S. L. RING
R. L. CARSTENS
SEPTEMBER 1970

Final Report
ISU – ERI – AMES – 82100

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

IOWA HIGHWAY RESEARCH BOARD PROJECT HR-147
CONDUCTED BY THE
ENGINEERING RESEARCH INSTITUTE
FOR THE
IOWA STATE HIGHWAY COMMISSION

Project 815-S

ENGINEERING RESEARCH INSTITUTE
IOWA STATE UNIVERSITY
AMES, IOWA 50010 USA

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16. Abstract

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 lack of an available rational 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 benefits to the road user to the cost of providing the added turning lane.

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INTRODUCTION

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).

The planning of an intersection development, especially during the phase of initial decisions regarding level of design, is best categorized as an "art" rather than a "science." This was the finding (in part) of an early report of the Highway Research Board Subcommittee on Warrants for Channelization of Grade Intersections (2). After an investigation of the various states' practices, the concensus of opinion was that each intersection was approached as an isolated design problem with its own peculiar characteristics. Interestingly, the two most important publications on channelization, the 1952 Highway Research Board Special Report #5 (3) and its later replacement in 1962 with Special Report #74 (4), both state that channelization design is based to a large degree upon the judgment and experience of the designer.

The highway engineer has available a large amount of general statistical data on vehicle operating characteristics, knowledge of individual driver behavior, laws and formulas relating to the physical sciences, and a reservoir of empirical and theoretical tools for decision making. However, the decision to provide additional traffic lanes, auxiliary turning lanes, channelizing

elements, etc., at a particular intersection is usually not made on the basis of a rational approach to the problem. More frequently it is the result of complaints from the travelling public, a personal arbitrary decision, or merely the application of generalized standards.

In the typical Iowa rural highway intersection design situation, satisfactory highway capacity is generally not a limiting parameter. The ability of the through traffic lanes to accommodate all of the traffic desires is not overtaxed. 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 the decision whether or not to provide an auxiliary lane for left-turning vehicles. This decision is important in that frequently it dictates the extensiveness of the intersection development, as well as being a determining factor in the future operational aspects of the intersection. Thus, it behooves the designer to be as objective as possible.

Drivers of vehicles approaching an intersection and desiring to turn left are faced with a number of decisions and are a part of a system of interaction between vehicles. When traffic volumes are low, the decisions and interactions are minimal and have little effect on operations. As traffic volumes increase, the presence of a left-turning vehicle creates more of a problem. The driver of the left-turning vehicle must observe the oncoming traffic flow and make a decision on whether to accept a gap in the traffic stream. All the while the left turn driver must be observing the rear view mirror for advancing traffic following his vehicle. The vehicles following must be prepared to interact with the left turner, slowing, stopping, and waiting, as the case may be. Should a gap of suitable length occur, the left turner will negotiate the left-turn movement in safety, and with little

personal delay or influence on other vehicles.

In the case of increased traffic volumes, fewer opportunities for gaps in the oncoming traffic flow will occur. The left-turning vehicle is required to stop and wait for an adequate gap length. On a two-lane highway, the delayed left-turn vehicles and the delayed through and right turning vehicles may form a queue. Significant delays may occur to the extent that emotions are strained and conditions leading to potential accidents will develop.

When delay becomes objectionable to the left turner he may elect to turn in a gap length that he would normally reject. Oncoming traffic may be required to "hit their brakes," creating an effect on the following drivers who do not anticipate the sudden reaction of the lead vehicle. Obviously, a direct relationship exists between through traffic volume, left turning volume, delay, and accident potential.

In order to develop a rational approach to decision making regarding the inclusion of an auxiliary left-turn lane, certain fundamental questions must be answered. A knowledge of traffic flow characteristics at local intersections is necessary. That is: What are the vehicle spacings and the time headways between successive vehicles? What is the size of a gap in an oncoming traffic stream that will be accepted by drivers desiring to turn left? What is the relationship between rates of traffic flow, and probability of a gap of a certain size occurring? etc. It is the purpose of this study to evaluate local conditions and to develop "Guidelines for the Inclusion of Left-Turn Lanes."

LITERATURE REVIEW

This study was undertaken not only to provide the end result "Guidelines," but to review and analyze the prior efforts in this field as presented in published literature. A few studies are particularly germane, and will be reviewed in detail. Other studies form a useful reservoir of background knowledge, but are not particularly applicable to this problem.

In a 1947 article in Traffic Engineering (5), Paul S. Robinette stated, "I known of no precise yardstick by which any traffic situation can be measured that will produce an exact answer as to just when left turn controls are warranted. If there is such, I would appreciate gaining this information." The same statement is being made today by many highway engineers as they grope for rational consistent decisions. Mr. Robinette went on to present a "rule of thumb" approach to decision making that he employed, which was based on capacity and safety. His concern and background were in urban areas rather than rural, however, and as he stated, these empirical formulae had not been fully developed or proven.

Mr. Ronald W. Failmezger of the Oregon State Highway Commission presented the results of a study in the April 1963 issue of Traffic Engineering (6). An Index of Hazard (IH) was developed, which was based on the relative difficulty in making a left turn due to traffic volumes, and on the physical features of the site. The basic model used was:

$$IH = V_L V_O (1 + F_C + F_E + F_{SA} + F_{SO} + F_S + F_M)$$

where

IH = Index of Hazard

V_L = The 8 hour maximum volume of left turning vehicles

V_O = The 8 hour through movement volume in opposition to the left turn volume used

F_C = The clearance width factor for a vehicle turning left

F_E = The escape width factor which is a measure of the useable shoulder width for an overtaking vehicle to pass on the right

F_{SA} = Sight distance ahead factor

F_{SO} = Sight distance overtaking factor

F_S = Vehicle speed factor

F_M = Miscellaneous factor.

In this case the primary factors representing the difficulty for left turning vehicles to select a safe gap is approximated by the product of the left turn volume and the opposing volume of traffic. The physical condition of the site serves as modifier. However, Mr. Failmezger reasoned that construction cost and accident records should be an element in establishing warrants. Consequently, he developed a Relative Warrant (RW) Formula of the form:

$$RW = \frac{IH}{C_T}\left(\frac{10 + Ap}{8}\right)$$

where

C_T = The total estimated project cost

Ap = The number of preventable accidents for a 5-year study period

and

8 = A constant to develop near unity results.

However, the Oregon State Highway Department decided that the cost of construction should not govern the decision regarding the needs for the

left-turn lane. Results were a modification of the formula to:

$$RW = \frac{IH}{124,000} (10 + Ap) .$$

This technique is used by Oregon as a guide, with each case being individually evaluated.

Oregon State made another earlier contribution in the form of a study reported by R. H. Baldock in August 1964 (7). It was their belief that the infrequent left-turn movement caused a disproportionate high rate of accidents. Thus a high accident potential may exist due to the surprise nature of the infrequent left turner. The results of the study were presented in tabular and graphic form, and these data show that the infrequent left turns, when the through volumes and speeds are consistent, cause accidents in a greater frequency than the higher rate of left-turning vehicles locations (Fig. 1).

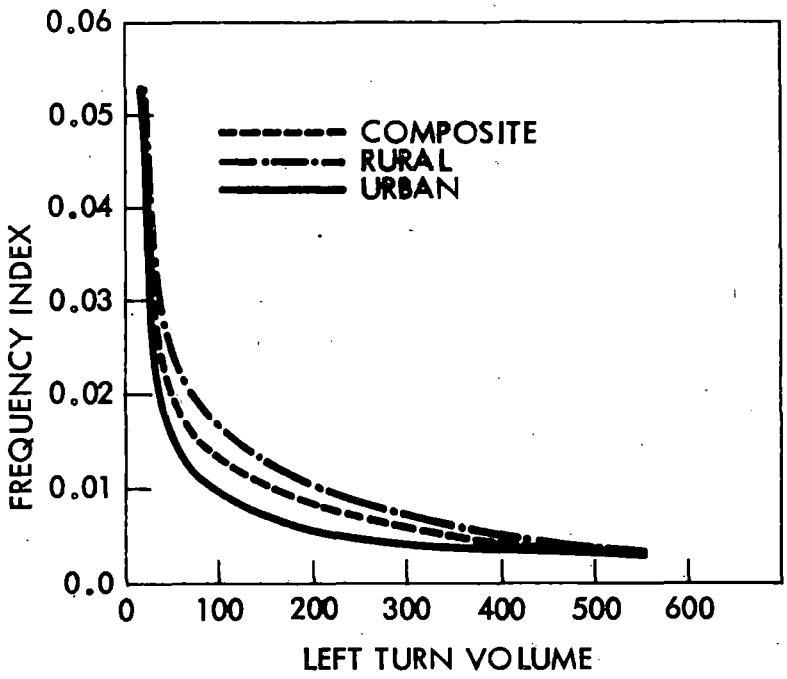


Fig. 1. Frequency index of accidents per year caused by one left turn per day by left turn volume, Nov. 1945. Oregon State Highway Department, Traffic Engineering Division (7).

Olin K. Dart Jr. in a doctoral dissertation at Texas A & M University in 1966, (8) presented a study entitled, "Development of Factual Warrants for Left Turn Channelization Through Digital Computer Simulation." The study was based on a computer simulation for determining vehicle delay at urban multilane signalized intersections, and consequently is not applicable to the rural situation. However, it is interesting to note that the final recommended warrants were based on vehicle delay, i.e.: "The intersection that delays less than 5 percent of the vehicles operating through it for more than one cycle length could be considered satisfactory."

Mr. Dart's simulation model was based on the following intersectional operational characteristics: time distribution of vehicles, vehicle approach speeds, lane change desires and gap requirements, acceleration and deceleration characteristics, car following characteristics, left turn gap requirements, starting delay, etc. Field studies utilized a 16 mm time lapse photography technique. The result of the study was a multiple regression equation with the dependent variable being the number of vehicles delayed more than one signal length and a number of independent variables. In order to provide a more convenient tool for the designer, the equation was translated into a set of graphical solution probability curves.

In January 1967 Mr. M. D. Harmelink of the Department of Highways, Ontario, Canada authored a report most applicable to the subject (9), (10). His study was entitled, "Volume Warrants for Left-Turn Lanes at Unsignalized Grade Intersections." He states that "the study was undertaken because of the lack of consistent volume warrants for left-turn storage lanes at unsignalized intersections." The basis for his warrants were established as traffic and operational characteristics, rather than benefit-cost analysis.

Under the direction of Mr. Harmelink a number of field studies were conducted to acquire vehicle operating data by a crew of five persons using synchronized watches. Primarily, the field data desired were: the critical gap in an opposing traffic stream for a left turning vehicle to make its maneuver, the time required to complete a left turn maneuver, and the time vehicles were delayed due to a left turning maneuver. It was assumed that the arrival and service headways followed a Poisson distribution.

Using predetermined probability of occurrence of an arrival behind a left turning vehicle, Mr. Harmelink developed a set of curves. With the volume of traffic, the highway speed, and the percent of left turns, the designer selected a particular figure to determine whether a left-turn lane is warranted.

In the report the author notes other factors that influence a decision, but which are not a part of his warrants. Also he recognizes that the basic assumption of exponential arrival and service headways was not tested, and that an analytical model could be considerably refined. This in fact is a very relevant observation if a theoretical distribution is used.

The 1965 Highway Capacity Manual (11) offers little regarding the ability of a two-lane rural highway unsignalized intersection to accommodate left turn demands. The general approach is to assume that if capacity is of concern, volumes will be of such magnitude that traffic signals will have been installed, and the procedures for signalized intersection conditions are then utilized. More appropriately, capacity is related to level of service, which is related to probability of delay, which becomes a key factor in the evaluation.

In 1968 Shaw and Michael presented the results of a study conducted in Indiana (12). The authors developed regression equations to be used as predictors of delay time and accident rates due to the absence of a left-turn

lane. Benefits accrue when a left-turn lane construction project reduces delay to through vehicles and in the number of accidents attributable to the left turner. Thus, a left-turn lane is warranted when the costs of construction are equal to or less than the economic benefits derived. The field study to determine delay to through vehicles employed a graphic recorder.

California has conducted a series of studies relative to the effects of minor highway improvement projects. The (part 5) report entitled, "Evaluation of Minor Improvements — Left Turn Channelization" was printed in 1968 (13). The purpose of the study was to evaluate the effectiveness of left turn channelization installations before and after accident studies. The report recommends the following warrants:

> "The use of left-turn channelization as a traffic control device should be considered at unsignalized intersections having a total of four or more left-turn plus rear-end accidents in a 12 month period (involving vehicles from intersection legs to be channelized), or 6 or more left-turn plus rear-end accidents in a 24 month period."

An example of a State Highway Commission policy for inclusion of a left-turn lane is in the Minnesota Road Design Manual (14). Section 5-291.331 states:

> "The roadway shall be widened to provide an extra lane for passing vehicles stopped for a left turn or making a right turn if:
>
> a. The access is to a public road, an industrial tract, or a commercial center, and;
>
> b. Accident records confirm an excessive hazard (more than 5 accidents per year involving turning vehicles), or;
>
> c. The projected two-way DHV exceeds 700 VPH."

PRELIMINARY EVALUATION

The investigators perusal of the published material which documents the state-of-the-art in left-turn lane analysis, was primarily oriented toward:

1. Identifying the variables that had been used in establishing the various warrants.
2. The techniques of data gathering.
3. The form that was used to present the warrants. (i.e., mathematical model, nomograph, statement, etc.)

A number of the studies reviewed are summarized in Table 1 to illustrate the variations that exist. Not only are a variety of elements considered, but the manner in which the warrants are presented for application takes various forms.

One of the first phases of this study was to determine what information would be desired from field measurements. Thus, an early decision was required to insure an adequate depth of data at the evaluation phase. Due to the limited time available, further field study after an analysis probably would not be possible, consequently the initial decisions were evaluated as completely as possible.

The relationship of physical site conditions as an element of a criterion for left-turn lane inclusion was the initial consideration. Included in this subject would be such items as: sight distance, the roadway foreslopes, shoulder conditions and dimensions, alinement, grades, adjacent land use effects, vehicle turning path inadequacies, etc. In many cases adverse physical conditions may be a very relevant factor as a measure of inadequacies of existing facilities.

Table 1. A summary of elements from selected left-turn lane warrants.

Study	Elements considered						The form of the warrant
	Physical site conditions	Vehicle speed	Traffic volumes & characteristics	Vehicle delay	Accidents	Capital costs	
Failmezger (Oregon) (6)	√	√	√		√		Mathematical equation to solve.
Harmelink (Canada) (9)		√	√	√			A series of curves.
Hammer (Calif.) (13)		√			√	√	A policy statement.
Shaw (Indiana) (12)	√	√	√	√	√	√	Mathematical equation to solve
Dart (Texas) (8)		√	√	√			A series of curves.
Minnesota Design Standard (14)			√		√		A policy statement.

However, it was noted that inadequacies as represented by unsafe conditions should perhaps be a criterion by itself. In other words, there are two typical situations facing the highway planner: (1) An isolated intersection which is under consideration for a spot improvement program and is considered because of particular problems at the location; and (2) the highway improvement project of considerable length within which is included an intersection (or a number of intersections) which must be evaluated as to the degree of development desired. In the first case the unsafe condition is very much a part of decision making, 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. But, in most cases the question of whether or not to include a separate left-turn lane is independent of the physical conditions. There may be a special case wherein it is not feasible to develop desirable sight distance, for example, 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.

In an effort to identify the factors to be considered and their relevancy, the questions were next asked: (1) What are the adverse conditions at a rural intersection that can be expected to be alleviated with a separate turn lane? and (2) What factors measure these conditions?

The answer to the first question includes the following:

a. Delay to through vehicles stopped and waiting for a left-turner to select a gap and clear the through lane.

b. Delay to through vehicles decelerating from the highway running speed, and the subsequent acceleration back up to running speed.

c. Accident potential due to the left-turner decelerating, stopping, and standing in the through traffic lane.

d. Reduction in the ability of the highway to accommodate the traffic demand within the level of service range desired.

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

FIELD STUDY SITE SELECTION

A number of parameters were established in the development of this project to the extent that they constituted a limitation on the choice of field study sites. It was found that the requirement of a two-lane rural pavement intersecting a nonprimary two-lane roadway ruled out the majority of the higher volume Iowa intersections. Initial investigation of potential study sites was primarily based on vehicular volume in an attempt to make the time spent in data gathering as fruitful as possible.

A review of possible study sites was held with the Secondary Roads Engineer and engineers in the planning division at the Iowa State Highway Commission. On a personal judgment basis, a number of sites were noted for investigation. This arbitrary initial selection basis was made on county traffic maps which had the volumes of traffic noted on each road link.

Having established a potential site list, each location was then evaluated in more detail. Details of the traffic volumes were investigated, especially the left-turn volumes. Where left-turn conflicts of any significance could be anticipated, a review of the physical site conditions followed. It was felt that a relatively unobtrusive safe location near the intersection must be available for the observers.

On the basis of reasonable potential left-turn conflict volumes, and on adequate observation locations, four central Iowa study sites were selected for obtaining field data:

Old US #30 and Dayton Road east of Ames

US #69 and Pine Hill Road north of Des Moines

Iowa #5 and SW 63rd Street west of Des Moines

Park Ave. and SW 63rd Street south of Des Moines

Figures 2 through 5 are ground level photographs at the study sites. Table 2 summarizes the traffic information available.

Fig. 2. Old #30 and Dayton Road east of Ames (looking west).

Fig. 3. US #69 and Pine Hill Road north of Des Moines (looking north).

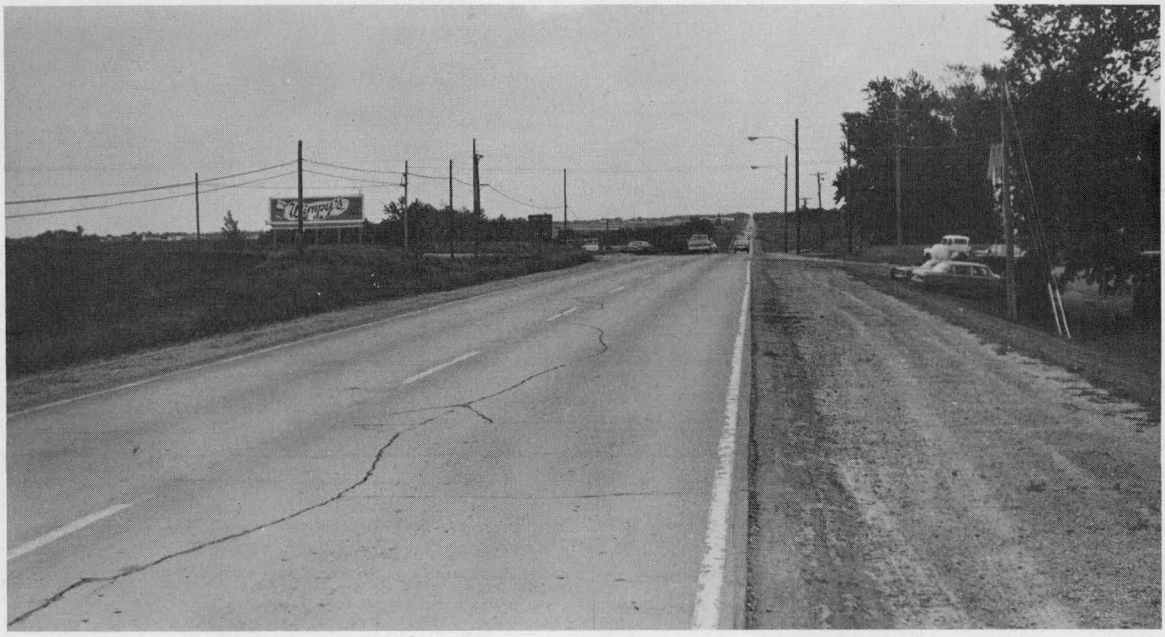


Fig. 4. Iowa #5 and SW 63rd west of Des Moines (looking east).

Fig. 5. Park Ave. and SW 63rd south of Des Moines (looking north).

Table 2. Traffic volume relationships at the study sites.

Location	Year of traffic count	DHV		AADT		Percent Truck	Percent Left turns
		Approaching	Opposing	Approaching	Opposing		
Old #30 & Dayton Road	1968	295	177	2423	1409	7	36
US #69 & Pine Hill Road	1968	423	395	3369	3622	6	11
Iowa #5 & SW 63rd	1966	240	329	2086	2886	7	17
Park Ave. & SW 63rd	1970 [a]	526	243	5000±	2500±	1	75

[a] Estimated from a two-hour count during a peak period.

INTERSECTION STUDY TECHNIQUE

A number of investigators have noted the problems associated with the gathering and interpreting of data regarding traffic performance at intersections (15 – 18). The usual practice is to use one of three techniques:

1. A graphic recorder is employed which has a moving paper on which up to 20 pens record responses from input electrical signals denoting the spatial arrangement of vehicles in relation to time. Hand switches, pavement detectors, signal controllers, etc., or combinations, provide the signal.
2. Time-lapse photography uses a series of time-spaced 16 mm photos to record all vehicular movements in the field of vision. This record can be later used to retrieve any particular characteristic of performance that can be visually identified.
3. Observers with synchronized watches and stop watches record each vehicular event as it occurs during the study period.

The information available from these procedures is detailed. For example, using the time-lapse photography technique with field reference lines, practically any traffic characteristic can be reproduced and evaluated in the office. The graphic recorder and the time-lapse photography technique however, require relatively expensive equipment, and involve tedious and time consuming reduction of data. This is especially true in the photographic method, which also has the disadvantage of requiring a reasonably elevated close-field vantage point. The observers with synchronized clocks require very close coordination, and usually a large number of observers are required, which has an adverse effect on maintaining the natural setting.

In many cases a less detailed analysis of traffic performance may be required and a simpler technique would be desirable. Wayne Volk, in a 1956 study on vehicle delay (18), noted the problem and advanced a simplified procedure.

He provided a metronome with a one-second audible click and a comptometer as the tallying device to record stopped time delay. When a vehicle came to a stop, the operator pushed the #1 key of the comptometer each time the metronome clicked, until the vehicle moved out. A number of observers were needed to make counts, operate the comptometers and observe and record the counts.

Harmelink (9) utilized the services of five persons with synchronized clocks to observe and record traffic performance at intersections.

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 insofar as the authors are able to determine.

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

The procedure is based on utilizing one tape recorder to "play back" a prerecorded signal of accurately spaced one-second clicks. This background time reference is played at the site from the first recorder, while the second tape recorder is recording the traffic events which are being

translated into audible form by two observers. The result is a tape which, when played back in the lab, yields a time band (which is referenced to real time) with interspaced coded sounds identifying specific traffic events. Note that this procedure provides what might be termed an "audiograph," similar to a visual graph obtained from a 20-pen recorder.

After a period of experimentation, a master background time signal tape cassette was created by using an electronic metronome with an audible beep. Adjustments were made until a deviation of less than 5 seconds in 30 minutes existed. A countdown (5, 4, 3, 2, 1, 0) initiates the start of the 1-second interval "clicks" for the purposes of coordination with a stop watch or reference to real time. Every 5 seconds the accumulated 5 seconds of incremented time is audibly noted on the sound tape. Each full minute is also accumulated and noted, up to 30 minutes. The initial tape was "gang" duplicated so that a number of masters were obtained.

At the site one observer can usually provide the audio input code for traffic characteristics on one approach to the intersection. The following alphabetical code was developed for this project and proved to be the least confusing combination of letters in the retrieval stage.

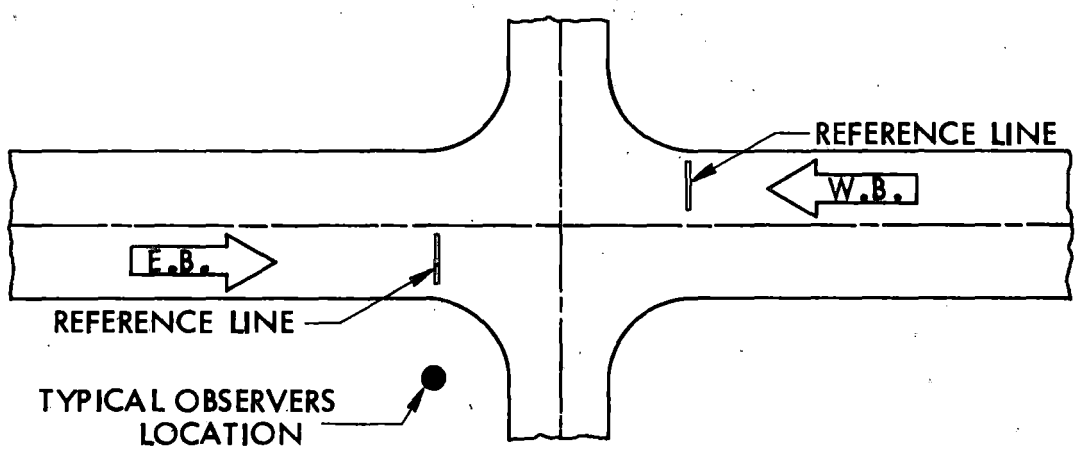


Fig. 6. Typical study-site characteristics.

East Bound Traffic

straight through — no stop .....A

left turn — no stop .....B

right turn — no stop C

vehicle stops .....Q

straight through after stop .....W

left turn — after stop X

right turn — after stop .....Y

West Bound Traffic

straight through — no stop L

left turn — no stop .....H

right turn — no stop .....F

vehicle stops .....O

straight through — after stop .....R

left turn — after stop .....M

right turn — after stop .....J

The two observers sat in the front seat of a passenger vehicle at a location which would allow unobtrusive observation of the reference lines and vehicle arrivals as noted in Fig. 6. Frequently these pavement reference lines were not themselves directly visible to the observers, but a line of sight reference was established so that a vehicle crossing the reference line could be noted. With the background time signal "turned on" and the second recorder "recording," each vehicle crossing the reference line or stopping was noted by calling out the appropriate alphabetical code. The result was a 30-minute tape yielding an audio time graph of traffic activities.

During the data retrieval phase the audio signals were reduced to their occurrence in time. Figure 7 illustrates the form utilized by the data retriever. It was found that it was quite easy to estimate the occurrence of an alphabetical letter code to within 0.2 second. Frequently the tape was stopped and replayed for a few seconds repeatedly until the time occurrence could be accurately identified. The only difficulty occurred when letter codes fell close together, which required repeated playback to be able to separate and identify the individual time occurrences of each letter.

After the occurrences of the traffic events were related to a time on the standard form, it was easy to determine gap and lag characteristics, headways, and delay time. The form was expanded to include columns for these calculations.

Location _____ Date _____ Observers _____

Observation period _____ Sheet No. _____

TIME	VEHICLE MOVEMENT														GAP		LAG		DELAY			HEADWAY	
	A	B	C	Q	W	X	Y	L	H	F	O	R	M	U	acc.	rej.	acc.	rej	lt.	th.	rt.		

Fig. 7. Data reduction form.

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 one-way traffic stream on a two-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 four-lane road carrying moderate volumes of traffic. However, if opportunities for passing are restricted, as must be the case on a two-lane two-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 than would be the case if arrivals were entirely random.

For this investigation, field data on vehicle headways were gathered for 94 one-way traffic streams representative of peak-hour conditions. One-way rates of flow during peak 15-minute periods varied between 148 vehicles per hour 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 seconds. 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 indicated in Fig. 8. In this figure, the observed frequencies of occurrence of headways of 4 seconds or shorter are compared with those calculated 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.

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 upon 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 as follows:

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

where

y = cumulative frequency of occurence of headways equal to or shorter than t

t = headway in seconds

q = one-way traffic volume in 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 occurrence of headways of any given length may be derived from Eq. (1):

$$P = \frac{0.04708\ q^{0.6094}}{(t - 0.9)^{0.6319}} \cdot \qquad (2)$$

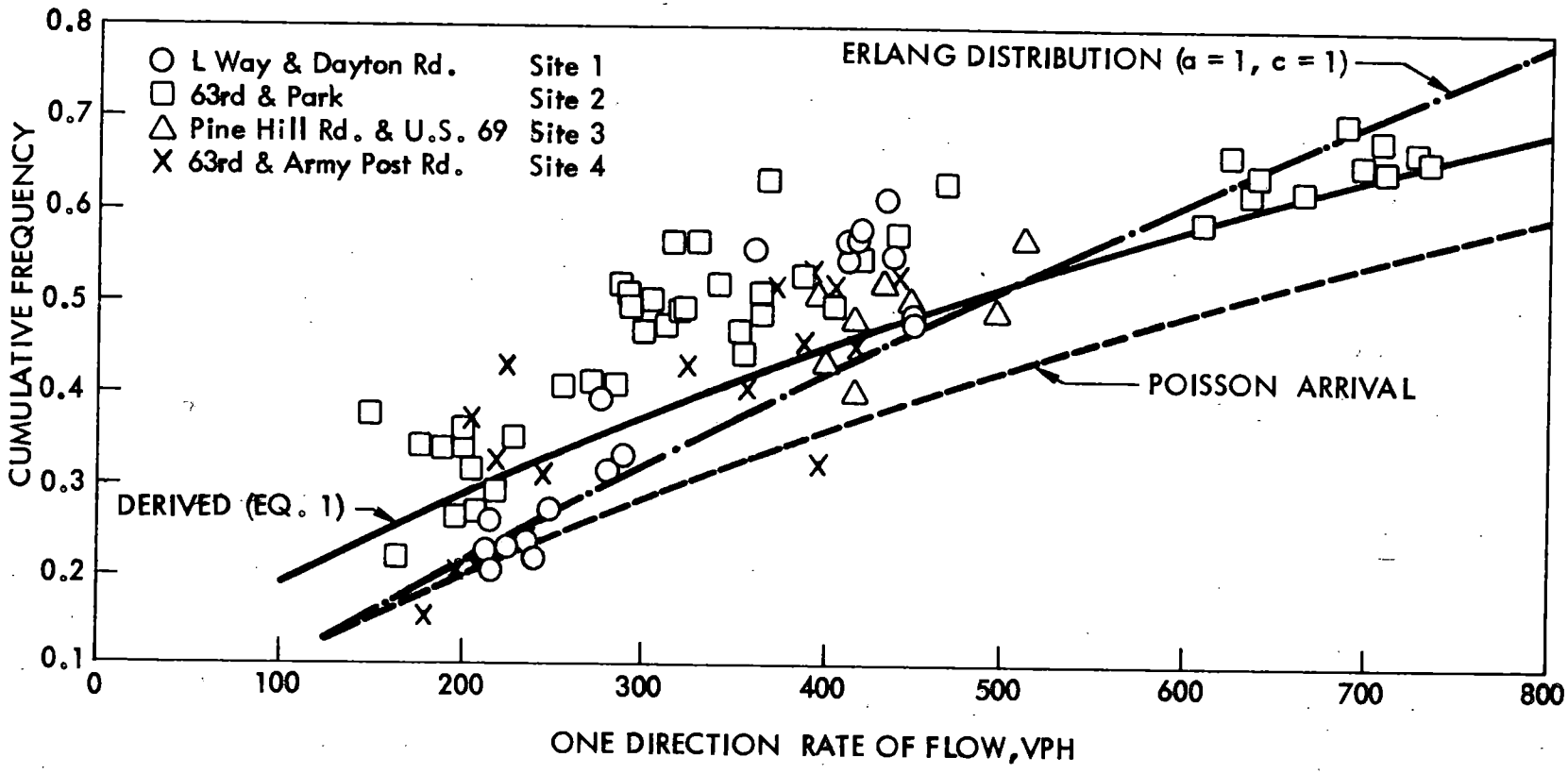


Fig. 8. Frequency of occurrence of headways of 4 seconds or less.

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 assuming Poisson arrival illustrates the effect of platooning. This example is based upon t = 4 seconds and q = 4.48 (448 vehicles per hour in one direction). Three samples from the observed data are also shown for comparison in Table 3.

Table 3. A comparison of theoretical equation headways with actual data.

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, vehicle delay could be estimated by appropriately considering the effect of queueing. Definitions pertinent to this discussion are as follows:

Gap — A gap is defined here as the headway (time space head-to-head

of successive vehicles) in the traffic stream opposing a vehicle that is stopped preparatory to effecting a left-turn maneuver.

Lag — A lag is that portion of a gap between the time of arrival of a left-turning vehicle (that has not stopped) at a point where it encroaches upon the opposing traffic lane and the arrival at the same point of an opposing vehicle.

Critical Lag or Gap — A critical lag or gap is one of a duration such that the same number of vehicles have accepted a lag or gap of that length or shorter as have rejected one of that length or longer. The probability of either acceptance or rejection of a critial gap is considered to be one-half. Acceptance means that a driver confronted with a choice has elected to utilize a lag (if his vehicle is in motion) or gap (if his vehicle has stopped) to complete a left-turn maneuver.

Critical lags and gaps were determined from data gathered at each study site. These did not differ significantly from one 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 four study sites. See Figs. 9 and 10. Results of this analysis are follows:

Critical lag	3.5 seconds
Critical gap for one car to complete left turn	5.5 seconds
Critical gap for two cars to complete left turn	7.3 seconds
Critical gap for three cars to complete left turn	9.5 seconds
Critical gap for four cars to complete left turn	11.6 seconds

Sample sizes for gap acceptance by more than one car were not sufficiently large 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 seconds when effecting left turns from a stop.

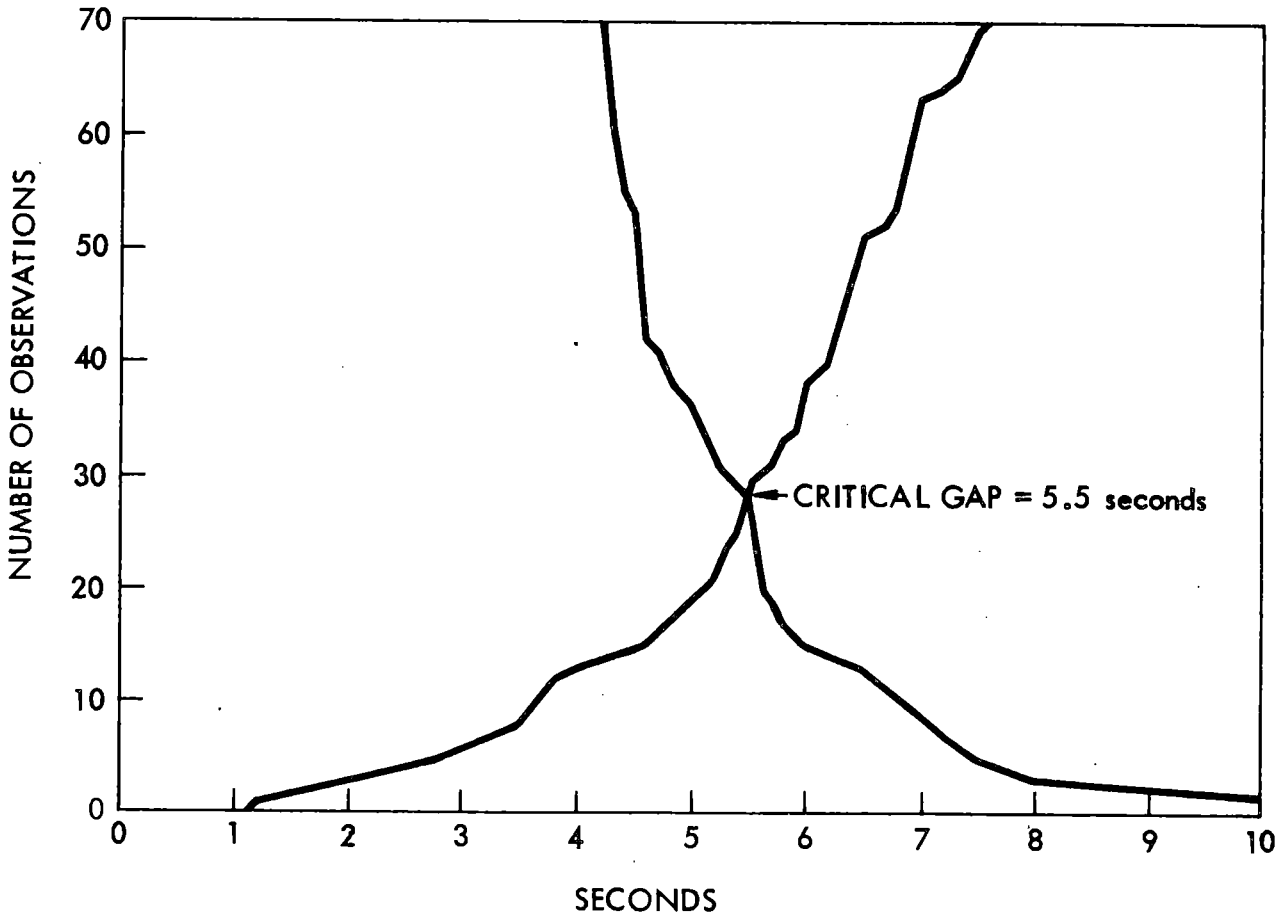


Fig. 9. Consolidated totals — critical gaps.

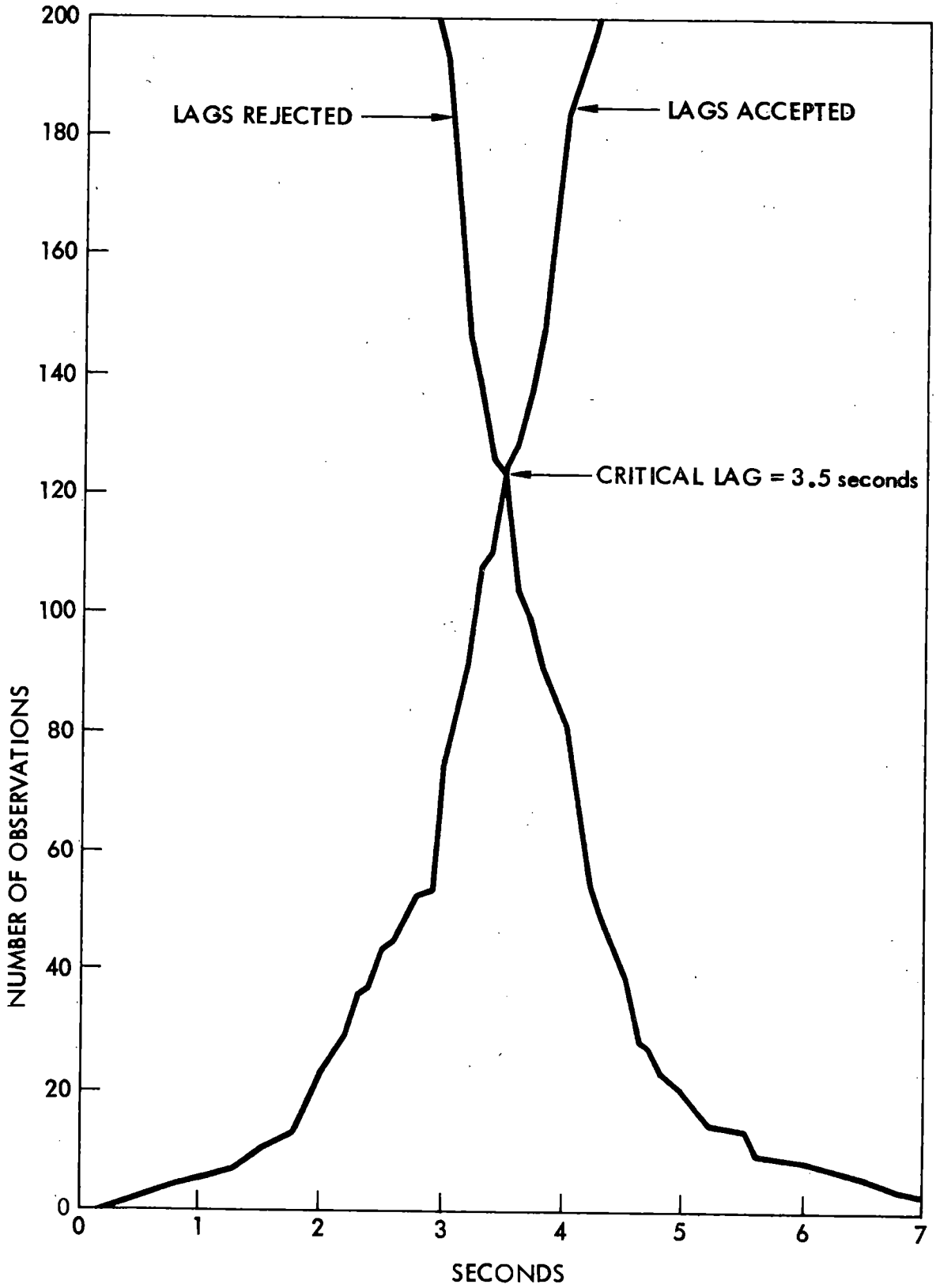


Fig. 10. Consolidated totals — critical lags.

Theoretical Stops and Delays vs Actual

With an expression for the spacing of vehicles in the traffic stream and knowledge of lag and gap acceptance characteristics, it is possible to 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 virtually all samples. The research principals concluded that results from an approach based upon this methodology could not be supported by the observed behavior of drivers at the test sites.

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 the 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 and 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 so that he lengthens a lag to a level acceptable to him and completes his turn without stopping. Or, he may decrease speed slightly to avoid an unacceptable lag. He may then complete his left turn immediately after an oncoming car has cleared the

intersection and avoid the necessity for a stop. The angle and location at which a driver crosses the opposing lane also may be varied so as to reduce delay and the necessity for stopping. If the approaching side-road lane is not occupied, drivers frequently initiate left turns very early by turning at a flat angle so as to clear the opposing lane before the arrival of an oncoming car. They may also delay their turns (without stopping) to permit an oncoming vehicle to clear and then cross the opposing lane with a turn of very short radius. Driver behavior, while difficult to predict with certainty, generally will be directed toward minimizing the amount of delay and the necessity for stops.

An Alternate Approach

Since the researchers were not able to accurately forecast driver behavior using a theoretical model, the mass of observed data was examined again to determine its predictive ability. Using multiple regression techniques, the following equations were derived:

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

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

where

D = average standing delay for all advancing vehicles in seconds

S = proportion of advancing vehicles that stop

q = one-way volume of opposing traffic in hundreds of vehicles per hour

a = one-way volume of advancing traffic in hundreds of vehicles per hour

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, shown in Table 4, 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 volumes in either traffic stream.

Table 4. Correlation matrix.

Variable	9	a	L	D	S
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 from the anticipated 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 as follows:

1. Due to 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 overstate 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 or not 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 two-way traffic in the data from which Eqs. (3) and (4) were derived. We must assume that the effect of the opposing 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.

As indicated above, Eqs. (3) and (4) may be multiplied by the advancing volume to yield total hourly delay and a 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 the 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 1967 – 1969. The following equations in terms of daily traffic volumes result when these factors are used:

$$D_D = A_a(2.147\ L + 0.00002393\ A_q + 0.00002669\ A_a) \qquad (5)$$

$$S_D = A_a(0.3071\ L + 0.000004228\ A_q - 0.000001931\ A_a) \qquad (6)$$

where

D_D = daily standing delay for all advancing vehicles in seconds

S_D = number per day of advancing vehicles that stop

A_a = one-way volume of advancing traffic in vehicles per day

A_q = one-way volume of opposing traffic in vehicles per day

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. But it will remove left-turn vehicles from the through lane so that straight through and right-turning vehicles may proceed essentially without delays. 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 the amount of delay accruing to through and right-turning vehicles which were 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:

$$K = 0.6134\ L - 0.5744\ L^2 \quad (0.0 < L \leq 0.8).$$

Values of K are tabulated in Table 5.

Table 5. Values of K

Proportion of left turns	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
0.30	0.132
0.35	0.145
0.40	0.153
0.45	0.160
0.50	0.163
0.55	0.163
0.60	0.161
0.65	0.156
0.70	0.148
0.75	0.137
0.80	0.123

COST-BENEFIT COMPARISON

Reduction in Vehicle Operating Costs

Benefits to road users through reductions in operating costs and time were calculated for two typical situations that are representative of most rural intersections in Iowa. See Table 6.

Table 6. Posted speed versus running speed

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 shown in Table 7.

Table 7. Unit costs for passenger vehicles.

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

The value for time saved is that suggested in reference 19. Other values are taken from reference 20.

An equivalency factor is commonly used to account for the presence of trucks in the traffic stream. This is indicative of the average relationship between operating and time costs for commercial vehicles and those 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 then multiplied by the costs (or benefits) calculated for passenger cars to account for the increased costs (or benefits) occasioned by commercial vehicles. T varies as shown in Table 8 depending upon the percentage of trucks in the traffic stream:

Table 8. Truck factors.

Percent trucks	T
0	1.00
5	1.10
10	1.20
15	1.30
20	1.40
25	1.50

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

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

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

where b_{70} and b_{55} are the annual reductions in operating and time costs for 70 mph 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)

typically is 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 four 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.

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

The first two items were obtained from the accident report forms. In some cases judgment was needed to interpret the forms as to whether the accident could be considered preventable. Obviously the estimated property damage was dependent on the officer's judgment, and in most cases must be considered incomplete and inaccurate to a degree. The personal injuries were noted, but no information regarding degree of injury or costs was available. A summary of the accident information obtained is presented in Table 9.

Table 9. Summary of accident records at selected Iowa rural intersections (1965 – 1970).

Location	Number of preventable accidents	Estimated property damage	Number of personal injuries involved
1. 63rd & Park Ave.	3	$ 885.	2
2. 63rd & Army Post Rd.	4	1200.	1
3. US #69 & N.E. 54th	9	3750.	1
4. Old #30 & Dayton Rd.	3	1112.	1
	19	6947.	

Due to the extremely small sampling no statistical significance can be associated with this information. However, a generalized statement regarding the accident information summarized in Table 9 is as follows:

- About one preventable property damage accident occurred per year per intersection.
- About $350 reported property damage was estimated at each property damage accident.
- 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, further supplementary information was reviewed. A number of studies have been conducted which assign a dollar value for the cost of various types of accidents.

The National Safety Council (21) in 1965 established the following schedule of accident costs:

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 (22) reviewed prior research in Massachusetts in 1953, in Utah in 1955, and in Illinois in 1959. Primarily based on the Illinois study they upgraded the accident costs to 1968 California conditions. They arrived at the following schedule:

Fatal	$9,700
Nonfatal injury	2,500
Property damage	500

(Note that only the direct costs of a fatal accident are considered, which explains the difference in 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:

Property damage accident ..... $ 500

Nonfatal injury $2500

As a result of the accident investigations at the four study sites, it was determined that the preventable accident rate norm would be set at one property damage and one-fifth of a personal injury per year. This decision yields $500 + (1/5)$2500 = $1000 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. Due to the rare occurence of fatal accidents, the consideration of this type would severely distort the small samples taken in this study.

Accident cost estimation will normally take one of two forms: (1) <u>an investigation</u> of the accident records at the intersection under review; (2) <u>an estimation</u> of the difference between accident rates that could be anticipated, by a comparison to similar situation records and then forecasting.

Condition (1) is appropriate for a reconstruction of the existing intersection type of project. In this case accident records are available and of sufficient detail that all preventable accidents (involving a left-turning vehicle) can be distinguished from nonpreventable type accidents. Five year records averaged to an annual basis are usually recommended because of frequently encountered variations and the minimal numbers involved. If only preventable accidents are retrieved for analysis, it can be assumed that all of this type will be eliminated with the construction of a separate left-turn lane. Consequently the annual cost saving (benefit) is the product of the

number of preventable accidents and the appropriate average cost factors noted.

Condition (2) requires the application of personal knowledge and judgement. A number of attempts have been made to predict accident rates using multi-variate equations based on studies of existing locations. Jorgensen (23), Shaw (12), and Hammer (13), discuss the merits of this technique. In most cases the resulting equations presented are completely unacceptable for widespread use. Differences in local geometric design, record keeping, driver attitudes and behavior, and selection of variables, tend to negate any potential application of this technique for other than local use.

The California study as reported by Hammer (13) and the Jorgensen report (23) have determined a predictive technique. It is based on using an estimated accident reduction rate obtained from before and after accident studies. For example, it is noted that the accident rate has been shown to be reduced about 30 percent with the inclusion of a left-turn lane. Consequently with the vehicular volume rate and accident cost factors, an estimate of annual accident savings can be made.

In the absence of detailed local accident study and analysis, the following approach for estimating annual accident cost reduction on a relocation design project is most applicable. Identify local two-lane intersections that as nearly as possible have geometrics, volumes, traffic operating characteristics, and adjacent land influence similar to the study site. Obtain accident records and identify the preventable accidents. Using the accident cost factors previously presented, arrive at an annual accident cost reduction value, and apply it to the relocation site.

In establishing the relative warrants the preventable accident costs at the field study sites were utilized. That is, $1000 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 $1500 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 (B/C) 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 two-lane pavement width 16 feet to provide for a separate left-turn storage lane. Painted pavement markings are used to effect the channelization. It is interesting to note that the California left-turn lane warrant statement previously discussed (13) includes the statement ... "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 two-lane pavement in one case, and a standard channelized intersection in the alternate case. The design is the standard ISHC type as shown in Fig. 11. Unit prices were obtained from the ISHC contracts, right of way, and maintenance departments reflecting current prices. A summary of the construction and maintenance cost items are presented in Table 10. Note that a very significant part of the capital costs is the initial investment in lighting. This design includes six - 400 watt mercury vapor luminaires.

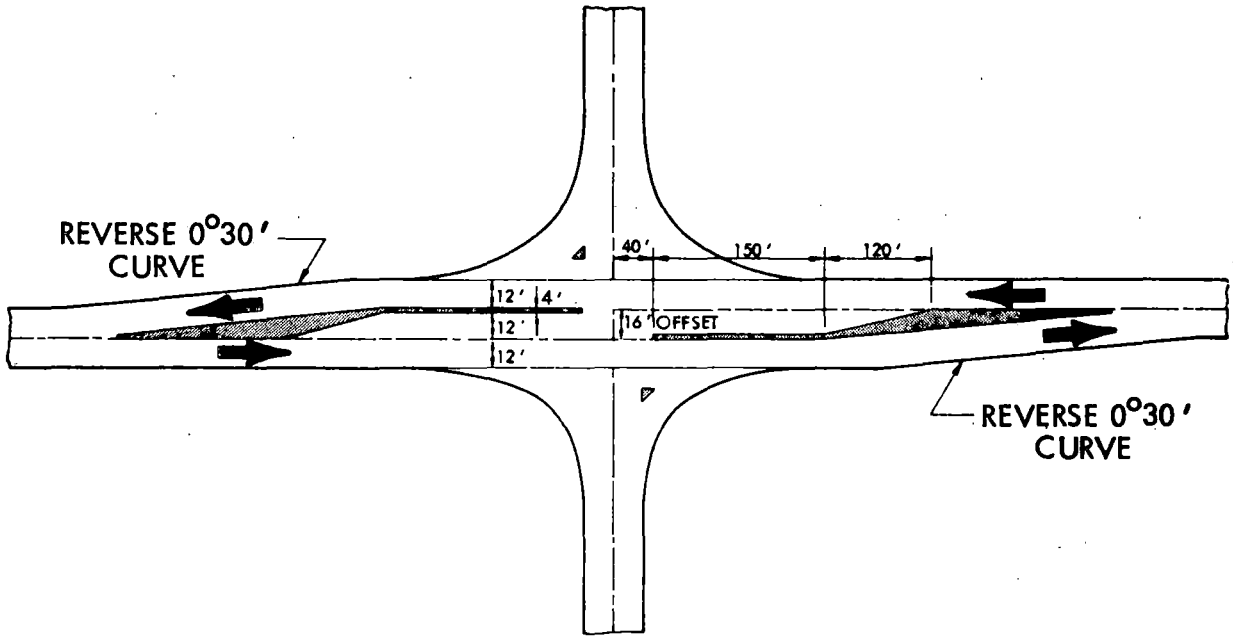


Fig. 11. Typical primary road intersection.

Table 10. Construction and maintenance costs (Iowa).

Item	Additional incurred costs of a 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 & arrow painting	$ 250
rumble strip	250
snow removal & salting	50
lighting maintenance & energy	60
Total	$ 610

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:

$$\Delta C = \left[C_1 K_1 + C_2 K_2 + \ldots C_n K_n\right] + \Delta M \qquad (9)$$

where

ΔC = average annual cost difference incurred by the construction of a channelized intersection

C_n = the capital costs of individual construction items

K_n = the capital recovery factors for a specific interest rate and service life

ΔM = the average additional annual maintenance costs incurred due to the construction of a channelized intersection.

For the purposes of this study it was assumed that the service life and interest rate were the same for each item of construction. The interest rate selected was 6 percent which is currently used in the ISHC planning division studies. The service life was selected as 20 years for every construction element. Consequently, the calculations of normal annual construction costs are:

$$\Delta C = 24{,}496(0.087185) + \$610 = 2746 \ .$$

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.

$$B/C_{70} = \frac{K \cdot T \cdot A_a \left[5.160\ L + 0.00006991\ A_q - 0.00002443\ A_a\right] + C_a}{2746} \quad (10)$$

$$B/C_{55} = \frac{K \cdot T \cdot A_a \left[3.685\ L + 0.00004961\ A_q - 0.00001516\ A_a\right] + C_a}{2746} \quad (11)$$

where:

B/C_{70} = the benefit-cost ratio for an area having a posted speed of 70 mph

B/C_{55} = the benefit-cost ratio for an area having a posted speed of 55 mph

K = a factor which is a function of L, and represents the proportion of through and right-turning vehicles which are stopped. See Table 5.

T = a factor representing the effect of trucks, and obtained from Table 8.

A_a = one-way volume of advancing traffic in vehicles per day.

A_q = one-way volume of opposing traffic in vehicles per day.

L = proportion of left turns in the advancing traffic stream.

C_a = annual accident cost saving.

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

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 Iowa two-lane highways. Distributions based on random arrivals do not correlate closely with actual one-way traffic stream data. An alternate approach was tested utilizing multiple regression analysis of field data to describe the frequency of headways. Then, with a knowledge of lag and gap acceptance characteristics, the theoretical magnitude of stops and delays can be calculated. 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 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 are added to a potential reduction in accident costs. When 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 greater than one, the construction is warranted. If less than one the construction is not warranted, (based on these factors alone).

Warrants for Left-Turn Lanes at Two-Lane Rural Intersections

For determining the B/C ratio in a specific application the following techniques are presented:

(a) benefit-cost ratio mathematical equations may be solved.

$$B/C_{70} = \frac{K \cdot T \cdot A_a \left[5.160\ L + 0.00006991\ A_q - 0.00002443\ A_a\right] + C_a}{\Delta C} \qquad (12)$$

$$B/C_{55} = \frac{K \cdot T \cdot A_a \left[3.685\ L + 0.00004961\ A_q - 0.00001516\ A_a\right] + C_a}{\Delta C} \qquad (13)$$

where:

B/C_{70} = benefit-cost ratio for posted speeds of 70 mph

B/C_{55} = benefit-cost ratio for posted speeds of 55 mph

K = obtain from Table 5

T = obtain from Table 8

C_a = obtain from accident record studies (or use the norm of $1000)

ΔC = obtain from cost estimates (or use the norm of 2746)

A_a = one-way volume of advancing traffic in vehicles per day

A_q = one-way volume of opposing traffic in vehicles per day

(b) For repetitive applications the mathematical formula have been reduced to nomograph form. See Figs. 12 and 13. Three values for A_c are incorporated into the nomograph representing a range each side of the $1000 norm value. The value of $2746 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 $2746 to the new value.

(c) For the case of equally distributed opposing and advancing traffic volumes, ($A_q = A_a$) a series of simplified charts (Figs. 14 – 19) have been presented for various posted speeds and accident cost savings (A_c). 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.

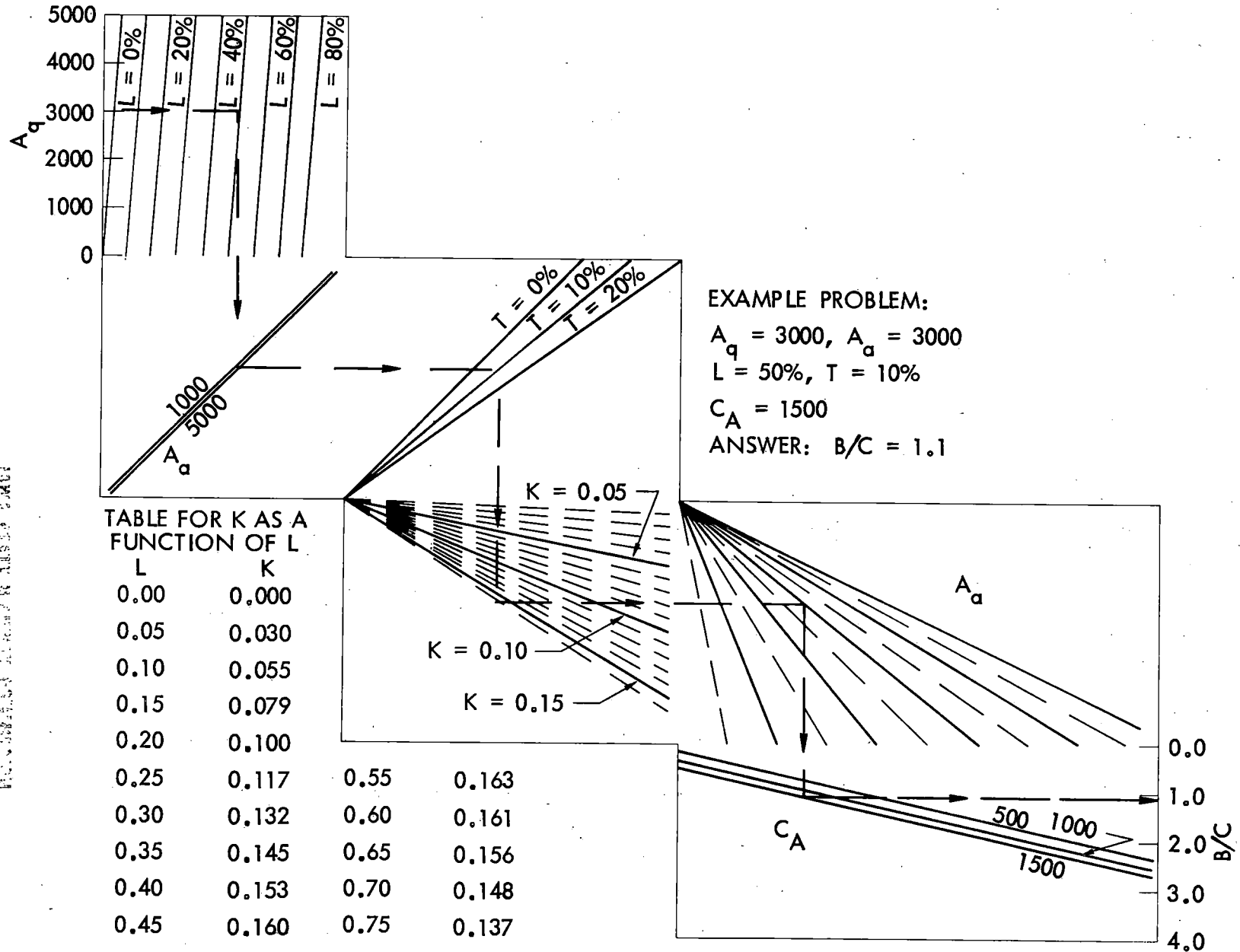
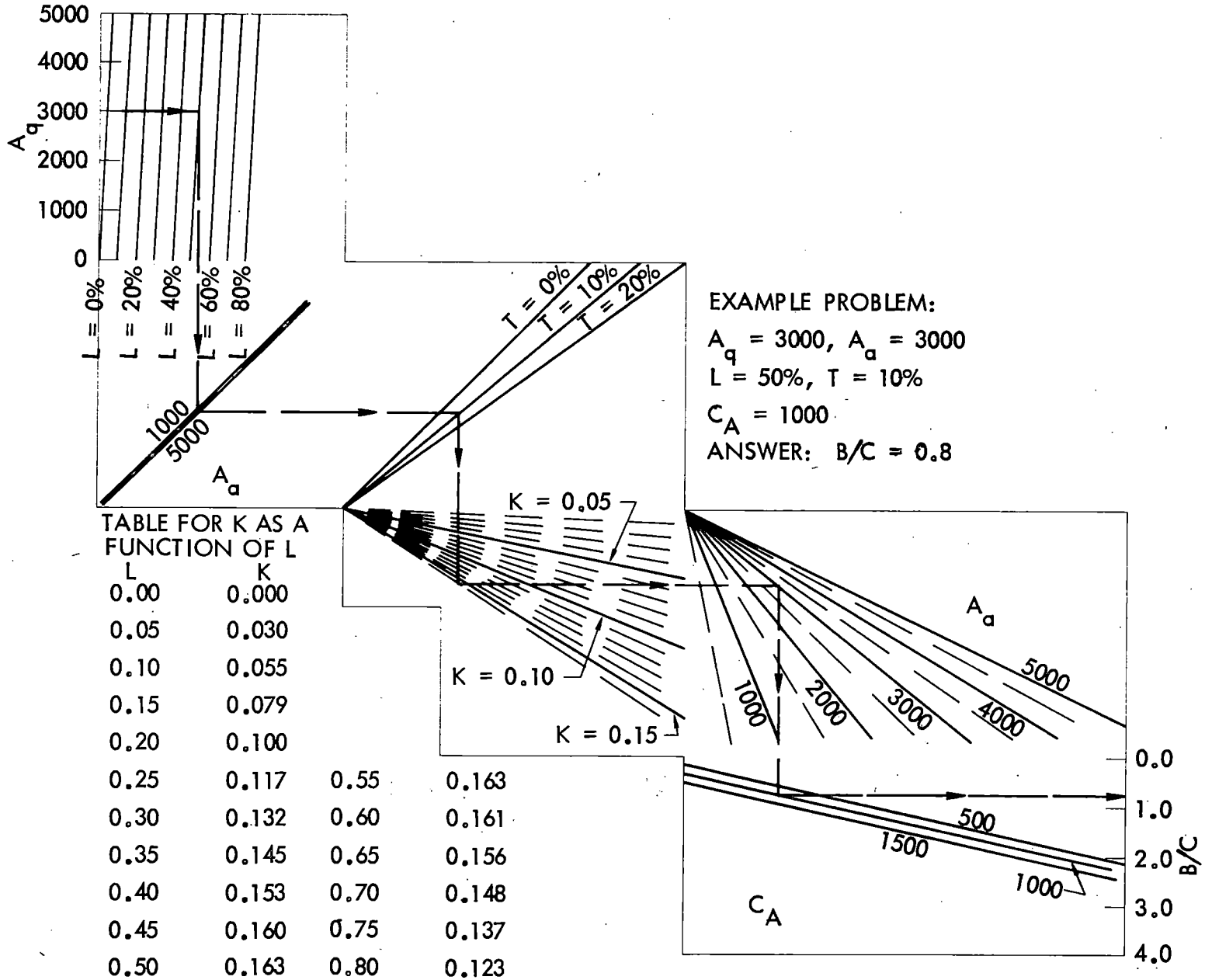


TABLE FOR K AS A FUNCTION OF L

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

Fig. 12. Nomograph for calculating benefit-cost ratio for left-turn lane, posted speed = 70 mph.



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0.40	0.153	0.70	0.148
0.45	0.160	0.75	0.137
0.50	0.163	0.80	0.123

Fig. 13. Nomograph for calculating benefit-cost ratio for left-turn lane, posted speed = 55 mph.

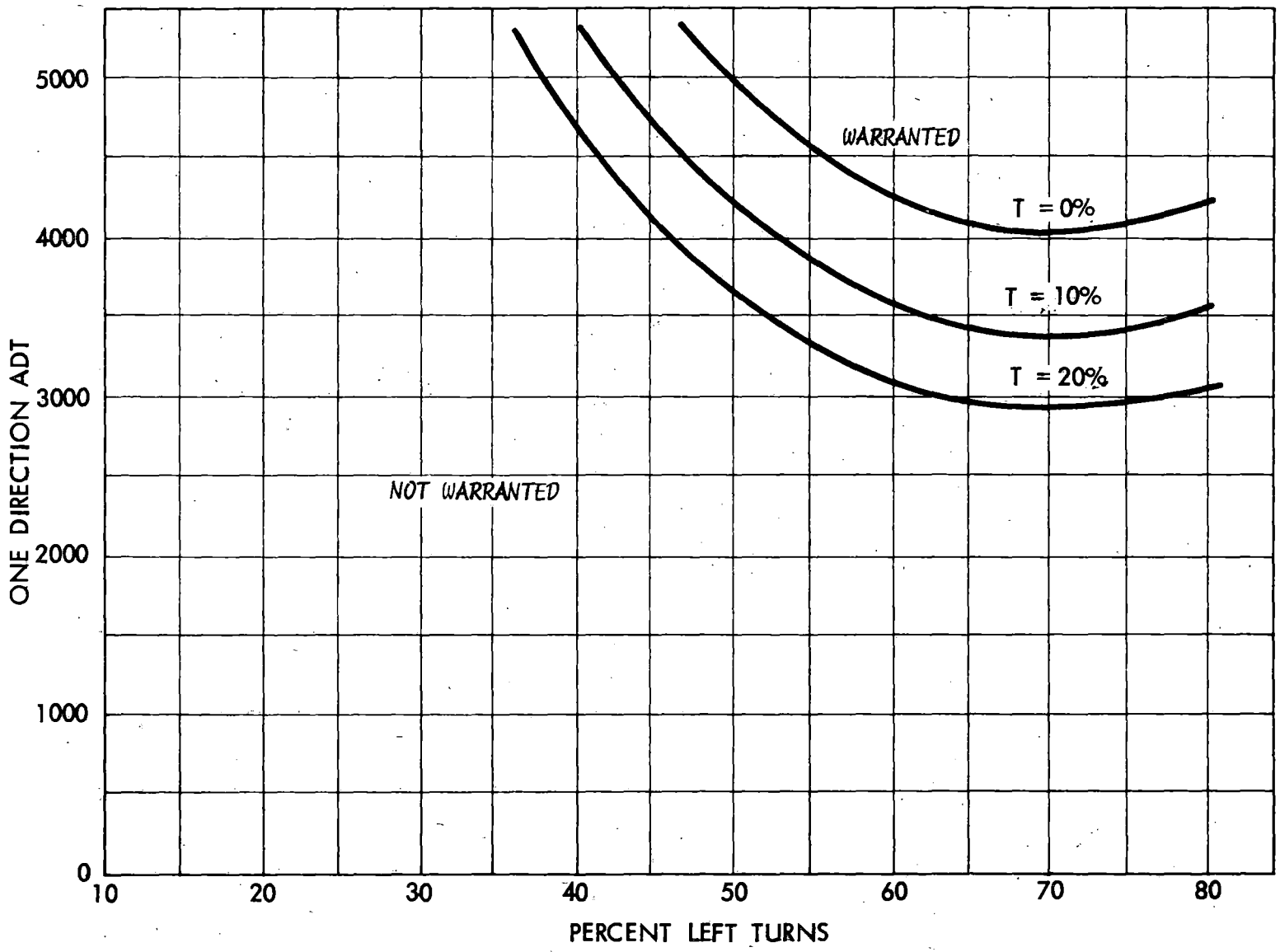

Fig. 14. Warrant for left-turn lane: posted speed = 70 mph, annual accident cost reduction = $500.

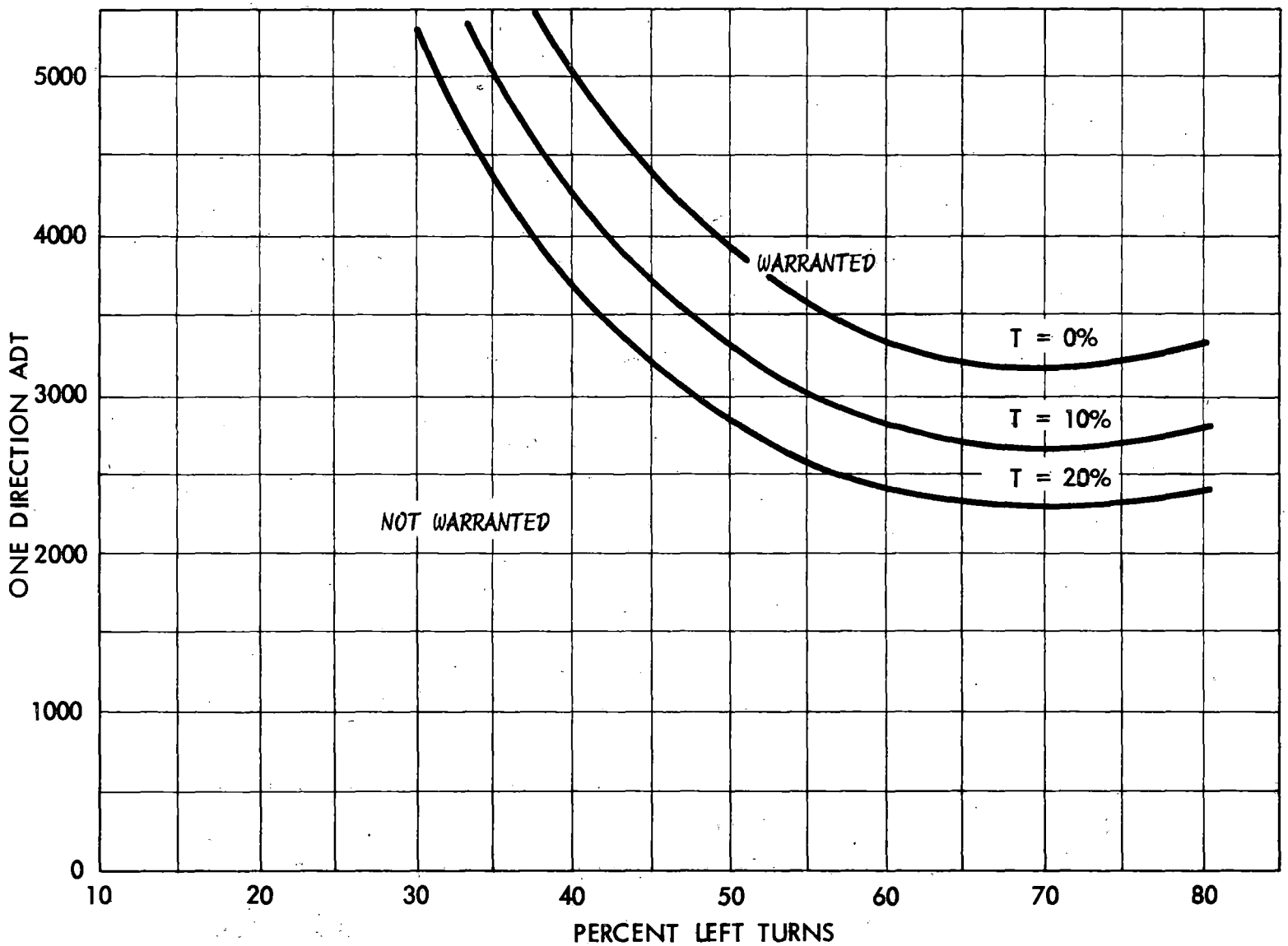

Fig. 15. Warrant for left-turn lane: posted speed = 70 mph, annual accident cost reduction = $1000.

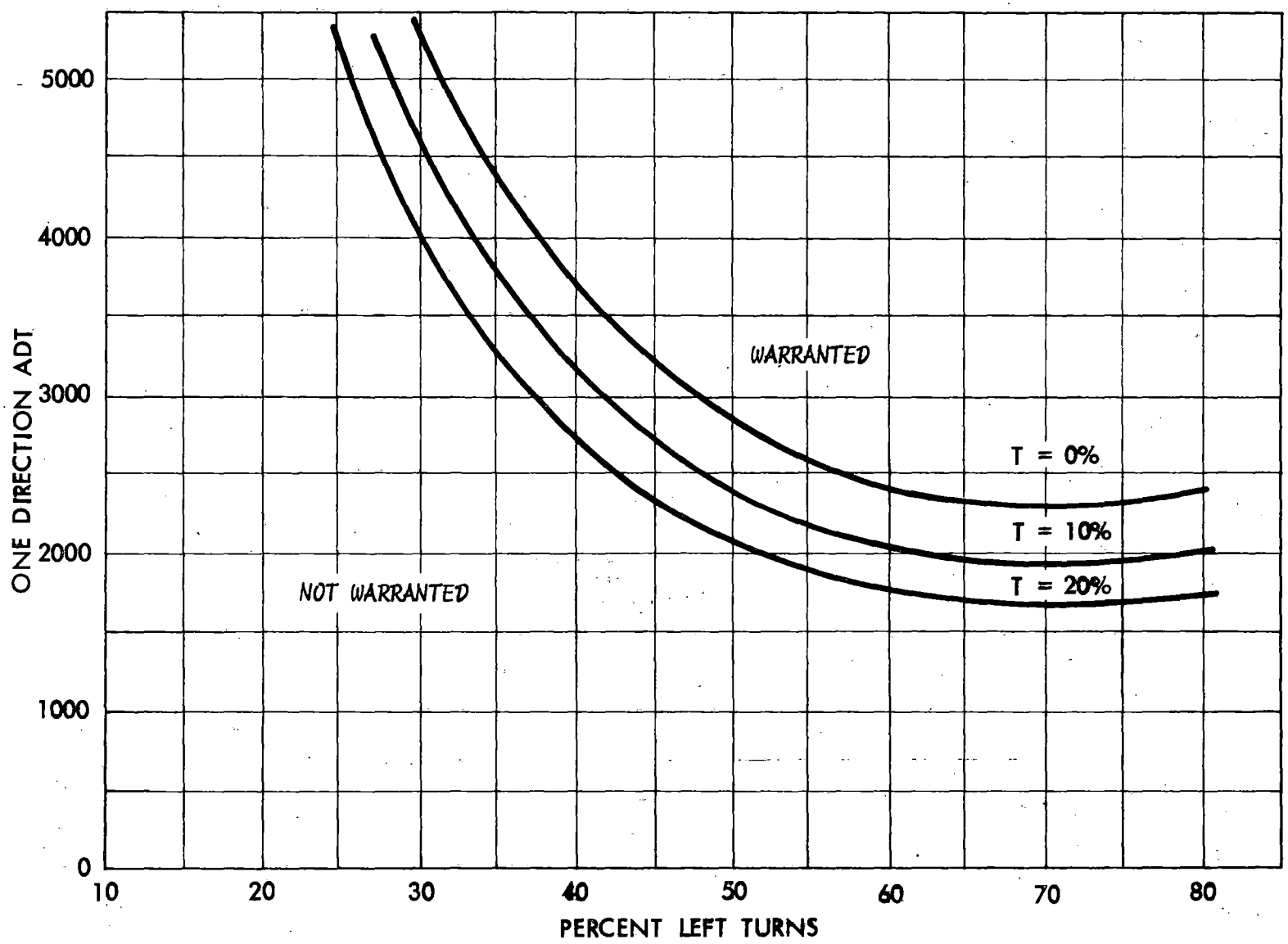

Fig. 16. Warrant for left-turn lane: posted speed = 70 mph, annual accident cost reduction = $1500.

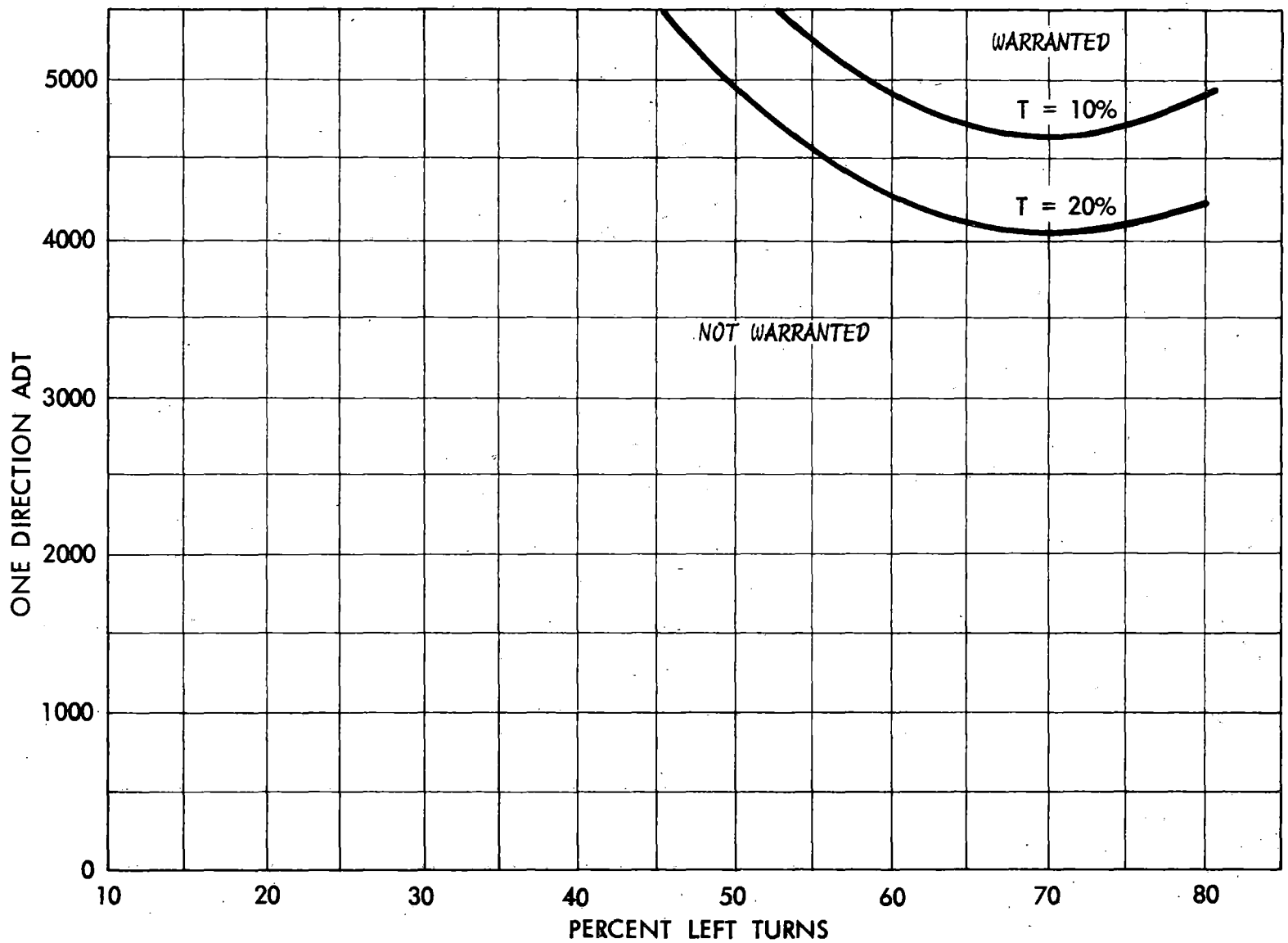

Fig. 17. Warrant for left-turn lane: posted speed = 55 mph, annual accident cost reduction = $500.

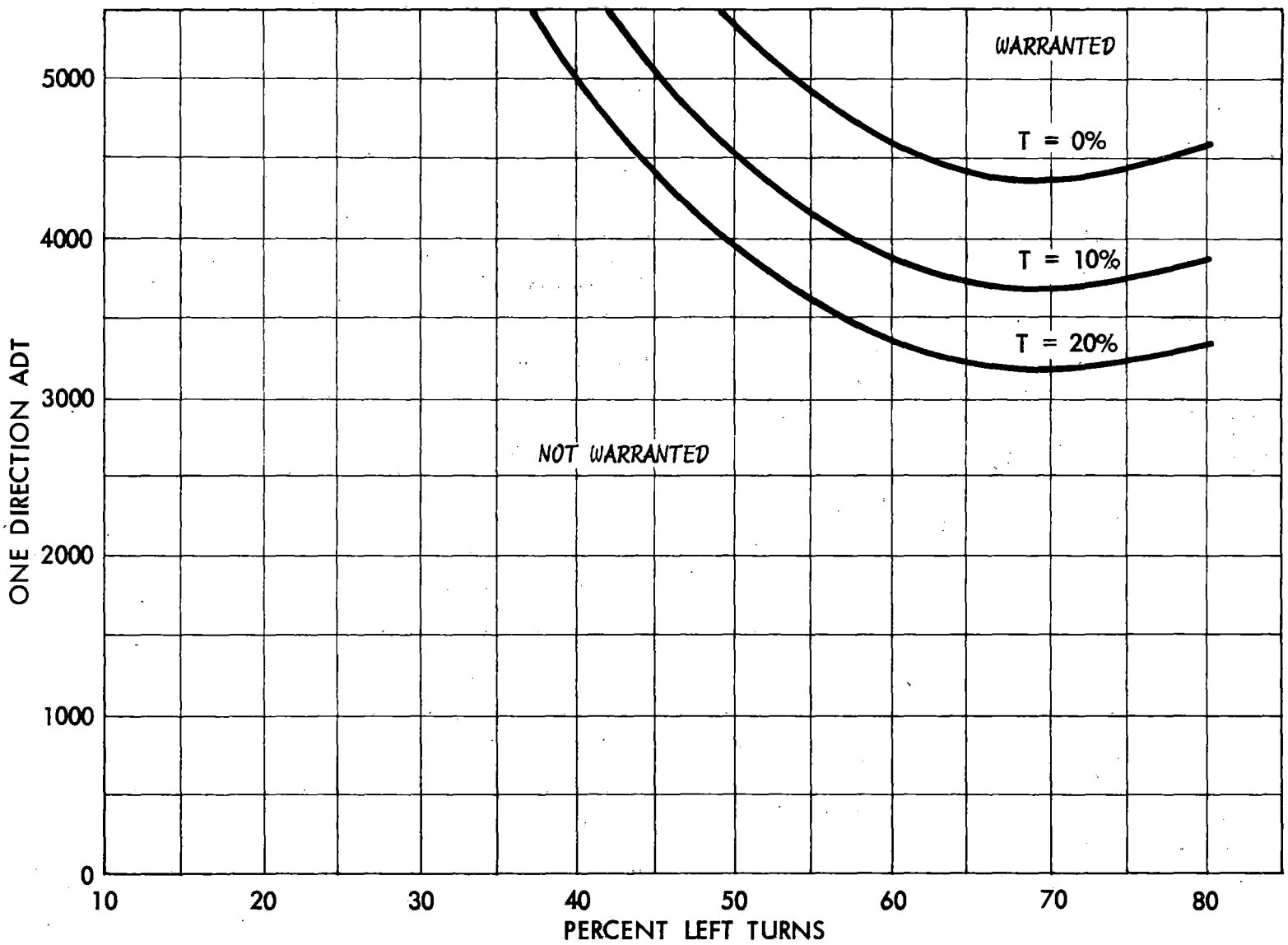

Fig. 18. Warrant for left-turn lane: posted speed = 55 mph, annual accident cost reduction = $1000.

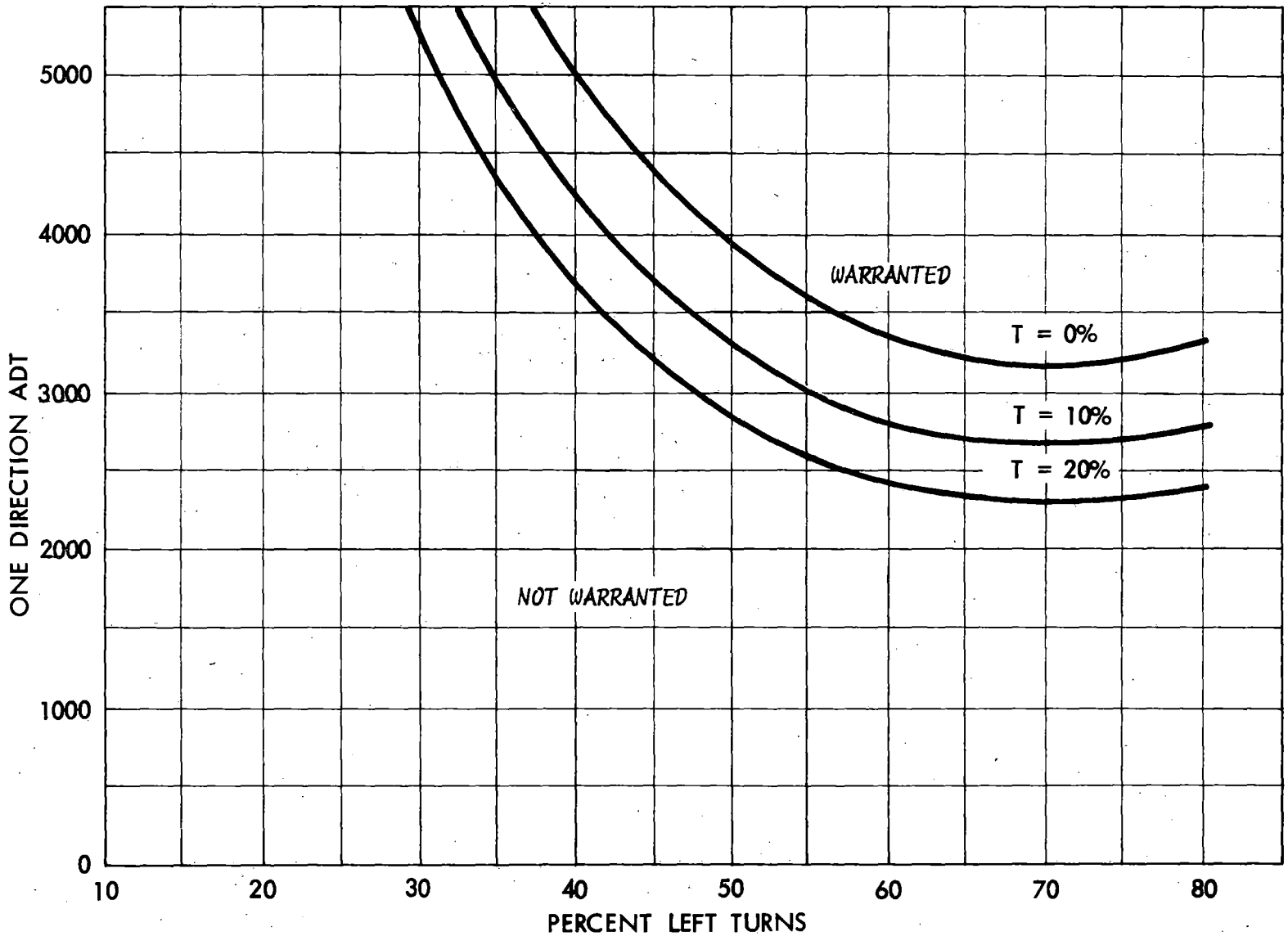

Fig. 19. Warrant for left-turn lane: posted speed = 55 mph, annual accident cost reduction = $1500.

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APPENDIX

Example Problems

1. Given Data:

ADT approaching traffic volume = A_a = 1800 vpd

ADT opposing traffic volume = A_q = 1800 vpd

L = 20%, T = 20%, Posted Speed = 70 mph

Preliminary Decisions:

A standard channelized intersection design is to be evaluated, therefore ΔC = \$2746.

No site accident records are available, therefore use the norm of \$1000 for A_c.

Solution:

Because the distribution of traffic flow is balanced use chart, Fig. 15.

Note the intersection of lines is well within the not warranted zone.

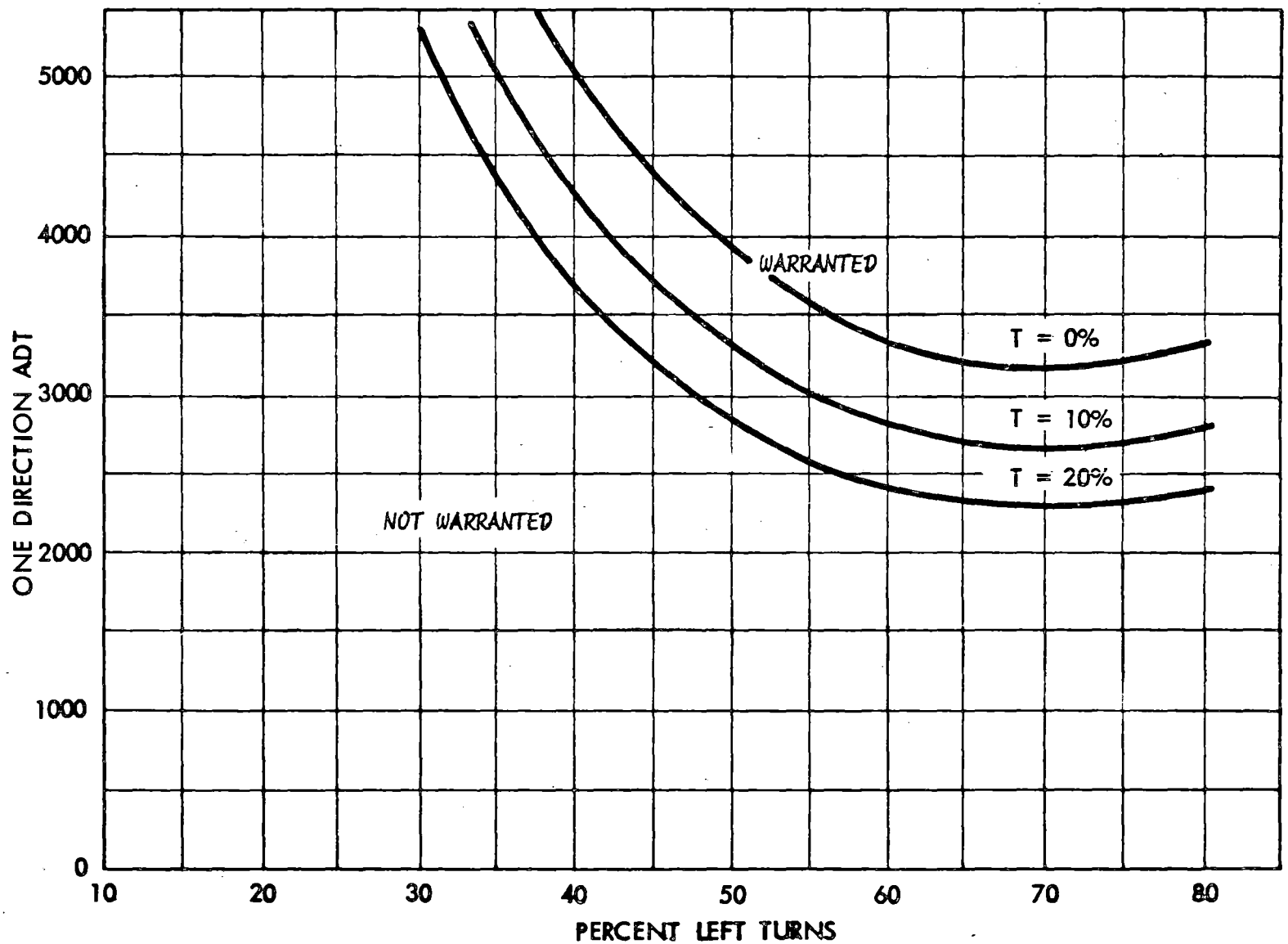
5000
4000
3000
2000
1000
0
ONE DIRECTION ADT
10
20
30
40
50
60
70
80
PERCENT LEFT TURNS
WARRANTED
NOT WARRANTED
T = 0%
T = 10%
T = 20%

Fig. 15.

2. Given Data:

A_a = 3500 vpd

A_q = 1800 vpd

L = 40%, T = 10%, Posted Speed = 55 mph

Accident record investigation at the site yields an annual preventable accident cost of $1500.

Due to a side road "T" configuration the intersection design costs have been reduced yielding a $\Delta C + \Delta M$ = $1400.

No additional information is required.

Solution:

Because the traffic flow distribution is unbalanced the charts are not applicable. Use Fig. 19.

3. Given Data:

A_a = 2700 vpd

A_q = 2700 vpd

L = 30%, T = 20%, Posted Speed = 70 mph

Accident record analysis at this site yields a high value of $2000 for annual preventable accidents.

Construction costs are reduced at this site due to geometrics, with ΔC calculated at $2100.

Solution:

Because of the variation in accident costs and annual project costs the mathematical solution would be used.

$$B/C_{70} = \frac{0.132(1.40)(2700)[5.160(.30) + 0.00006991(2700) - 0.00002443(2700)] + 2000.00}{2100.00}$$

$$= \frac{559 + 2000}{2100} = 1.22$$

and, construction is warranted.

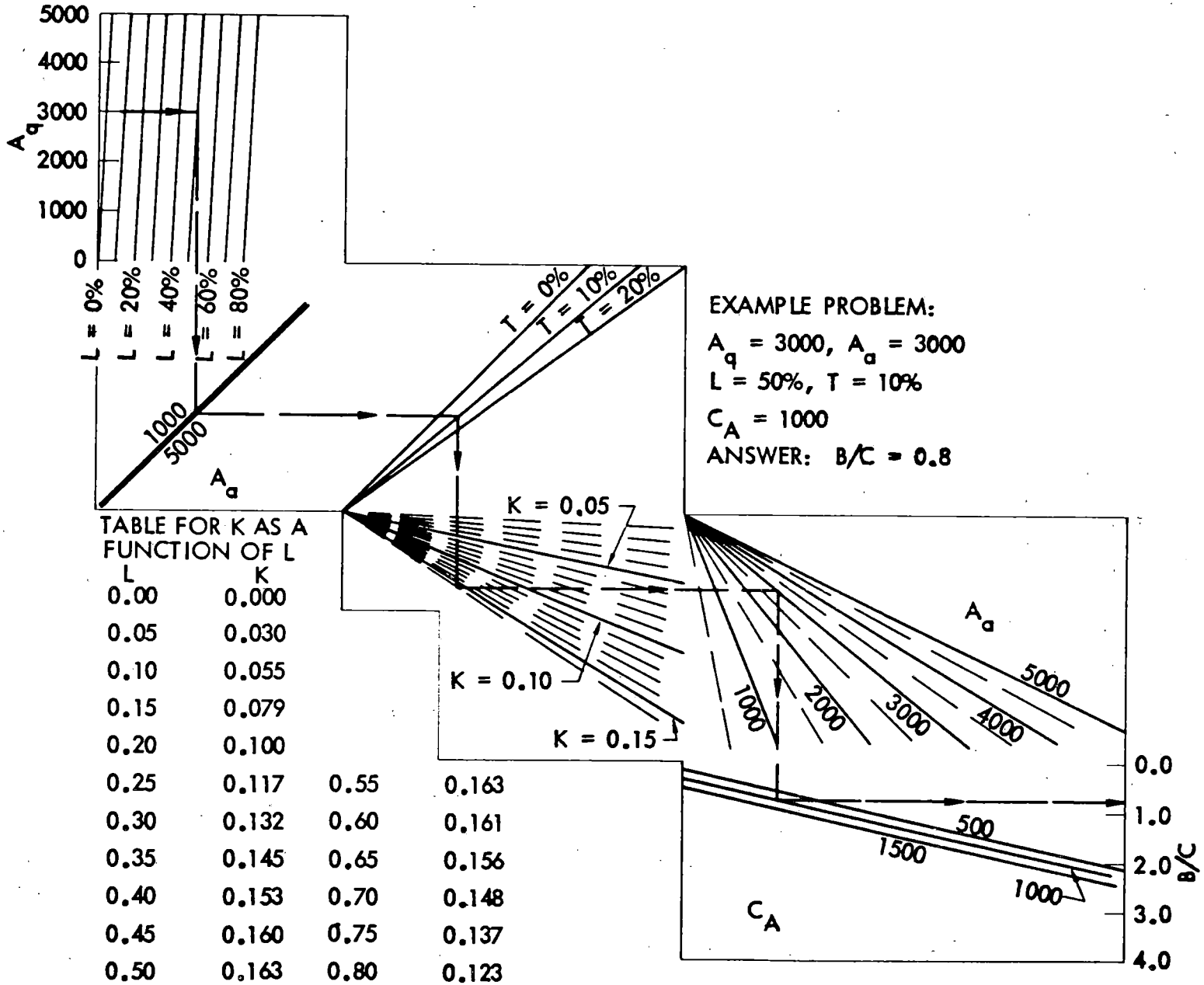

TABLE FOR K AS A FUNCTION OF L

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

Fig. 13.