GUIDELINES FOR THE INCLUSION OF LEFT-TURN LANES AT RURAL HIGHWAY INTERSECTIONS

S. L. Ring, and R. L. Carstens, Engineering Research Institute,
Iowa State University of Science and Technology

The design of rural at-grade intersections is often referred to as an art rather than a science. The specific decision of whether to provide a left-turn lane is an example of the unavailability of a rational and objective approach to a major problem. This research has reviewed the various techniques and procedures in use, has measured traffic characteristics at typical Iowa intersections, and has developed a rational approach as a guideline for inclusion of a left-turn lane. The procedure is based on relating the road-user benefits to the cost of providing the added turning lane.

•THE AT-GRADE rural highway intersection is the weakest link in the process of planning and designing a highway. Increased vehicle operating costs, driver irritation, accidents, and all of the variously occurring operational inefficiencies are manifestations of the inability to maintain uninterrupted traffic flow conditions. According to the National Safety Council about one-fourth of all rural accidents occur at intersections (1).

In the typical design of rural highway intersections in Iowa, satisfactory highway capacity is generally not a limiting parameter. Through-traffic lanes easily accommodate all traffic desires. Thus, the analytical techniques applicable to high-volume urban areas have little application to this situation. Consequently, a very troublesome facet of intersection design is whether to provide an auxiliary lane for left-turning vehicles. This decision frequently dictates the extensiveness of the intersection development and is a determining factor in future operational aspects of the intersection. Thus, the designer should be as objective as possible.

A literature search was conducted in which prior efforts in this field were reviewed and analyzed. A few studies were found to be particularly germane; other studies formed a useful reservoir of background knowledge. Applicable studies were reviewed in detail (2, 3, 4, 5, 6, 7, 8).

To develop a rational approach to decision-making regarding inclusion of an auxiliary left-turn lane requires that certain fundamental questions be answered. A knowledge of traffic-flow characteristics at local intersections is necessary. The purpose of this study is to evaluate local conditions and develop guidelines for the highway designer (9).

PRELIMINARY EVALUATION

One of the first phases of this study was determination of desirable field measurement information. Thus, an early decision was required to ensure an adequate depth of data at the evaluation phase. Because of the limited time available, further post-analysis field study was not a probable possibility; consequently, the initial decisions were evaluated as completely as possible.

Physical site conditions as criteria for left-turn lane inclusion were the initial considerations. Included in this subject were items such as sight distance, roadway foreslopes, shoulder conditions and dimensions, alignment, grades, adjacent land use effects, and vehicle turning path inadequacies. In many cases adverse physical conditions may be a relevant factor as a measure of inadequacies of existing facilities.

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Inadequacies represented by unsafe conditions should perhaps be a criterion by itself. In other words, there are 2 typical situations facing the highway planner: (a) an isolated intersection under consideration for a spot improvement program because of particular problems at the location, and (b) a highway improvement project that is of considerable length and that includes an intersection or intersections that must be evaluated regarding desired development. In the first case, the unsafe condition is very much a part of decision-making and is relative to establishing a project for improving inadequate physical elements. In the second case, existing substandard physical conditions are not relevant to the left-turn inclusion decision in view of the modern design standards that will automatically be utilized in the improvement project.

In either case it can be theorized that adverse physical site conditions will generate an improvement project. In most cases, however, the question of whether to include a separate left-turn lane is independent of the physical conditions. For example, there may be a special case where it is not feasible to develop desirable sight distance, and the situation may be alleviated by a left-turn storage lane. However, in the establishment of warrants for the inclusion of left-turn lanes at intersections, it was decided that substandard physical site conditions would not be included as a variable.

The factors to be considered and their relevancy were identified by asking the following questions: What are the adverse conditions at a rural intersection that can be expected to be alleviated with a separate turn lane? What factors measure these conditions? The answer to the first question includes delay to through vehicles stopped and waiting for a left-turner to select a gap and clear the through lane; delay to through vehicles decelerating from highway running speed and the subsequent acceleration to running speed; accident potential due to the left-turner decelerating, stopping, and standing in the through traffic lane; and reduction in the ability of the highway to accommodate the traffic demand within the service range desired.

Capacity is seldom of concern in the rural 2-lane highway situation under consideration. Consequently, it was determined that the investigation would be primarily concerned with vehicle delay and accidents as the 2 significant factors in establishing warrants for a left-turn lane. Measurement of these factors became the initial task.

INTERSECTION STUDY TECHNIQUE

A number of investigators have noted the problems associated with gathering and interpreting data regarding traffic performance at intersections (10, 11, 12, 13). The usual practice is to use 1 or 3 techniques, as follows:

1. A graphic recorder that has a moving paper on which as many as 20 pens denote the spatial arrangement of vehicles in relation to time by recording responses from input electrical signals provided by switches, pavement detectors, or signal controllers or by combinations of these;
2. Time-lapse photography in which all vehicular movements in the field of vision are recorded by a series of time-spaced 16-mm photos that can be later used to retrieve any particular characteristic of performance that can be visually identified; and
3. Observers with synchronized watches and stopwatches that record each vehicular event as it occurs during the study period.

In many cases a less detailed analysis of traffic performance may be required, and a simpler technique would be desirable. The authors observed the same problems that had been noted in the literature regarding the recording and retrieving of field data. A number of methods were tested in an attempt to hold the number of persons involved and the retrieval time to a minimum while maintaining reasonable tolerances. The technique finally adopted has not been previously employed in traffic operations measurement so far as the authors are able to determine.

In this method 2 unskilled observers using 2 inexpensive cassette tape recorders can obtain field data. One unskilled individual can reduce the data in a short time. Not only are equipment and labor costs reduced drastically, but also a more natural field study condition is maintained because of the unobtrusiveness of the observers.

The procedure is based on utilizing one tape recorder to play back a prerecorded signal of accurately spaced 1-sec clicks. This background time reference is played at the site from the first recorder, while the second tape recorder records the traffic events that are translated into audible form by 2 observers. The result is a tape that, when played back in the laboratory, yields a time band (which is referenced to real time) with interspaced coded sounds identifying specific traffic events. This procedure provides what might be termed an "audiograph," similar to a visual graph obtained from a 20-pen recorder. After the occurrences of the traffic events are related to a time, it is easy to determine gap and lag characteristics, headways, and delay time.

ANALYSIS AND EVALUATION

Distribution of Vehicle Headways

One objective of field data gathering was to determine whether significant error would be introduced by modeling traffic flow using some theoretical distribution. From the results of research reported previously by others, the actual distribution of headways in a 1-way traffic stream on a 2-way road often does not correlate closely with a distribution based on an assumption of random arrival of vehicles. If passing opportunities are unrestricted, vehicle arrival will be nearly random in accordance with a Poisson distribution, and headways will be distributed in accordance with a negative exponential expression. This situation would occur on a 4-lane road carrying moderate volumes of traffic. However, if opportunities for passing are restricted, as must be the case on a 2-lane 2-way road, platoons of vehicles are formed for which the speed is established by the lead vehicle. Thus, the number of closely spaced vehicles is greater, and the entire distribution of headways is different from the distribution were the arrivals entirely random.

For this investigation, field data on vehicle headways were gathered for 94 1-way traffic streams representative of peak-hour conditions. One-way rates of flow during peak 15-min periods varied between 148 and 732 vehicles per hour. These data were tested for conformity with a negative exponential distribution and with several Pearson Type III distributions. There was not significant agreement between the actual and the theoretical. The most substantial lack of agreement between observed data and a theoretical distribution for the frequency of occurrence of headways was with headways ranging from 1 to 3 sec. These actually occurred far more frequently than would be indicated by the theoretical distributions tested—the natural result of the formation of platoons of closely spaced vehicles. This lack of agreement is shown in Figure 1. The observed frequency of occurrence of headways of 4 sec or shorter is compared with that calculated by assuming a Poisson arrival process. Results using an Erlang distribution (a special case of the Pearson Type III distribution) are also shown. The research principals consequently concluded that further work based on an assumption of random arrivals would not be valid.

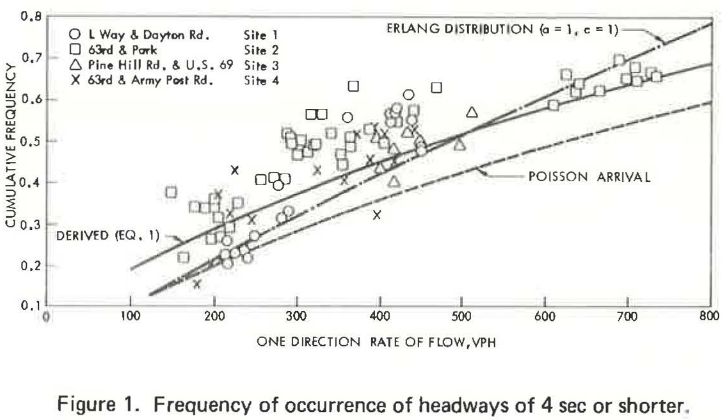

Figure 1. Frequency of occurrence of headways of 4 sec or shorter.

An alternative approach was to test the hypothesis that a satisfactory equation describing the frequency of occurrence of headways could be derived by multiple regression from the observed data. Such an equation, of course, would not be based on an assumption of randomness in the arrival process but would take into account the effect of platooning. It could then be used to calculate the probability of stops and the magnitude of delays caused by left-turning vehicles. The resulting equation is

$$y = 0.1279(t - 0.9)^{0.3681}q^{0.6094} \tag{1}$$

where

y = cumulative frequency of occurrence of headways equal to or shorter than t;
t = headway, sec; and
q = 1-way traffic volume, hundreds of vehicles per hour.

The coefficient of variation, R^2, for Eq. 1 is 0.79, indicating the appreciable amount of scatter that is common in samples of headway data.

An equation for the probability P of headway occurrence of any given length may be derived from Eq. 1.

$$p = (0.04708\ q^{0.6094})/[(t - 0.9)^{0.6319}] \tag{2}$$

Equation 2 is not unconventional in form in that P is a negative exponential function of the length of headway. While possessing some theoretical imperfections, Eqs. 1 and 2 satisfactorily reproduce the observed data for a wide range of traffic volumes and for the fairly narrow range of values of t that are pertinent for subsequent calculations.

A comparison of results using Eq. 1 with those calculated by assuming Poisson arrival illustrate the effect of platooning. This example is based on t = 4 sec and q = 4.48 or 448 vehicles per hour in 1 direction. Three samples from the observed data are also shown for comparison in the following:

Source	Value of y
Calculated from Eq. 1	0.484
Assuming Poisson arrival	0.392
Observed, site 1, 3-13-70	0.491
Observed, site 1, 4-28-70	0.473
Observed, site 2, 3-17-70	0.500

Lag and Gap Acceptance

A further objective for gathering field data was to determine lag and gap-acceptance characteristics for left-turning vehicles. With knowledge of these critical values and the probable vehicle headway distribution, we could estimate vehicle delay by appropriately considering the effect of queuing. Critical lags and gaps were determined from data gathered at each study site. These did not differ significantly from 1 site to another. However, sample sizes were quite small at sites 1, 2, and 4 so that values established for subsequent calculations were derived from a composite of the data gathered at all 4 study sites (Figs. 2 and 3). Results of this analysis are as follows:

Critical Lag or Gap	Sec
Lag	3.5
Gap for 1 car to complete left turn	5.5
Gap for 2 cars to complete left turn	7.3
Gap for 3 cars to complete left turn	9.5
Gap for 4 cars to complete left turn	11.6

Sample sizes for gap acceptance by more than 1 car were too small to be treated with a high degree of confidence. However, taken together they indicate rather clearly that vehicles are spaced at headways of about 2 sec when effecting left turns from a stop.

Theoretical Stops and Delays Versus Actual

With an expression for the spacing of vehicles in the traffic stream and knowledge of lag and gap-acceptance characteristics, one can calculate the probable number of vehicles that are forced to stop and the magnitude of delays to stopped vehicles. However, values calculated in this manner did not agree at all closely with those observed for the number of stops or vehicle delays. Significantly fewer vehicles stopped, and standing delay was markedly less than the theoretical values in nearly all samples. The research principals concluded that results from an approach based on this methodology could not be supported by the observed behavior of drivers at the test sites.

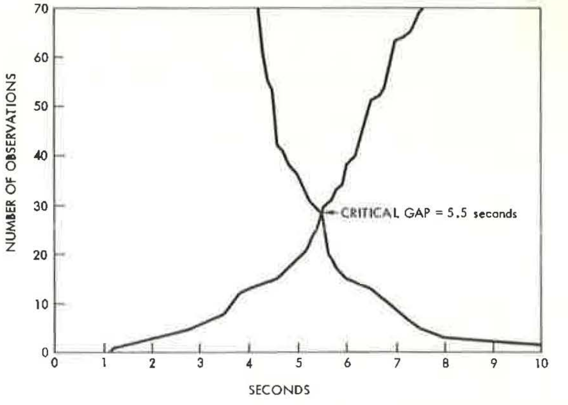

Figure 2. Consolidated totals for critical gaps.

There are several possible reasons why human behavior might not conform with the expectations of a theoretical model in the situation studied. Lag acceptance, for example, is likely to be a function of several characteristics of traffic streams that are difficult or impossible to measure. A driver might risk acceptance of a very short lag if he observes a long line of traffic behind the car that will conflict with his left turn. On the other hand, he might reject a longer lag if the oncoming car is the only one visible to him; he knows that he will be delayed only a few seconds by waiting for it to pass. If sight distances are adequate, a driver approaching an intersection at which he is to turn left will adjust his speed in any of several possible ways according to his evaluation of vehicle spacing in the approaching traffic stream. He may speed up slightly and lengthen a lag to a level acceptable to him and complete his turn without stopping. Or, he may decrease speed slightly to avoid an unacceptable lag. He may avoid the necessity of stopping by completing his left turn immediately after an oncoming car has cleared the intersection. The angle and location at which a driver crosses the opposing lane also may be varied so as to reduce delay and the necessity of stopping. If the approaching side-road lane is not occupied, drivers will frequently initiate left turns early by turning at a flat angle and clearing the opposing lane before the arrival of an oncoming car. They may also delay their turns (without stopping) and then cross the opposing lane with a turn of very short radius to permit an oncoming vehicle to clear. Driver behavior, while difficult to predict with certainty, generally will be directed toward minimizing the amount of delay and the necessity for stops.

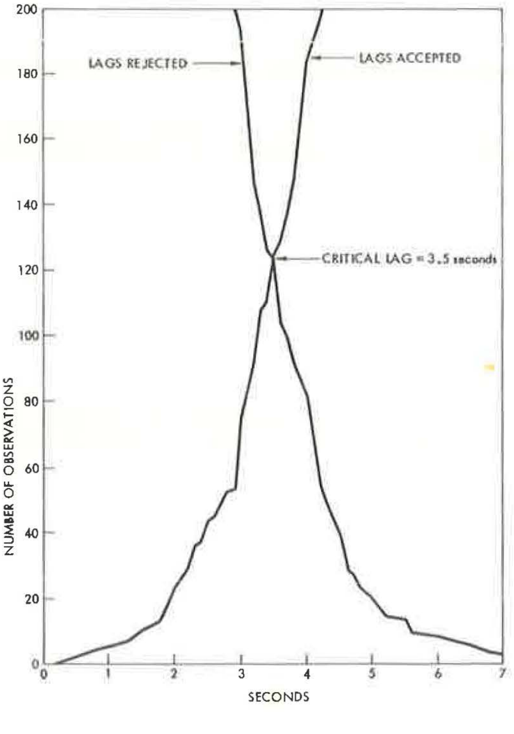

Figure 3. Consolidated totals for critical lags.

Highway Rese

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... was examined again to determine its predictive ability. Multiple regression techniques were used to derive the following equations:

$$D = 0.04393\ q + 0.04901\ a + 2.147\ L \tag{3}$$

$$S = 0.007764\ q - 0.003546\ a + 0.3071\ L \tag{4}$$

where

D = average standing delay for all advancing vehicles, sec;
S = proportion of advancing vehicles that stop;
q = 1-way volume of opposing traffic, hundreds of vehicles per hour;
a = 1-way volume of advancing traffic, hundreds of vehicles per hour; and
L = proportion of left turns in the advancing traffic stream.

R^2 values for these equations are 0.75 for D and 0.88 for S. The correlation matrix is interesting in that it indicates that D and S are much more strongly correlated with the proportion of left turns than they are with traffic volume in either traffic stream. The matrix is as follows:

Variable	q	a	L	D	S
q ~~9~~	1.00				
a	-0.27	1.00			
L	-0.65	0.51	1.00		
D	-0.08	0.41	0.59	1.00	
S	-0.26	0.36	0.71	0.83	1.00

The lack of significant correlation between the observed number of stopped vehicles and the opposing volume is surprising. This research was initiated with acceptance of an a priori assumption that there would be a direct and calculable relationship between stops and opposing volume. However, the observed number of stops differed markedly from values that could be expected in accordance with any theoretical distribution that the researchers could devise. Conclusions as to why this deviation could and did occur were reached only after a great deal of re-evaluation of the observed data and the techniques of analysis.

Conclusions as to why the number of stopped vehicles should be affected so strongly by the proportion of left-turning vehicles and so little by the opposing traffic are briefly summarized in the following.

1. Because of the occurrence of imperceptible or nearly imperceptible speed changes or adjustments in the location of the initiation of a turning maneuver, left-turning vehicles avoid a number of stops that are indicated as necessary by a theoretical relative positioning of opposing vehicles. Hence, use of theoretical spacings of opposing vehicles in combination with observed characteristics of lag and gap acceptance substantially overstates the necessity for stops and the magnitude of delays.

2. For purposes of this analysis, all stops were considered to be caused by left-turning vehicles forced to wait for opposing traffic to clear. Thus, it is logical to expect that the proportion of stops in the advancing traffic stream would bear a direct relationship to the proportion that turn left. This is true even though we may not be able to predict whether a given left-turning vehicle will be required to stop.

3. The total delay and the number of stops for all vehicles are calculated by multiplying Eqs. 3 and 4 by the advancing volume. Hence, these values are a function of a, and indirectly of q, where there is a reasonable directional balance in traffic flow. The advancing traffic stream constitutes from 30 to 70 percent of the 2-way traffic in the data from which Eqs. 3 and 4 were derived. We must assume that the effect of the op-

posing volume would be more significant if a greater imbalance existed and that Eqs. 3 and 4 would not then accurately predict stops and delays.

Research personnel concluded that Eqs. 3 and 4 would be more suitable than theoretical models for use in predicting stops and delays caused by left-turning vehicles at typical intersections in Iowa.

Equations 3 and 4 may be multiplied by the advancing volume to yield total hourly delay and total number of stops. However, q and a were assumed to be peak-hour volumes, so this calculation would be appropriate only for the peak hour. The average delay and proportion of stops will be somewhat less for all other daily periods. Appropriate factors for average hourly percentages of weekday traffic must be substituted for each of the 24 hours of the day in order to calculate daily totals. These factors were developed by the Iowa State Highway Commission from 54 automatic traffic recorder stations on rural primary highways in Iowa during the period from 1967 to 1969. The following equations in terms of daily traffic volumes result when these factors are used:

$$D_D = A_a(2.147\ L + 0.00002393\ A_q + 0.00002669\ A_a) \tag{5}$$

$$S_D = A_a(0.3071\ L + 0.000004228\ A_q - 0.000001931\ A_a) \tag{6}$$

where

D_D = daily standing delay for all advancing vehicles, sec;
S_D = number per day of advancing vehicles that stop;
A_a = 1-way volume of advancing traffic, vehicles per day;
A_q = 1-way volume of opposing traffic, vehicles per day; and
L = proportion of left turns in the advancing traffic stream.

The average delay per stopped vehicle is D_D/S_D.

Left-turning vehicles constitute a majority of those that stop and are delayed. However, some straight-through and right-turning vehicles are also delayed and required to stop behind vehicles waiting to execute a left turn. Construction of a left-turn lane will not change the number of left-turning vehicles that are required to stop and will have an insignificant effect on the amount of standing delay that they encounter. A left-turn lane will remove left-turn vehicles from the through lane so that straight-through and right-turning vehicles may proceed essentially without delay. Hence, a part of the benefit derived from construction of a left-turn lane is measured by a reduction in the number of stops and amount of delay accruing to through and right-turning vehicles as a result of left-turn maneuvers. Field data were analyzed to establish the proportion of stopped vehicles that proceeded straight through or turned right. This factor, K, is a quadratic function of L, as follows: $K = 0.6134\ L - 0.5744\ L^2$, $(0.0 < L \leq 0.8)$.

COST-BENEFIT COMPARISON

Reduction in Vehicle Operating Costs

Benefits to road users through reductions in operating cost and time were calculated for the following 2 typical situations that are representative of most rural intersections in Iowa:

Situation	Posted Speed Limit (mph)	Assumed Running Speed (mph)
1	70	55
2	55	45

The running speeds used are associated with a moderately congested level of service and are commonly used for analysis of operating conditions in Iowa.

Unit costs for passenger cars were assumed as follows:

Item	Amount
Value of time of vehicle occupants, $/hour	1.85
Operating cost for idling during standing delay, $/hour	0.11486
Operating cost for stop from 55 mph, $/stop	0.03143
Operating cost for stop from 45 mph, $/stop	0.01999
Excess time consumed per stop from 55 mph, hour	0.00584
Excess time consumed per stop from 45 mph, hour	0.00490

The value for time saved (14) and other values (15) are taken from other reports.

An equivalency factor is commonly used to account for the presence of trucks in the traffic stream. This is indicative of the average relationship of operating and time costs for commercial vehicles and for passenger cars. A factor of three to one is generally used by the Iowa State Highway Commission and has been used here. A quantity, T, is multiplied by the costs (or benefits) calculated for passenger cars to account for the increased costs (or benefits) occasioned by commercial vehicles. T varies depending on the percentage of trucks in the traffic stream.

Combining the costs for standing delay and stops, considering the effect of commercial vehicles, and converting to an annual basis result in the following equations:

$$b_{70} = K\ T\ A_a(5.160\ L + 0.00006991\ A_q - 0.00002443\ A_a) \quad (7)$$

$$b_{55} = K\ T\ A_a(3.685\ L + 0.00004961\ A_q - 0.00001516\ A_a) \quad (8)$$

where b_{70} and b_{55} are the annual reductions in operating and time costs for 70- and 55-mph posted speeds respectively. Pertinent costs are those associated with removing any necessity for stops by through or right-turning vehicles behind left-turning vehicles. The other variables were defined previously.

Most of the benefit calculated by Eqs. 7 and 8 is that occasioned by reducing the necessity for stops. The cost of standing delay (after stop) is typically less than 10 percent of the cost of stops from 55 mph and only about 13 percent of the running speed is 45 mph.

Accident Costs

A study was made of accident reports at the 4 rural intersections selected for field study. Records for the past 5 years were obtained but yielded a small number of accidents and a paucity of detailed information. The following information was desired:

1. The left-turn accidents that could be considered preventable (i. e., if a left-turn lane were available, the accident would not have occurred);
2. The estimated property damage per accident; and
3. The number and estimated cost of personal injuries.

Because of the extremely small sampling, no statistical significance can be associated with this information. However, generalized statements regarding the accident information are as follows:

1. About one preventable property-damage accident occurred per year per intersection;
2. About $350 of reported property damage was estimated at each property-damage accident; and
3. About one personal-injury accident occurred every 5 years at each intersection.

In order to interpret the generalized accident information, based on the minimal Iowa accident rate conditions investigated, we reviewed further supplementary information. A number of studies have been conducted that assign a dollar value for the cost of various types of accidents. The National Safety Council (16) in 1965 established the following schedule of accident costs:

Accident Type	Cost
Fatal	$34,400
Nonfatal injury	1,800
Property damage	310

Included are wage loss, medical expense, overhead cost of insurance, property damage, and the indirect costs of anticipated future earnings for a death.

A study by Smith and Tamburri (17) reviewed prior research in Massachusetts in 1953, in Utah in 1955, and in Illinois in 1959. From research primarily based on the Illinois study, they upgraded the accident costs to 1968 California conditions and arrived at the following schedule:

Accident Type	Cost
Fatal	$9,700
Nonfatal injury	2,500
Property damage	500

Only the direct costs of a fatal accident are considered in their schedule, and this explains the difference with the NSC schedule.

Based on the local accident study and the accident cost assignments noted, the following accident cost schedule is established for this study:

Accident Type	Cost
Property damage	$ 500
Nonfatal injury	2,500

As a result of the accident investigations at the 4 study sites, it was determined that the preventable accident rate norm would be set at 1 property-damage and $\frac{1}{5}$ personal-injury accident per year. This decision yields $500 + $\frac{1}{5}$ $2,500 = $1,000 per year as a normal anticipated accident cost reduction. In the preparation of relative warrant equations and graphs, a provision is made for adjusting the norm results to reflect local conditions. Because of the rare occurrence of fatal accidents, the consideration of this type would severely distort the small samples taken in this study.

Accident cost estimation will normally take 1 of 2 forms: (a) an investigation of the accident records at the intersection under review, and (b) an estimation of the difference between accident rates that could be anticipated by a comparison to similar situation records and then forecast.

In the establishment of the relative warrants, the preventable accident costs at the field study sites were utilized. That is, $1,000 per year is the annual accident cost saving. However, provision has been made for using any value in the equation, or values of $500 or $1,500 in the graphical solution.

Highway Costs

It has been established that reduction of vehicular delays and accidents, by the construction of a separate left-turn lane, reflects a benefit. It follows that decision-makers will evaluate the cost-effectiveness of a left-turn lane design alternative in relation to the benefits that may be anticipated. The benefit-cost ratio has been selected as the method of evaluation for establishing relative warrants for left-turn lane inclusions.

A standard design for a channelized intersection has been adopted by the Iowa State Highway Commission in recent years. This design widens the normal 2-lane pavement width 16 ft to provide for a separate left-turn storage lane. Painted pavement markings are used to effect the channelization. The California left-turn lane warrant statement previously discussed (6) includes the following statements: "If the state highway is zoned for speeds of 55 mph or greater, the use of painted channelization should be considered. If the zoned speeds are less than 55 mph, the use of a physically protected form of channelization is suggested."

The estimate of construction cost used in this study is based on the difference between a normal 2-lane pavement in one case and a standard channelized intersection in the alternate case. The design is the standard Iowa State Highway Commission type shown in Figure 4. Unit prices reflecting current prices were obtained from commission contracts and right-of-way and maintenance departments. A summary of the construction and maintenance cost items is as follows:

Item	Additionally Incurred Costs of Channelized Intersection
Initial construction costs	
Pavement	$13,870
Earthwork	1,165
Drainage	336
Right-of-way	125
Lighting	9,000
Total	24,496
Annual maintenance costs	
Median and arrow painting	250
Rumble strip	250
Snow removal and salting	50
Lighting maintenance and energy	60
Total	$ 610

A very significant part of the capital costs is the initial investment is lighting. This design includes six 400-watt mercury-vapor luminaries.

The average annual project costs are estimated as the sum of the capital costs on an annual basis, plus the annual maintenance costs. The differential annual cost of a channelized intersection is calculated by the following equation:

$$\Delta C = (C_1K_1 + C_2K_2 + \ldots + C_nK_n) + \Delta M \qquad (9)$$

where

ΔC = average annual cost difference incurred by the construction of a channelized intersection;
C_n = capital costs of individual construction items;
K_n = capital recovery factors for a specific interest rate and service life; and
ΔM = average additional annual maintenance costs incurred because of construction of a channelized intersection.

For the purposes of this study it was assumed that the service life and the interest rate were the same for each item of construction. The interest rate selected was 6 percent, which is currently used in the commission's planning division studies. The service life was selected as 20 years for every construction element. Consequently, the calculations of normal annual construction costs are ΔC = 24,496(0.087185) + \$610 = \$2,746.

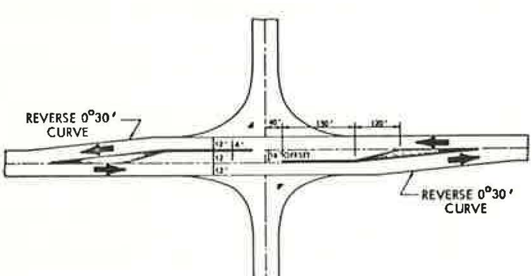

Figure 4. Typical primary road intersection.

Benefit-Cost Ratio

The benefit-cost ratio utilizes the savings from reduced stops and delays to through and right-turn vehicles (Eqs. 7 and 8) and from the elimination of preventable left-turn involvement accidents, C_a, as the benefit and the average annual project costs, AC, as the cost.

$$B/C_{70} = [K\ T\ A_a(5.160\ L + 0.00006991\ A_q - 0.00002443\ A_a) + C_a]/2{,}746 \quad (10)$$

$$B/C_{55} = [K\ T\ A_a(3.685\ L + 0.00004961\ A_q - 0.00001516\ A_a) + C_a]/2{,}746 \quad (11)$$

where B/C_{70} and B/C_{55} are the benefit-cost ratios for an area with 70- and 55-mph posted speeds respectively.

If the annual benefits exceed the annual costs (i. e., if B/C is greater than one), construction of a left-turn lane is warranted. Obviously other factors, such as safety or maintaining functional classification integrity, may in fact be dominant.

Equations 10 and 11 provide the highway engineer with a rational approach to decision-making regarding the added expenditure for a separate left-turn lane design.

SUMMARY AND CONCLUSIONS

An analysis of field data gathered under this project indicates that the use of theoretical distribution to describe vehicle headways is not applicable to rural 2-lane highways in Iowa. Distributions based on random arrivals do not correlate closely with actual 1-way traffic-stream data. An alternate approach was tested that uses multiple regression analysis of field data to describe the frequency of headways. Then, with a knowledge of lag and gap-acceptance characteristics, one can calculate the theoretical magnitude of stops and delays. However, values determined in this manner do not correlate at all well with observed data.

As an alternate approach, the mass of field data gathered were examined by using multiple regression techniques to yield equations for predicting stops and delays. Benefits accruing to road users by reducing stops and delays to through and right-turning vehicles were added to a potential reduction in accident costs. When they are compared to the added cost incurred by a left-turn lane construction project, a method of evaluating the cost effectiveness of the construction results.

The benefit-cost ratio technique is thus recommended as the criterion for decision-making. If the benefit-cost ratio is more than one, the construction is warranted. If less than one, the construction is not warranted (based on these factors alone).

ACKNOWLEDGMENT

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APPENDIX

WARRANTS FOR LEFT-TURN LANES AT 2-LANE RURAL INTERSECTIONS

For determining the benefit-cost ratio in a specific application, the following techniques are presented.

1. Benefit-cost ratio mathematical equations, Eqs. 10 and 11, may be solved.
2. Mathematical formulas, reduced to nomograph form (Figs. 5 and 6) may be used for repetitive applications. Three values for A_c are incorporated into the nomograph and represent a range on each side of the $1,000 norm value. The value of $2,746 for annual cost is incorporated in the nomograph, but any variation in this value may be used by multiplying the results by the ratio of $2,746 to the new value.

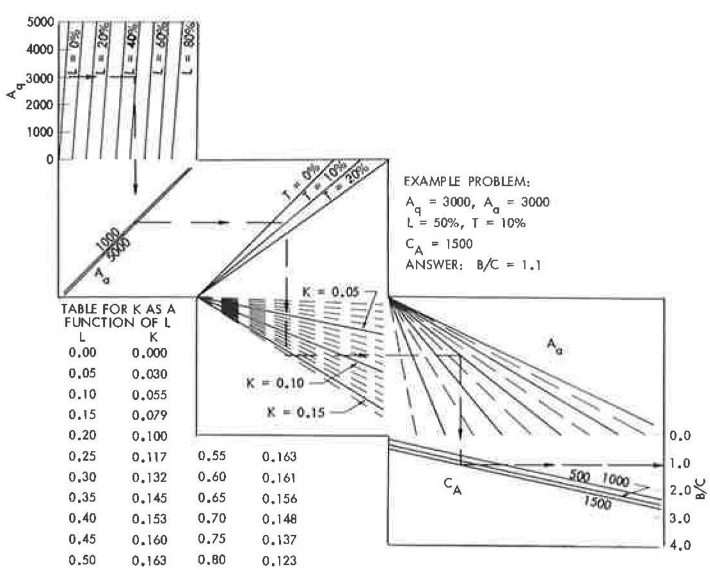

TABLE FOR K AS A FUNCTION OF L

L	K	L	K
0.00	0.000		
0.05	0.030		
0.10	0.055		
0.15	0.079		
0.20	0.100		
0.25	0.117	0.55	0.163
0.30	0.132	0.60	0.161
0.35	0.145	0.65	0.156
0.40	0.153	0.70	0.148
0.45	0.160	0.75	0.137
0.50	0.163	0.80	0.123

Figure 5. Nomograph for calculating benefit-cost ratio for left-turn lane—posted speed = 70 mph.

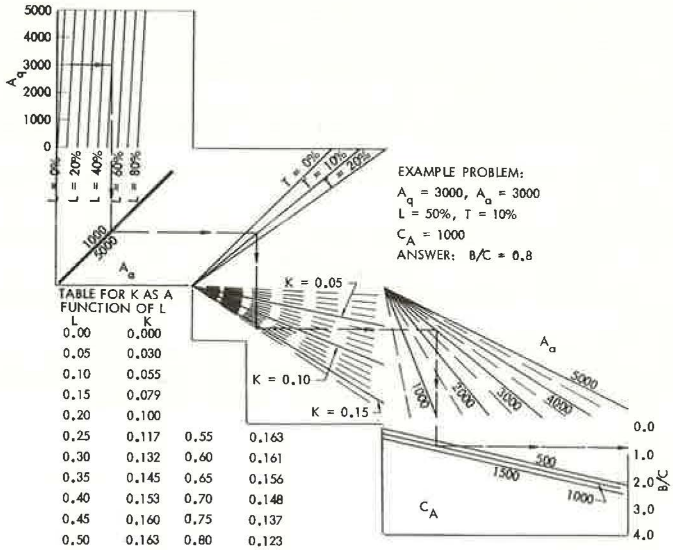

Figure 6. Nomograph for calculating benefit-cost ratio for left-turn lane—posted speed = 55 mph.

3. A series of simplified charts (Figs. 7, 8, 9, 10, 11, and 12) for various posted speeds and accident cost savings, A_c, may be used for cases of equally distributed opposing and advancing traffic volumes, $A_q = A_a$. Shown on these charts are curves connecting the points where B/C = 1. Thus, the range above the appropriate truck percentage line warrants a left-turn lane whereas the range below does not warrant the construction.

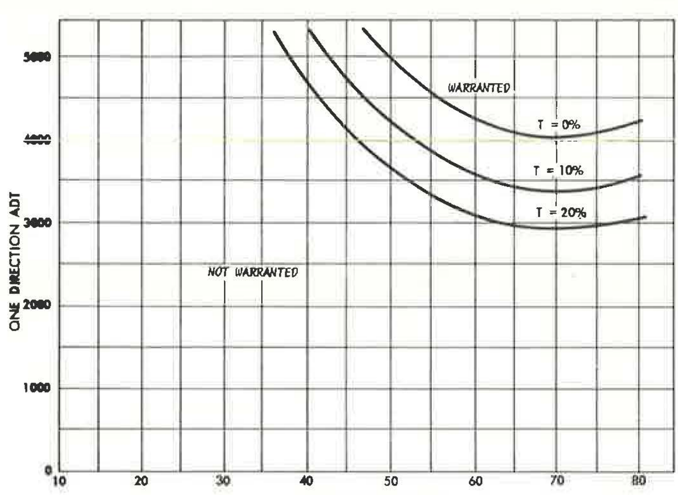

Figure 7. Warrant for left-turn lane—posted speed = 70 mph and annual accident cost reduction = $500.

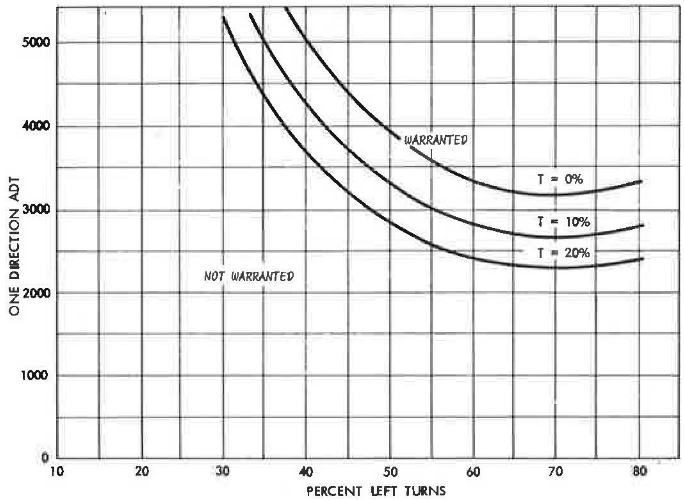

Figure 8. Warrant for left-turn lane—posted speed = 70 mph and annual accident cost reduction = $1,000.

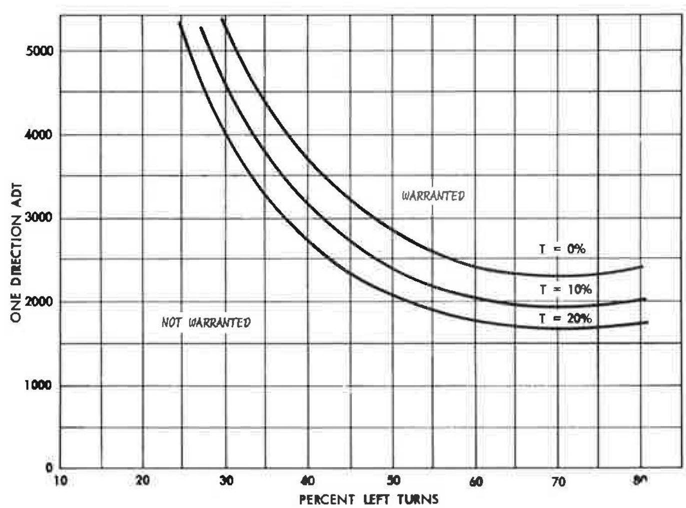

Figure 9. Warrant for left-turn lane—posted speed = 70 mph and annual accident cost reduction = $1,500.

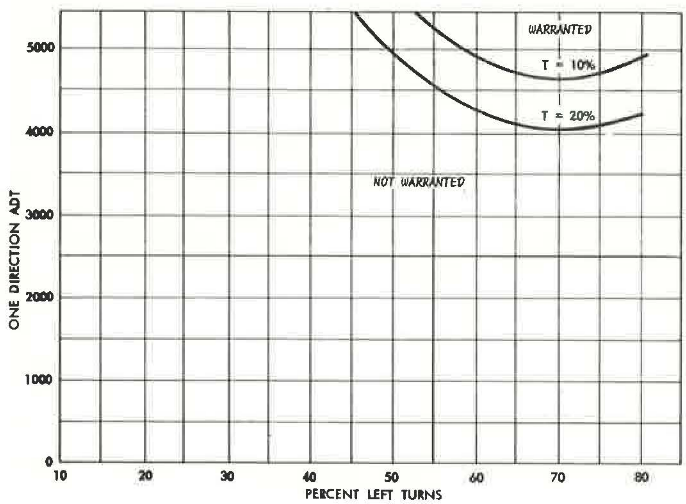

Figure 10. Warrant for left-turn lane—posted speed = 55 mph and annual accident cost reduction = $500.

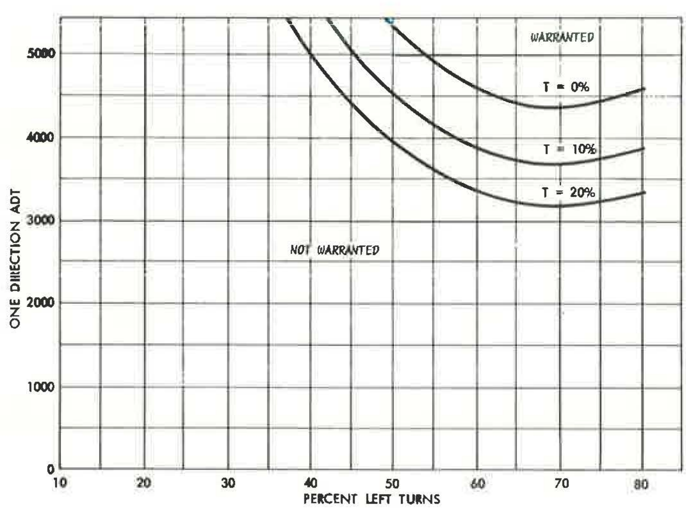

Figure 11. Warrant for left-turn lane—posted speed = 55 mph and annual accident cost reduction = $1,000.

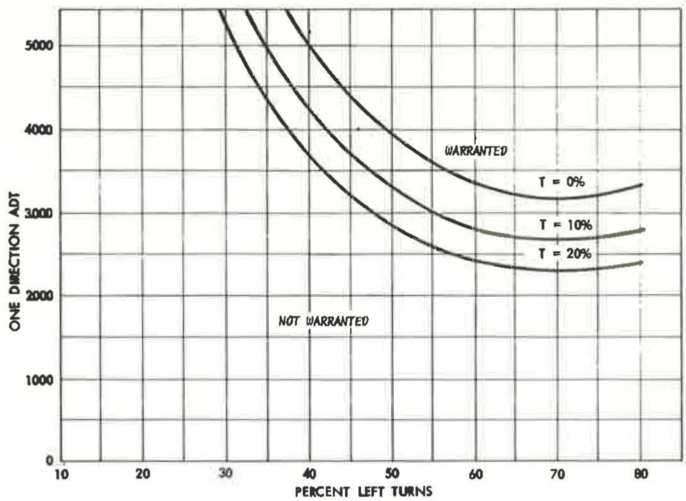

Figure 12. Warrant for left-turn lane—posted speed = 55 mph annual accident cost reduction = $1,500.