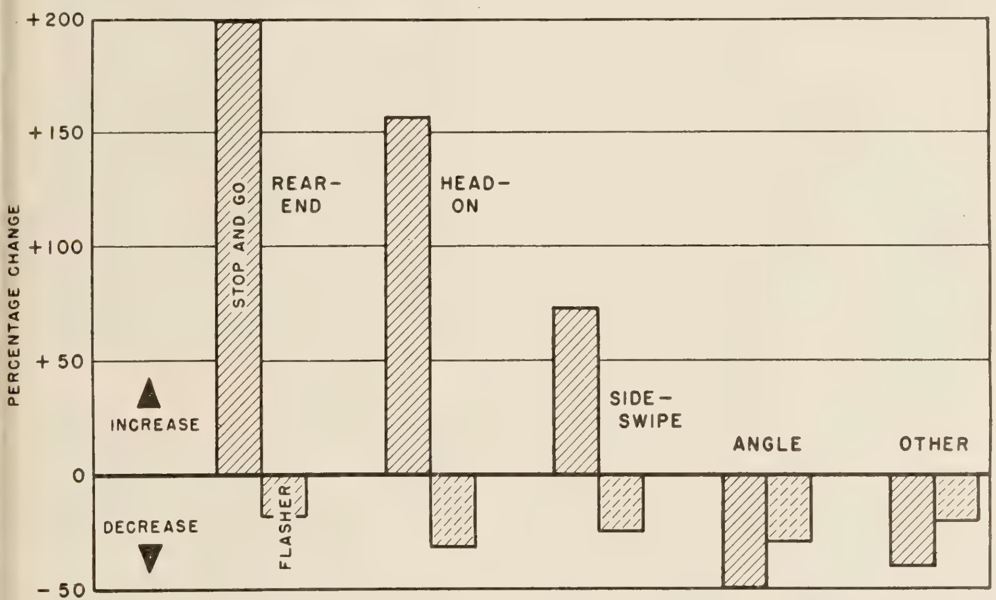

Figure 3.—Percentage change in the number of accidents, by type of collision, after installation of traffic signals.

intersection from all approaches must exceed 750 vehicles per hour for each of 8 hours of an average day in urban areas and 500 vehicles per hour for each of 8 hours of an average day in rural areas in order to justify stop-and-go signals. An equivalent 24-hour traffic volume to satisfy these conditions would probably be about 15,000 vehicles per day in urban areas and 10,000 vehicles per day in rural areas.

Table 4 shows that stop-and-go signals installed at these volumes have resulted in an increase in accident rates. Table 3 shows this even more strikingly for four-leg undivided highways. The 4 lowest volume stop-and-go intersections with an average daily traffic volume of 15,800 vehicles showed an increase of 81 percent in the accident rate after the signals were installed. The injury rate also increased. By way of contrast, both tables show that installation of flashing devices at these low volumes resulted in decreases in both accident and injury rates. It seems apparent, therefore, that consideration should be given to greater use of flashing beacons at intersections that barely qualify for stop-and-go signals according to provisions of the manual. The manual does state that stop-and-go signals should be operated as flashing red and flashing yellow signals for the side street and main street, respectively, if the traffic volume falls below 50 percent of the required minimums during two or more consecutive hours of the day. The present study provides additional support for this policy.

Throughout the article, the number of accidents, injuries, and fatalities have been expressed as a percentage relation of the experience before and after installing stop-and-go signals or flashing beacons. In order to test statistically the significance of the percentage changes that have occurred through modification of a highway intersection, two techniques have been developed by R. M. Michaels. His article titled *Two Simple Techniques for Determining the Significance of Accident-Reducing Measures* appears on pages 238–239 of this issue.

In testing for significance, the investigator seeks to determine whether a percentage change of a given magnitude is great enough to be attributed to the change introduced (e.g., installation of a traffic signal) or whether it is due to chance.

The two methods developed by Michaels, one labeled as a liberal test and the other as a conservative test, are based on the chi-square and the Poisson distributions. To make use of Michaels' methods of evaluation, the reader should refer to figure 1 on page 238. The percentage changes in the number of accidents, injuries, and fatalities reported in the tables of this article may be compared with the two curves shown in figure 1 of Michaels' article. In doing so, it will be observed that the relative changes reported in the tables for frequencies of accidents and injuries are, for the most part, statistically significant, based on a 5-percent probability level. In effect, this means that a percentage reduction in accidents of a given magnitude could not have occurred by chance more than once in 20 times. Percentage changes that are not considered statistically significant by either test are footnoted in the tables.

Recommendations

While more precise and complete information is needed to determine the conditions under which stop-and-go signals or flashing beacons may be most effective, the data presented here indicate that more widespread use of flashing beacons would be desirable at the simpler types of intersections.

A general and large-scale study of the relation between various design features of intersections and accidents should be undertaken to determine what type of traffic control device should be employed at a given intersection that would not only aid the movement of traffic but also provide maximum safety.

Flashing signal operation of certain stop-and-go signals during inclement weather seems to offer a possible method for reducing accidents. In determining when such signal operation should be used, the intersection accident and capacity factors should be fully considered.

Table 6.—Number of accidents, according to light and weather conditions, before and after traffic signal installations at 89 intersections

Light and weather conditions	Accidents before signal installation	Accidents after installing **stop-and-go signal**	Percentage change	Accidents before signal installation	Accidents after installing **flashing beacon**	Percentage change
Daytime:						
Clear	171	186	[1] 9	132	110	−17
Cloudy	59	81	37	58	43	−26
Rain, snow, or fog	39	88	126	45	36	[1] −20
All conditions [2]	269	355	32	235	189	−20
Nighttime:						
Clear	104	106	[1] 2	91	63	−31
Cloudy	51	55	[1] 8	34	15	−56
Rain, snow, or fog	34	48	41	32	27	[1] −16
All conditions [2]	189	209	11	157	105	−33

[1] Percentage change is not statistically significant.
[2] The number of accidents in this table are slightly less than those given in other tables because light and weather conditions were not known in every case.

Two Simple Techniques for Determining the Significance of Accident-Reducing Measures

BY THE DIVISION OF TRAFFIC OPERATIONS
BUREAU OF PUBLIC ROADS

Reported by RICHARD M. MICHAELS, Research Psychologist

WHENEVER an attempt is made to reduce accidents on a section of highway through modification of some of the accident-producing features, the question arises as to whether an observed reduction is due to anything more than chance alone. The most common approach to evaluation has been to institute improvements on sections of highways where a large number of accidents have occurred. Then after modification, reductions in accidents are reported as a percentage relationship of the before and after experience. At this point it must be determined whether the observed percentage reduction is great enough to be ascribed to the change introduced or whether it is due to chance factors. An attempt is made in this article to describe two simple relations that are available for determining the significance of a percentage reduction in accidents.

Accident Study Requirements

In order to make a comparison on a before-and-after basis, one must be certain that the two study periods are reasonably comparable. The following factors must be considered in the evaluation: (1) Some measure of vehicle-miles is needed for both the before and after periods in order to equate accident exposure; (2) the traffic volumes for each of the two periods should be approximately the same, for marked differences in volumes will affect the accident experience;[1] (3) the composition of the traffic on the study section should be unchanged during each of the two periods as this, too, may influence accident experience; and (4) since the fatal accident rate has consistently tended to decrease over the years, the accident totals in the after period should be corrected for any existing trends.

The accident figure used for analysis should be the total number of accidents by type, either fatality, injury, or property damage. The total number of fatalities or the total number of injuries should not be used whenever the number of accidents is less than 50.[2] The restriction in considering accidents by type reflects the fact that the number of deaths or injuries will usually be determined by factors peculiar to the individual accident. Thus, the number of occupants of the vehicle, its speed, or the geometry of collision may be the variables that determine the number of casualties. These are usually independent of the highway and any modification made thereon. Therefore, analyzing the frequency of accidents by type should yield a more precise measure of the effects of highway modifications.

Application of Poisson Distribution

Accidents are rare events in time, distance, or among individuals. For example, the probability of a fatal accident is only about 5 chances in 100 million miles of driving. The same low probability exists whether the reference is thousands of vehicles or frequencies of accidents among a group of people. An applicable statistical model is the Poisson distribution which is an approximation to the binomial density function when the probability of the event is low and the population in which it occurs is large.

In general, the assumptions of the Poisson distribution are met in most accident situations. Several studies have shown that this

[1] *The Interstate Highway Accident Study*, by Morton S. Raff. PUBLIC ROADS, vol. 27, No. 8, June 1953; also Highway Research Board Bulletin 74, 1955, pp. 18–45.

[2] Although the selection of a sample size of 50 is arbitrary, it should generally minimize any bias that might be imposed by the rare accident in which a large number of fatalities or injuries occurred. The probability of more than five deaths or injuries occurring in a single accident is very low. Consequently, with a sample size of 50 or more, these single cases will inflate the accident frequency rarely more than 10 percent.

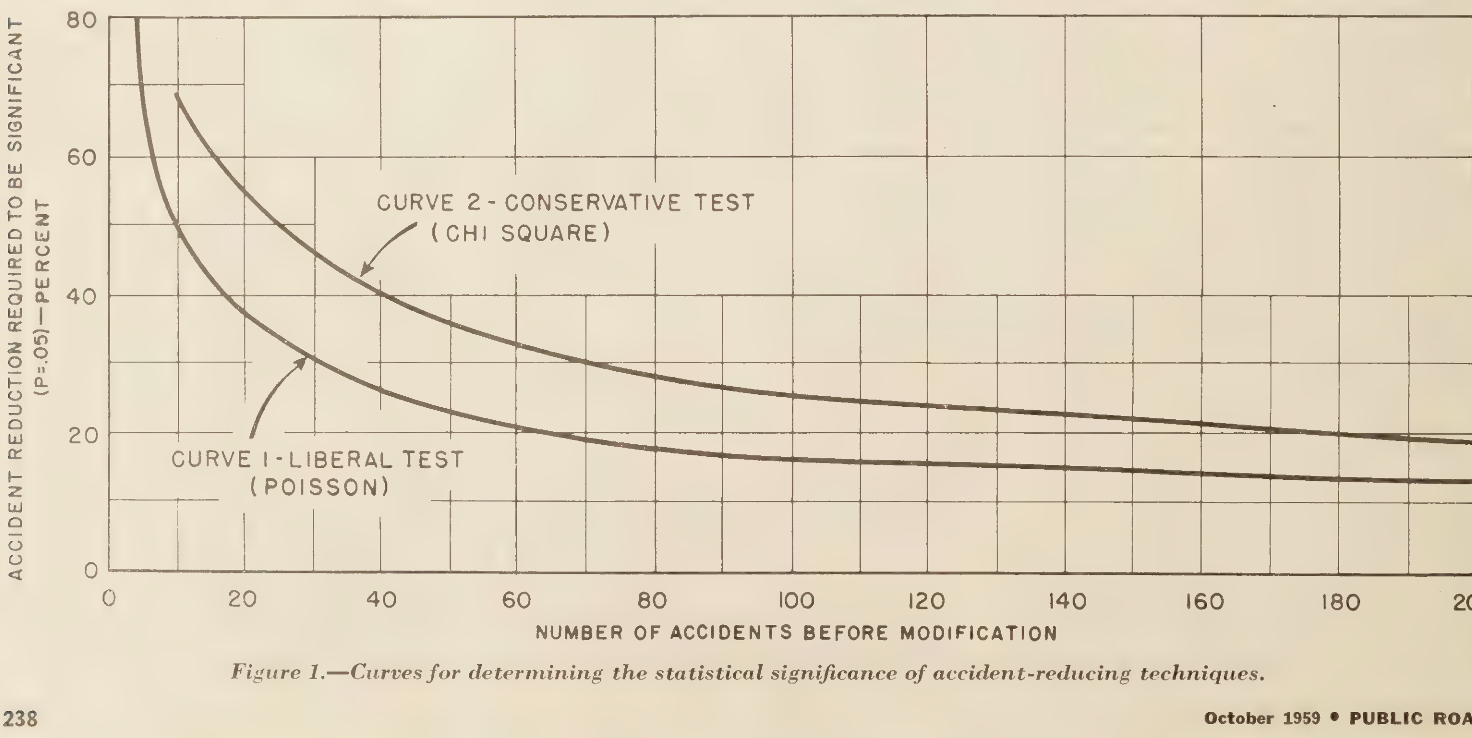

Figure 1.—Curves for determining the statistical significance of accident-reducing techniques.

model fits traffic accident data very well.[3] Therefore, one way to approach the problem of determining the significance of a percentage reduction in accidents is to assume that the observed data are a sample from a Poisson distribution.

If the before time period accident frequency is used as the expected value of the Poisson distribution, then it is possible to determine the percentage decrease in accidents that is statistically significant. Given the number of accidents before modification, the solution of the Poisson distribution leads to curve 1 which is illustrated in figure 1. The curve shows the percentage reduction in accidents that can be attributed to something more than chance. To illustrate the application of figure 1, assume that 40 accidents occurred on a given section of highway before modification. Reading from the graph (curve 1), it is observed that a 26-percent reduction in the number of accidents would indicate a significant change at a 5-percent probability level. Thus, a reduction in accidents due to chance would occur only once in 20.

Curve 1 is a liberal estimate of significance, and it must be interpreted carefully. The number of accidents in the before time period is assumed to be the true mean of the Poisson distribution. Since the number of accidents varies from year to year, taking any one year as the expected value of the population is arbitrary. Consequently this estimate of the mean may be in considerable error. It is especially suspect when the number of accidents in the before time period is abnormally high or low. Averaging over several comparable years is probably the best compromise. Within this limitation the test requires the smallest percentage reduction for obtaining statistical significance.

Application of Chi-Square Test

In order to make a more conservative estimate, an alternative is the chi-square test. By using this test it is possible to determine whether the two samples differ significantly; that is, whether the number of accidents occurring after corrective measures are instituted is reliably less than the number before. Curve 2 in figure 1 shows the percentage reduction in accidents that is significant as a function of the number of accidents occurring before improvements are made. The test assumes that the highway sections remain the same or can be adequately corrected from year to year.

The two curves represent two limits which can be applied to determine whether there is a reliable reduction in accidents. Curve 1 may be used to minimize the chance of calling a reduction not significant when in fact it is, and curve 2 may be used to minimize the chance of calling a reduction significant when it is not. In general, curve 2 should be used where there is limited before data. If accident data for several years before modification are available, and show a variation of no more than 20 percent from year to year, they may be averaged. Then the after time period may be compared with this average and curve 1 used to determine whether the reduction is statistically significant.

Testing for Significance

In order to demonstrate the use of the curves, four examples from two studies have been selected. In one study, the effect of traffic control devices on accidents at different types of intersections was investigated.[4] It was found that prior to installation of flashing beacons at five- and six-leg intersections, 19 accidents were recorded. After installation of flashing beacons, the number of accidents decreased 37 percent. Figure 1, curve 1, shows that for this size sample, it is reasonable to conclude that the flashing device did not significantly reduce accidents and the reduction could have been due to chance alone.

At four-leg intersections on divided highways, it was found that after installation of flashing beacons the number of accidents were reduced from 50 to 35, or 30 percent. Reference to figure 1 shows that for a before value of 50 accidents, a 30-percent reduction is significant according to the liberal test, but it is not according to the conservative test. Since only 1 year of before data is available, the conservative test is preferred. Therefore, it is reasonable to conclude that the flashing device did not significantly reduce accidents at these intersections.

One of the remaining two examples of accident experience, selected from a second study,[5] indicated a significant reduction in accidents according to the liberal test, whereas the other example met the requirements of both tests.

At an intersection of a four-lane divided highway and a two-lane highway, there was an average of 27 accidents over a 2-year period. Most of these accidents were due to left-turn movements from the major to the minor highway. In an attempt to reduce accidents, a left-turn lane was added to the major highway and a left-turn arrow was added to the traffic signal. After these modifications there were 16 accidents per year, or a 41-percent reduction. Figure 1 shows that this is a significant reduction using the liberal test but not using the conservative test. Since the data are averages for 2 comparable years, the liberal test is the logical one to use. On this basis it is reasonable to conclude that the traffic engineering measures significantly reduced accidents at this intersection.

At one point on a four-lane divided highway, drivers had to negotiate a sharp curve. Over a 3-year period there was an average of 17 accidents per year. In an attempt to reduce accidents, highway lighting was added and also road-edge delineators. After the changes, there were 5 accidents per year, or a 70-percent reduction. With reference to figure 1, such a reduction is statistically significant by both tests. Therefore, it is reasonable to conclude that the engineering changes did cause a statistically reliable reduction in accidents.

Statistical Limitations

A word of caution is in order concerning the use of the curves. Both tests require fulfillment of certain statistical assumptions to be wholly valid. There is no way to prove that these assumptions are met in any given field situation. Accident processes are not constant in time, and the factors influencing their change are poorly understood at best. Furthermore, there is never good control over all the variables influencing changes in accident incidence. Populations of drivers change, as do populations of vehicles. In addition, several accident reduction measures are always operating, e.g., enforcement and education. It is therefore rarely possible to eliminate the influence of all such forces and ascribe an observed reduction unequivocally to some single specific measure.

The curves shown may be used as an estimate of the percentage reduction in accidents that is required to achieve statistical significance. In using these curves the limitations inherent in any field study should be recognized. Where techniques or measures do appear to show a significant influence on accident incidence, a more intensive evaluation of them may be required. This can and should be done with emphasis given to careful control and more precise measurements.

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[3] *Poisson and Traffic*, by Daniel L. Gerlough and André Schuhl. Eno Foundation for Highway Traffic Control, Saugatuck, Conn., 1955, 75 pp.

[4] *Traffic Signals and Accidents in Michigan*, by David Solomon. See pp. 234–237 of this issue.

[5] *Reducing Accidents by Traffic Engineering.* Virginia Department of Highways, Richmond, 1957, 31 pp.

Estimated Travel by Motor Vehicles in the United States, 1957

BY THE DIVISION OF HIGHWAY PLANNING
BUREAU OF PUBLIC ROADS

Reported by ALEXANDER FRENCH
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THE average motor vehicle traveled 9,571 miles in 1957, almost one-half of it in cities, and averaged 12.47 miles per gallon of fuel. Total motor-vehicle travel in 1957 amounted to 647 billion vehicle-miles. For 1958 the total is estimated at 665 billion, and nearly 700 billion is forecast for 1959.

Of the 1957 travel, 40 percent was on main rural roads, which constitute 16 percent of the Nation's 3.4 million miles of roads and streets. Another 46 percent of the travel was on urban streets, which include only 11 percent of the total mileage. Local rural roads, comprising 73 percent of all mileage, carried 14 percent of the travel.

Passenger cars represented 83 percent of the vehicles and performed 82 percent of the travel in 1957. The average passenger car traveled 9,391 miles and consumed 652 gallons of fuel at a rate of 14.40 miles per gallon. Trucks and combinations accounted for 16 percent of the vehicles and 17 percent of the travel. The average truck or combination traveled 10,328 miles, but consumed twice as much fuel as the average passenger car, 1,302 gallons, at a rate of 7.93 miles per gallon. Buses accounted for the remaining 1 percent of the vehicles and 1 percent of the travel.

Table 1.—Estimate of motor-vehicle travel in the United States, by vehicle types, in the calendar year 1957

Vehicle type	Motor-vehicle travel: Main rural road travel	Motor-vehicle travel: Local rural road travel	Motor-vehicle travel: Total rural travel	Motor-vehicle travel: Urban travel	Motor-vehicle travel: Total travel	Number of vehicles registered	Average travel per vehicle	Motor-fuel consumption: Total	Motor-fuel consumption: Average per vehicle	Average travel per gallon of fuel consumed
	Million vehicle-miles	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle miles*	*Million vehicle-miles*	*Thousands*	*Miles*	*Million gallons*	*Gallons*	*Miles/gal.*
Passenger cars [1]	203, 542	71, 492	275, 034	254, 371	529, 405	56, 375	9, 391	36, 769	652	14. 40
Buses:										
Commercial	942	156	1, 098	1, 943	3, 041	85	35, 776	641	7, 541	4. 74
School and nonrevenue	552	551	1, 103	249	1, 352	179	7, 553	184	1, 028	7. 35
All buses	1, 494	707	2, 201	2, 192	4, 393	264	16, 640	825	3, 125	5. 32
All passenger vehicles	205, 036	72, 199	277, 235	256, 563	533, 798	56, 639	9, 425	37, 594	664	14. 20
Trucks and combinations	54, 960	18, 110	73, 070	40, 136	113, 206	10, 961	10, 328	14, 271	1, 302	7. 93
All motor vehicles	259, 996	90, 309	350, 305	296, 699	647, 004	67, 600	9, 571	51, 865	767	12. 47

[1] Includes taxicabs, motorcycles, and light trailer combinations pulled by passenger cars.

Travel Estimate Procedures

The travel data in table 1 are based on comprehensive studies made by the States in connection with the highway cost allocation study undertaken by the Bureau of Public Roads.[1] In that study, travel in municipalities of less than 5,000 population is included with rural travel, in accordance with the rural-urban classification of the Federal-aid systems. In table 1, however, all municipal travel is shown as urban. This is in accordance with the motor-vehicle travel data that have been published annually by the Bureau of Public Roads for a number of years.[2]

During 1957, more than 365,000 traffic volume counts of 16 hours' duration or longer were obtained either manually or with machines of various types. Permanently located counters are operated continuously at more than 1,500 locations throughout the United States. The continuous counts, when properly grouped and related to road sections having similar traffic characteristics, provided factors by which 24- or 48-hour counts were used to calculate the average traffic for the year.

Traffic counts in which the numbers of vehicles of each type are determined or classifications counts, as they are called, were obtained manually since no machine has yet been developed to obtain such information economically. To calculate reliable values for annual travel, observations were made during all seasons of the year, at all hours of the day, and on all days of the week. Approximately 28,700 classification counts were obtained during 1957, making it possible to compute the annual vehicle-miles of travel for each visual class of vehicle. These estimates were prepared by the States and supplied to the Bureau of Public Roads.

The new studies have made possible a broader, more reliable base for travel estimates. However, the total travel of 647,004 million vehicle-miles, calculated from the new estimate base, was very close to the total travel determined from the former trend procedures, being only 0.7 percent higher.

Significance of New Estimate Base

The new estimates indicate that the proportion of truck travel as well as the proportion of travel on local rural roads was previously overestimated. While the 1957 figure of 113,206 million vehicle-miles for all trucks and combinations is 2.5 percent below the 116,10[illegible] million vehicle-miles reported for 1956, the new trend data indicate that there was actually an increase of 1 to 2 percent. Similarly, 1957 estimates show that local rural roads carried 5 percent less traffic than was reported in previous years, and main rural roads and urban streets carried larger percentages; but from the new trend data it appears that increases in travel of 2 percent or more occurred on each of the three classes of roads and streets.

Values for fuel consumption by vehicle type have been recalculated on the basis of information developed for the *Highway Cost Allocation Study*, adjusted to 1957, and published registration data.[3] As a result, the average travel per gallon of fuel consumed for some vehicle types is slightly different from earlier 1957 published estimates. The new figures indicate a lower value of 7.35 miles per gallon for schoolbuses. The fact that this value is less than that for trucks and combinations is probably due to the stop-and-go driving characteristics of schoolbuses. The comparatively high mileage per gallon of fuel for trucks and combinations is due to the fact that panel and pickup trucks account for nearly half of all travel by trucks and combinations.

[1] *Third Progress Report of the Highway Cost Allocation Study*, H. Doc. 91, 86th Cong., 1st sess., 1959.

[2] See previous articles in PUBLIC ROADS magazine: the most recent article, for 1956, appears in vol. 29, No. 11, December 1957.

[3] *Highway Statistics 1957*, Bureau of Public Roads, table MV–1, p. 48.

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