HIGHWAY RESEARCH RECORD

Number 312

Relationships of Highway Geometry to Traffic Accidents

9 Reports

Subject Areas

51	Highway Safety
52	Road User Characteristics
53	Traffic Control and Operations

HIGHWAY RESEARCH BOARD

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Foreword

Highway design engineers having responsibility for development of design standards and for applying them to specific design situations will find the several papers in this RECORD of interest. Traffic engineers having responsibility for safe operation of facilities may also find the papers useful in pointing to remedial measures on existing highways. Both should find this RECORD helpful in furthering the necessary dialogue between designer and operator.

The RECORD opens with a broad-based study that attempted to determine which rural geometric variables contributed most to accidents in Louisiana. Dart and Mann investigated accident records for a 5-year period on approximately 1,000 miles of highway in an attempt to find the relationships of 11 different variables. They concluded that the 2 geometric variables having the most important effects on accident rates are horizontal alignment and pavement cross slope, and they ranked the other variables in order of decreasing effect.

A Bureau of Public Roads researcher sought relationships between accidents and the lengths of speed-change and weaving lanes. Julie Anna Cirillo used accident data from Interstate segments in 24 states to show that accident rates in weaving areas decrease with increased length of the area for weaving. In addition, she showed that increased length had a beneficial effect on accidents in both acceleration and deceleration lanes, although the effect was greater for acceleration lanes. Effects are also related to the proportion of merging traffic in the main stream. Understanding of this work is also enhanced by discussions that follow the paper.

Glennon reports on his review and evaluation of current AASHO stopping sight distance standards. He draws a number of conclusions in the nature of challenges to several elements of design criteria and offers recommendations for both immediate action and further research. As with the preceding paper, a thoughtful discussion extends the value of this work.

In the fourth paper, a Kentucky researcher examined accident experience on limited-access facilities to determine the need for and value of median crossovers. Garner found direct relationships, and he presents guidelines that could result in a 5 percent reduction in accidents on such facilities.

Computer analysis of Interstate accident data permitted Yates to develop relationships between accident rates and curvature on loop and outer connection ramps. Data are presented to show these relationships for several types of ramps in common usage.

The next paper describes work in Israel relative to visibility problems in crest vertical curves. In it, Livneh and others describe the theoretical, as opposed to graphical or field measurement, determination of passing restriction zones from the standpoint of passing maneuver safety on 2-lane roads.

Yates and Beatty also used computer analysis of Interstate accident data in their search for relationships between mainline lighting and accident experience. They report finding higher accident rates for lighted highways than for unlighted highways, although the only characteristics considered other than lighting were ADT and number of lanes.

Glennon made a state-of-the-art evaluation concerning truck operating characteristics and accident involvement rates as related to the design of truck climbing lanes. His conclusions offer practical advice for the designer faced with decisions regarding application of these special lanes for trucks in order to maximize safety of operation.

In the final paper, Rizy and others investigated passing performance with in-car displays on 2-lane roads. This work was in connection with the Passing-Aid System II study and resulted in a rejection of an auditory display in favor of a flashing visual display.

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Relationship of Rural Highway Geometry to Accident Rates in Louisiana

OLIN K. DART, JR., and LAWRENCE MANN, JR., Louisiana State University

The purpose of this project was (a) to determine which geometric variables contribute most to accidents, and (b) to predict the accident potential of a certain section of Louisiana highways. This study involved approximately 1,000 miles of rural highways distributed evenly throughout Louisiana. The accident records investigated cover a 5-year period from 1962 to 1966. The variables studied in order to find their relation to accidents were percentage of trucks, traffic volume ratio, lane width, shoulder width, pavement cross slope, horizontal alignment, vertical alignment, percentage of continuous obstructions, marginal obstructions per mile, and traffic access points per mile. These 10 variables, their squares, and their first order interactions were used in a regression analysis to construct mathematical models to determine the contribution of the variables to total accidents, accidents on wet roads, accidents on dry roads, accidents during the day, accidents during the night, total injuries, and total fatalities. One mathematical model shows that a total of approximately 46 percent of all accidents are explained by the 10 variables included in this study. The variables not investigated—involving the driver, the vehicle, and other geometrics—account for the remainder of the variation in total accidents. Based on their interaction with traffic volume, the 2 geometric variables having the most important effect on accident rates are pavement cross slope and traffic conflicts. The remaining geometric variables studied in order of decreasing effect on accident rates are lane width, horizontal alignment, and shoulder width.

•FOR SEVERAL YEARS Louisiana has found itself in the unenviable position of possessing the highest motor vehicle accident fatality rate in the nation. Table 1 gives the trend for a 10-year period from 1957 through 1966. Particularly alarming is the rate of increase in accidents and fatalities between 1962 and 1966.

In designating the cause of accidents, most authors cite 3 principal factors—the driver, the vehicle, and the roadway. In one concept, recently recalled by Baldwin (2), each accident is assigned a primary cause that results in 75 percent of all accidents being caused by driver failure, 10 percent by vehicle deficiencies, and 15 percent by highway deficiencies. However, Baldwin goes on to discuss the more probable situation that any accident is caused by an interaction of the 3 principal factors:

> . . .If it is possible to hazard a guess, it seems likely that the driver as a cause might show up in from 80 to 90 percent of accidents, but that the highway would also appear in 40 to 50 percent of the cases and the vehicle in a somewhat smaller percentage (but certainly larger than the commonly quoted ten percent). By the very nature of the concept, there would of course be many accidents in which causal factors might be identified in each of the three general categories. This would merely mean that more than one type of countermeasure might be useful in terms of accident prevention.

Paper sponsored by Committee on Operational Effects of Geometrics and presented at the 49th Annual Meeting.

TABLE 1

LOUISIANA HIGHWAY ACCIDENTS AND ENFORCEMENT FROM 1957 TO 1966

Year	Rural Accidents	Rural Fatalities	Automobile Fatalities	Death Rate per 100 mvm[a]	Hazardous Citations	Ratio of Citations to Accidents
1957	17,052	568	817	7.3	-	-
1958	16,301	608	845	7.4	-	-
1959	16,972	560	803	7.2	45,501	2.68
1960	15,623	594	841	7.4	51,699	3.31
1961	15,732	576	779	6.7	63,351	4.02
1962	17,299	546	770	6.4	71,876	4.16
1963	19,638	624	846	8.3	67,017	3.41
1964	23,934	791	1,037	9.7	68,667	2.82
1965	25,904	857	1,106	9.0	78,992	3.05
1966	29,916	917	1,232	9.4	89,061	2.98

[a]Accidents per 100 million vehicle-miles.

A large part of Louisiana's problem may be caused by the driver in that enforcement has not been maintained at a sufficient level in recent years (Table 1). Authorities say that a 6 to 1 ratio of moving violation citations to total accidents is a minimum for effective control. Louisiana's ratio has generally ranged from between 2.6 and 4.2 to 1 in the 8-year period shown.

Nevertheless, Baldwin's point is well taken, and we resolved to pursue all available avenues of attack, including an elevation of highway deficienceis. This paper describes the work done on the project initiated in 1966 and includes the analysis, results, conclusions, and recommendations having to do with the rural highway system in Louisiana. The purpose of the research was (a) to determine what continuous type of geometric highway design variables may be contributing most to the large traffic accident and death toll that exists in Louisiana, and (b) to predict accident potential and geometric design parameters. Prediction is the elusive goal here. Unless accidents can be predicted above the level of chance, the processes that cause accidents cannot be understood with any degree of confidence.

Miller (23) reports that traffic accidents were responsible for an estimated total economic loss of $11 billion in the United States in 1967. Thus, there is that much reason, strongly reinforced by humane considerations, to gain a better understanding of the causes of highway accidents so that we may reduce their frequency and the distress they cause.

The highway itself is an important determinant of where accidents happen (35). Roadway elements or characteristics, therefore, can provide an effective means for predicting accident "proneness" of a given roadway. The purpose of this research was to estimate the true liability of the highway itself in causing accidents and to assess the proportionate share of liability that individual physical highway characteristics should bear.

PREVIOUS STUDIES

The literature contains many items dealing with investigations of the relationship of highway design features to highway accidents. One publication alone summarizes the findings of several hundred references (35).

Some previous research dealt with the problem in its entirety (7, 9, 12, 30); other research was limited to one variable such as shoulders or a group of factors (3, 5, 14, 29). In reviewing the material available, the authors limited their search to studies that dealt more directly with the problem under investigation. Some of the most pertinent findings of previous research are briefly summarized in the following discussion.

In 1953, Raff (30) presented his findings on a study to find how accident rates on main rural highways are affected by design features and use characteristics. The factors studied include number of lanes, average daily traffic (ADT), degree of curvature, pavement and shoulder widths, frequencies of curves and other sight-distance restrictions, percentage of intersection traffic on the minor road, and many others. Surprisingly, Raff concluded that a number of roadway features, including grade and frequency of curves, do not appear to have any consistent effect on accident rates. He also found, as would be expected, that traffic volume and sharp curves seem to cause accidents. Wide pavement and shoulders encourage safety on 2-lane curves. Raff mentions that the most striking feature of his study was the amount of irregularity in most of the results, but he adds that the principal cause is probably the tremendous complexity of the problem itself. The very complexity of the problem dictates that the data should be as homogeneous as possible. This can only be accomplished by having only one reporting agency furnish all the data. The authors have tried to do this by using data from one state.

Blensly (5) found a significant relationship between increased accident frequency and increased paved shoulder width. He states that "the results of this study should be interpreted with extreme caution, inasmuch as the traffic volumes on the bulk of the sections were less than 5,000 vehicles per day." Two different approaches were taken in his study. Correlation procedures were used to evaluate the relationship between paved shoulder width and accident occurrence, and variance measures were employed to analyze the difference between the average accident frequency on sections with narrow paved shoulders (4 ft or less) and the average accident frequency on sections with wide paved shoulders (8 ft or more). The analysis of covariance procedures established that, when the effect of ADT was controlled, the mean number of property damage and total accidents was significantly higher on sections with wide paved shoulders than on sections with narrow paved shoulders in the 1,000 to 5,600 ADT range.

Belmont (3) found a similar relationship between personal injury accident frequency and paved shoulder width on sections ranging in volume from 2,000 to 12,000 vehicles per day. Belmont concluded that "apparently the advantages of wider shoulders are more than offset by tendency of drivers to be less careful. As shoulder width increases, drivers may gain an unjustified feeling of security." This theory is one possible explanation for the relationship found in his study. Clearly, this is a startling result. Another explanation can possibly be found in the way that Blensly attenuated his data. In order to obtain a sufficient number of sample elements, he multiplied each 1-mile section of rural 2-lane highway, which was level and tangent and had paved shoulders, by the number of full years for which accident data were available after the paved shoulders were constructed.

Moskowitz (26) and Crosby (10) investigated the relative safety of various existing types of median designs. Although the research described did not include an analysis of medians, the references are included for continuity.

Schoppert's investigation (31) describes research by the Oregon State Highway Commission to develop equations that can be used to predict accidents on rural 2-lane highways from roadway elements such as ADT, lane width, shoulder width, sight-distance restrictions, commercial and residential driveways, and intersections. A sample of nearly 1,400 miles of 2-lane highways is utilized. The data were analyzed through the use of multiple correlation techniques. The result of the analysis is a series of equations that can be used to predict total accidents on rural 2-lane highways in Oregon.

In 1966, Roy Jorgensen and Associates and Westat Research, Inc., published (17) the results of a research project for the Bureau of Public Roads. This work is a valuable source of procedures for evaluating safety programs and identifying spot locations. The section on forecasting accident reduction relates the regression analysis technique to the problem and appeals for accident record data to be coordinated with existing highway geometric characteristics.

Two reports released after the start of our project have direct bearing on this type of research and are summarized here. Sparks (32) describes a project to investigate the influence of highway characteristics on accident rates in Oklahoma. The approach

used was first to select independent variables thought to be related to accident occurrence and then attempt to measure the amount of correlation each variable has with some form of accident index. The accident rate per million vehicle miles was used as the dependent variable. The ultimate goal of this research was to develop one formula that could be applied to all classifications of rural highways and that could be used to predict accidents effectively so that hazardous locations might be identified and corrected prior to accident occurrence. As a secondary benefit, he hoped that the formula also would detect or measure the true importance of various highway elements. The independent variables that he attempted to correlate with the accident rate (the dependent variable) were surface width, surface type, shoulder width and type, curvature, gradient, stopping sight distance, passing opportunity, hazard rating, surface condition, and shoulder condition. As expected, most of the independent variables had negative correlations with the accident rate. The surprising aspect of the study was the low correlations that actually existed between the independent variables and the dependent variable. Only 1.91 percent of the variance in the accident rate was explained by the independent variables used in this study. The standard error of the equation was in excess of 5.00 indicating that the equation is practically worthless for predicting accident rates.

Kihlberg and Tharp (18) of the Cornell Aeronautical Laboratory investigated rates of accidents caused by specific geometric features such as number of lanes, access control, median presence, highway curvature, gradient, ADT, and the presence of intersections or structures. From these data a mathematical model was evolved but no coefficient of correlation was given; however, multiple R^2 values are available in the appendix of their report. Of course, the model is valid only for the data considered, i.e., for Connecticut, Florida, and Ohio. Briefly, Kihlberg and Tharp found that (a) access control had the most powerful accident-reducing effect; (b) rates for one-vehicle accidents per million vehicle miles decrease with increasing ADT while those for multiple-vehicle accidents increase; (c) without access control, undivided 4-lane highways have accident rates higher than those on 2-lane facilities; (d) medians tend to decrease the number of accidents, although the effect was not clear-cut; (e) curvature, gradient, intersections, and structures increase accident rates, the dominant element being intersections and the least significant being gradient; (f) combinations of these elements generate accident rates higher than those of the individual elements; and (g) no evidence substantiated the hypothesis that the effects of grades and curves varied with steepness or sharpness above the limits of 4 percent and 4 degrees.

Fairly recent research by Mulinazzi and Michael (27), although dealing with urban arterials, produced multiple linear regression models relating accident rates to design characteristics. One hundred sections ranging in length from 0.254 to 4.167 miles were studied with regard to 26 independent variables. One conclusion is that accident rates increase with increasing numbers of "friction points" per mile (the "sum of the number of approaches to the arterial, intersections, and driveways").

UNIFORM SECTION DEFINITION AND INITIAL SECTION SELECTION

This research of the effects of highway geometry on the frequency of occurrence of automobile accidents was concerned with rural highway sections. Each section selected was required to possess uniform characteristics as follows:

1. Same pavement and shoulder width throughout the section;
2. Same pavement and shoulder type throughout the section;
3. No reconstruction during the period from 1962 to 1966;
4. Generally consistent alignment throughout the length of the section (spot locations, e.g., single excessively sharp horizontal curve, usually distinguished by a corresponding high-accident frequency, were excluded from this study and formed break point between sections);
5. No major intersections within section length;
6. Relatively constant traffic volume throughout section; and
7. Relatively constant accident history throughout section.

Initially, various roadway lengths (highway department control sections) were selected for suitability through consultation with district engineers or their representatives of the Louisiana Department of Highways. Twenty-five to 30 control sections were selected from each of the 9 highway districts.

ROADWAY DATA OBTAINED FROM CENTRAL OFFICE FILES

Historical records of each section were used to determine suitable beginning and ending mileposts for each roadway to be studied in the field. At the same time, file numbers for the most current roadway plans were determined. The mileposts and file numbers were recorded on specifically designed roadway characteristic forms along with certain other pertinent characteristic data, e.g., brief history of road, district number, and parish.

Once the location of each roadway's plans was known, the plans were withdrawn from the files and the information on them transferred to specially designed work sheets. These sheets provided for the recording of alignment and grade, roadside obstructions, and traffic conflict data.

Traffic classification and volume data were obtained and recorded from the records of the highway department. The values were averaged over the 5-year period from 1962 to 1966 for each section. Level of service calculations were made from the Highway Capacity Manual (13). Equations were designed for the calculation of traffic service volume for a level of service B and the corresponding ratio of the peak-hour traffic volume to the service volume. The results of these calculations were also entered on the data summary sheets.

Traffic accident information was obtained from the highway department on the numbers of fatalities, injuries, and total accidents, as well as accident breakdowns by light and weather conditions for each section. The location of each accident was given to the nearest tenth of a mile in each section.

FIELD STUDY PROCEDURES

Roadway data not obtainable from available records were gathered from field work. Each control section was thoroughly scrutinized in order to represent properly its characteristics through field measurement. Continuous and individual marginal obstructions, sources of traffic conflicts, e.g., driveways, shoulder width, lane width, cross slope, and other values were obtained and recorded on field measurement data sheets. The following procedures were used for field measurement.

Pertinent facts on each section were recorded on the Field Measurement Data Sheet (FMDS). An adjustable calibrated Stewart-Warner car odometer was used to determine to the nearest hundredth of a mile the actual length of each section. At the same time, the odometer was used to determine milepost values for reference purposes.

Along each section, readings of lane width, shoulder width, and cross slope were taken and recorded on the FMDS. Readings were taken at 2, 3, or sometimes 4 different section mileposts depending on the length of the section. Measurements were made only in tangent sections to avoid superelevated curves. The value for the cross slope was determined by touching one end of a 5-ft-long aluminum level to a point 1 ft from the roadway center stripe and perpendicular to it. The level bubbles were then centered. The vertical distance to the roadway surface at the other end of the level was then recorded. This same procedure was also followed for the adjacent lane.

Notebook size sheets with station alignment grids were designed and mimeographed for use in recording alignment and grade, roadside obstructions, and traffic conflict data. As each section was run in the field, the number of driveways and obstructions and their locations were recorded on grid sheets.

The different variables were denoted by a color code, e.g., red for any type of obstruction and green for all types of driveways. Whether or not a certain object could be classed as an obstruction was a somewhat qualitative question, decided in the field. A "reasonable man in a typical car" was the standard use to decide on the question of obstructions. All the area within approximately 50 ft of the pavement edge on each side

of the road was observed. A dangerous object or continuous hazard was classed as either a continuous or a marginal obstruction if it would be encountered by a reasonably good driver in an accident situation. An example of a continuous obstruction might be a long line of trees perhaps 10 ft from the roadway or a rather steep drop-off near the pavement edge. An example of a marginal or discrete obstruction would be a lone tree near the road.

The independent variables measured were traffic, width, cross slope, alignment, and roadside friction. The traffic variables were as follows:

1. Trucks—the percentage of the ADT that is made up of vehicles other than passenger cars and pickup trucks.
2. Traffic volume—24-hour, 2-way total traffic flow, commonly referred to as ADT.
3. Traffic volume ratio—the ratio of the peak hourly traffic volume (2-way) for a highway section to its corresponding service volume for level B operation as defined by the Highway Capacity Manual (13).

The width variables were as follows:

1. Lane width—the width in feet measured between the center of a 2-lane roadway and the edge of the pavement or traveled way.
2. Shoulder width—the width in feet between the edge of the pavement or traveled way and the edge of any shoulder, generally indicated by a change in slope between the shoulder and the side slope.

The cross-slope variable is the slope of the pavement or traveled way from the center of the roadway toward the shoulder measured in feet per foot of pavement width.

The alignment variables were as follows:

1. Horizontal alignment—the percentage of the length of a highway section that has a horizontal highway curvature in excess of 3 deg.
2. Vertical alignment—the percentage of the length of a highway section that has a highway gradient in excess of 3 percent.

The roadside friction variables were as follows:

1. Continuous obstruction—the percentage of the total length of a highway section that has some roadside feature or obstacle that runs for more than a few feet on either or both sides of the roadway. Such a feature would be a deep roadside ditch or steep side slope that presents an obstacle to a vehicle's safely leaving the roadway in an emergency at posted highway speeds.
2. Marginal obstructions—the total number of discrete objects on both sides, such as a driveway embankment, culvert, roadway culvert headwall, tree, or telephone pole, per mile of a highway section within the cleared right-of-way. This is not to be confused with the marginal obstruction within 6 ft of the pavement edge used in capacity analysis (13).
3. Traffic conflicts—the total number of traffic access points on both sides per mile of highway section. These access points include only minor road intersections (intersections with major roads were considered as break points between study sections) and principal access driveways to abutting property along highway section.

FINAL DATA PREPARATION PROCEDURES

From the summary sheets, data were transferred to code sheets for keypunching onto computer cards. Nearly all of the information previously mentioned was then recorded on summary sheets that condensed the data for analysis. Sections were then checked for high-accident frequency values. If, after carefully studying the locations of each of these values, we found that the accidents were obviously being caused by conditions other than roadway geometry and weather conditions, we separated the extreme values from the sections and formed new smaller subsections, if feasible. All new summary data were then prepared on new summary forms.

DATA ANALYSIS PROCEDURE

Levonian (20) emphasizes that mulivariate analysis not only provides for statistical control but also retains that reliability associated with the entire sample. An important consequence is the likelihood that the prediction of accidents will be enhanced when several predictors are used. An even more important consequence of multivariate analysis is the clarity with which results can be interpreted; this last consequence accrues from the fact that multivariate results can be presented in an integrated rather than piecemeal fashion.

The analysis consisted first of selecting and punching the pertinent data by using a computer program to select needed data from raw data file and then a multiple regression program to perform the regression analysis.

For each analysis, the independent variables used as input data and described earlier were the same. In addition to these variables, a number of indexing characteristics were used in order to segregate the data for more detailed analysis. These included the following:

1. Highway maintenance district,
2. Section number,
3. Shoulder type,
4. Traffic volume,
5. Bridges in section,
6. Service volume,
7. Fatal accidents,
8. Injury accidents,
9. Total accidents,
10. Day accidents,
11. Night accidents,
12. Accidents on wet surface, and
13. Accidents on dry surface.

The dependent variable (accidents) was grouped differently for each run. The groups, each per 100 million vehicle miles, were as follows:

1. Accidents,
2. Injuries,
3. Fatalities,
4. Accidents during day,
5. Accidents during night,
6. Accidents on wet roads, and
7. Accidents on dry roads.

It was of interest to investigate the main effects and the first order interactions (including the main effect squared terms) of the independent variables. The large number of highly significant cross products means not necessarily that one variable depends on the other but that elements of the roadway are generally built as compatible units. For example, very seldom would one suggest today that a roadway be built with a high degree of adequacy for surface width but a low degree of adequacy for sight distance.

SUITABILITY OF THE SAMPLE AND RANGE OF VALUES OBTAINED

In undertaking a project of the type and scope previously described, one needs to be particularly concerned with the adequacy of the sample to cover the full ranges of the variables involved. The following points seem appropriate as a result of this research.

1. The researchers thought that past studies attempted to analyze distributions that were too large and diverse to be meaningful in view of the scarcity of data. The sample for this study included 246 sections of rural roadway, varying in length from 1 to 17 miles, on which occurred more than 6,000 accidents during the term covered by the study. Locations of the study sections are shown in Figure 1. Excellent geographical coverage of the state was obtained.

2. A primary difference between this study and others "that demonstrated a low probability of answering specific questions" lay in the fact that the researchers attempted to determine primarily a heirarchy of accident-cause factors. (This hierarchy might also be useful in determining optimum use of highway construction funds.) The sample of highway sections used in this study experienced over 6,000 accidents during a 5-year period. Particular care was exercised to determine that no study section had been altered during the time covered by the study. In choosing the sample sections, we took care that the entire sample provided coverage of the full range of each variable.

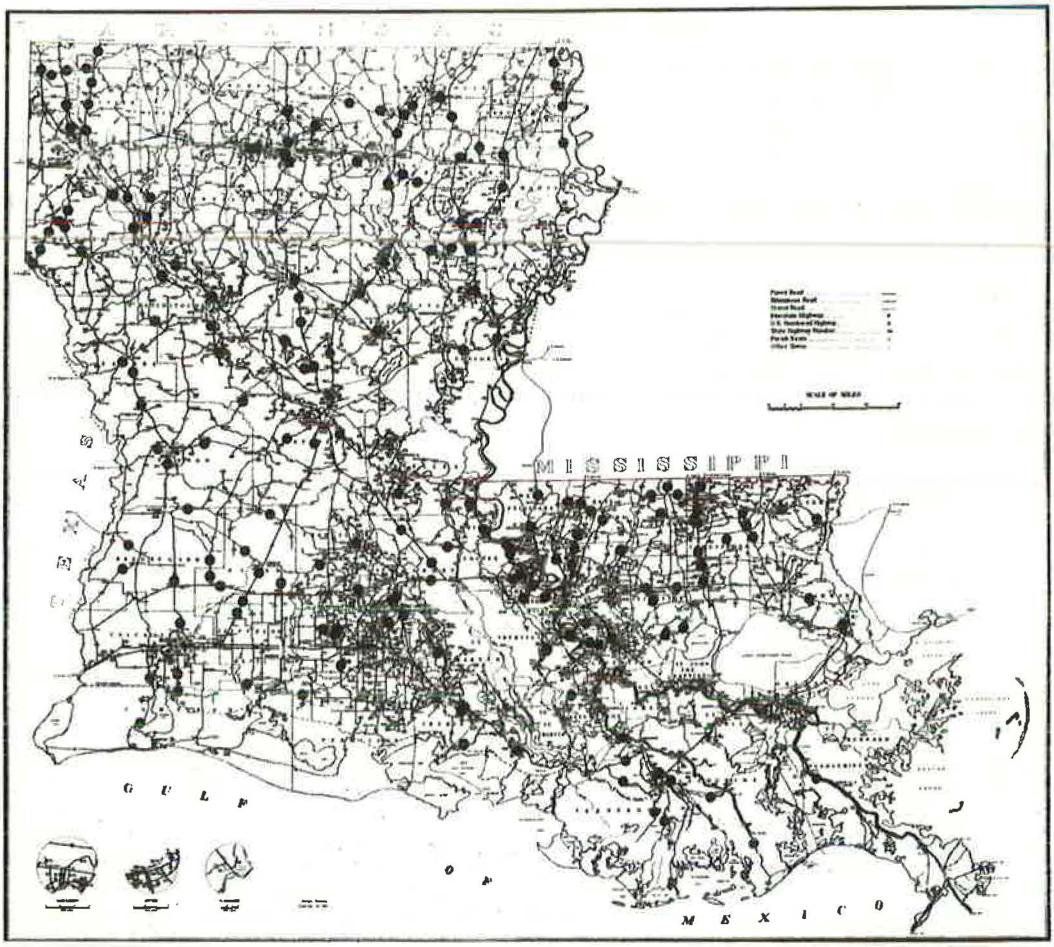

Figure 1. Location of study sections.

3. The manner in which accidents are investigated and recorded in Louisiana is entirely compatible with the requirements of the study. The accidents are reported to the nearest 0.1 mile and all the accident data required for the study were in the memory of the highway department's electronic data processing equipment. After the sections were selected, they were examined for a uniform distribution of accidents. Clustering of accidents was cause for eliminating or breaking the section up into smaller sections. No attempt was made to study only high-accident sections.

4. Every section of roadway used in this study was traveled twice for data collection and surveillance purposes. If a section was not homogeneous with respect to the variables, it was either deleted or divided into 2 sections, if feasible. The terrain of Louisiana is largely rolling and hilly in the north, flat in the south. This situation helped in obtaining sections that encompassed the wide range of variables necessary for the proposed analysis. Particular attention was paid to the sections for homogeneity; that is, if a section were hilly for any part, it must be uniformly hilly throughout. If a section had few traffic access points, it must have few over the entire length of the section.

Another word about the sample. Because all data were taken in Louisiana, then the results from the satistical manipulation of the data can only apply to Louisiana. It is

reasonable to expect that causes of accidents may be different in each state; therefore, comparisons with other states are interesting but there should not necessarily be a direct relationship.

The ranges of values for each of the independent variables studied on the 246 study sections are given in Table 2.

TABLE 2
VALUE RANGES FOR INDEPENDENT VARIABLES

Variable	Value Range
Traffic	
Trucks	2 to 33 percent of traffic
Traffic volume	190 to 11,933 vehicles per day
Traffic volume ratio	0.04 to 2.12
Width	
Lane width	9 to 12 ft
Shoulder width	1 to 12 ft
Cross slope	0.000 to 0.038 ft/ft
Alignment	
Horizontal alignment	0 to 57.5 percent of length > 3 deg
Vertical alignment	0 to 48.0 percent of length > 3 percent
Roadside friction	
Continuous obstructions	0 to 100 percent of section length
Marginal obstructions	0 to 34 per mile
Traffic conflicts	0 to 46 per mile

INDIVIDUAL VARIABLE RELATIONSHIPS TO ACCIDENT RATE

Each independent variable was plotted against total accident rate (accidents per million vehicle miles) to see what trends existed. Some of these relationships are shown in Figures 2 through 5. In each case the curves represent general trends of the raw data available.

Of particular interest are those that can be compared with previously reported studies. Figure 2 shows a trend typical of many previous studies (35, Ch. II) in which accident rates increase as traffic flow increases. More specifically, the curve shown in Figure 2 compares well with that in Rykken's Figure 6 (35) where a congestion index is used similar to the traffic volume ratio used here. The data shown in Figure 3 permit a direct comparison with those obtained by Charlesworth (34). The general aggreement of the data in Figures 2 and 3 with those previously reported by others indicates that the data obtained in this research are representative and quite usable for the intended purposes.

Of particular interest are the comparisons of accident rate with the 2 most important independent variables as shown by regression analyses. Figure 4 shows a very solid straight-

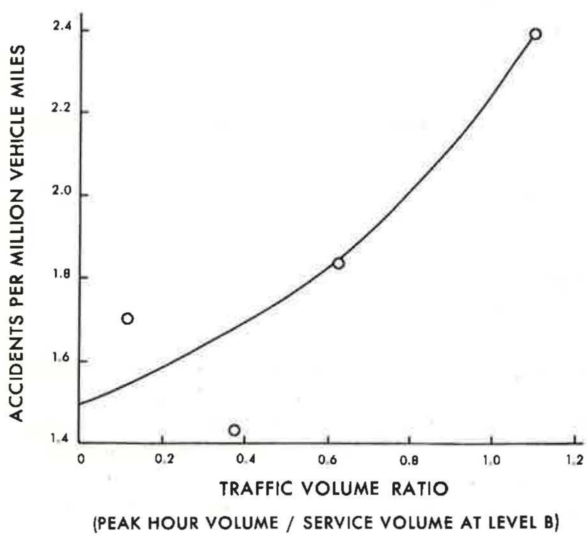

Figure 2. Accident rate versus traffic volume ratio.

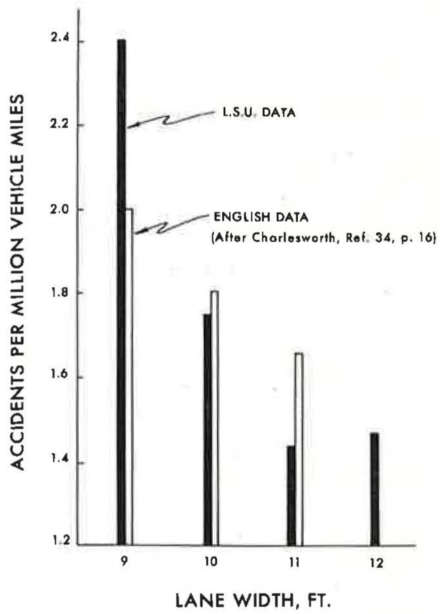

Figure 3. Accident rate versus lane width.

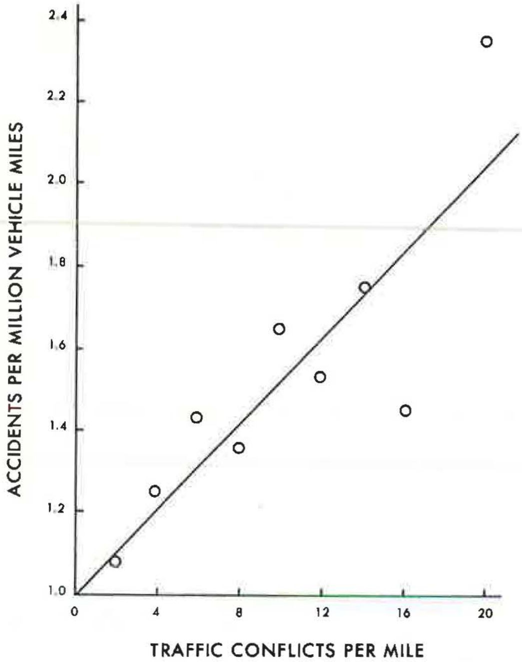

Figure 4. Accident rate versus traffic conflicts.

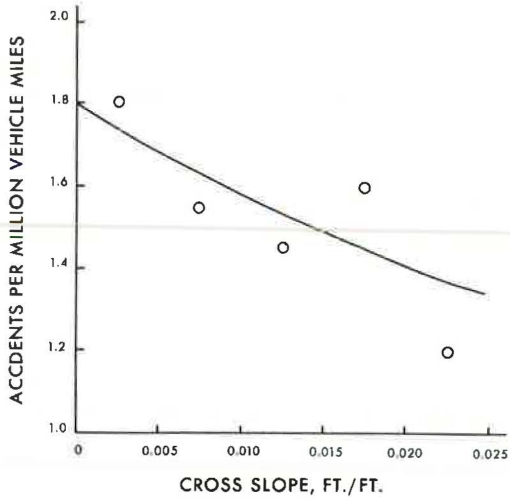

Figure 5. Accident rate versus pavement cross slope.

line trend of accident rate increasing with an increase of traffic conflict points per mile. This trend also correlates well with previously reported research, especially that of Mulinazzi and Michael (27). Figure 5, on the other hand, shows a relationship not previously reported and one that definitely indicates that roadways with relatively flat cross slopes are more accident prone than those with better slopes.

REGRESSION ANALYSES

The primary purpose of this study was to define those geometric characteristics of the roadway that were most responsible for accidents.

Total Accidents Analysis

The analysis of all 246 sections with respect to total accidents yielded the following hierarchy of importance (all first order interactions):

1. Traffic volume and pavement cross slope,
2. Traffic conflicts and traffic volume,
3. Lane width and traffic conflicts,
4. Traffic volume and horizontal alignment,
5. Shoulder width and horizontal alignment, and
6. Traffic volume and trucks.

The effects of traffic volumes, traffic conflicts, horizontal alignment, lane widths, and shoulder widths have been cited in previous research. However, it is important to note the hierarchy of their effects in the Louisiana evironment. Of particular interest in this analysis is the relative importance of cross slope. This is of particular significance in areas such as Louisiana because of the high rainfall (60 in. per year near Baton Rouge) that may lead to hydroplaning effects in the presence of low cross-slope values.

Other Analyses

The regression analyses of all sections with respect to accidents classed as injury, fatalities, wet surface, dry surface, day, and night yielded some differences in the hierarchy of variables. Table 3 gives the results of 8 regression analyses including both total accident analyses (all variables versus main effects only). Only the top 4 variables are given because they had the highest F-values for being entered into the regression analysis and also generally contributed to no less than about 1 percent of the cumulative R^2 value.

In all analyses it is significant that the one variable that contributed most to the accident rate was the traffic volume ratio. This indicates that the more nearly a roadway carries traffic volumes approaching or greater than its design service volume, the

TABLE 3

FOUR VARIABLES CONTRIBUTING MOST TO MULTIPLE R^2 VALUES

Accidents Analyzed	Variables	Variable Order	Multiple R^2
Total accidents: all first order variables versus main effects			
First order	TVR × CS	1	0.305
	TVR × TC	2	0.365
	LW × TC	3	0.386
	TVR × HA	4	0.398
	All		0.587
Main effects	TVR	1	0.241
	CS	2	0.265
	VA	3	0.272
	TC	4	0.278
	Ten		0.295
Fatalities versus injuries			
Fatalities	TVR × CS	1	0.136
	CS × TC	2	0.179
	HA × VA	3	0.199
	SW × CO	4	0.209
	All		0.424
Injuries	TVR × CS	1	0.304
	TVR × TC	2	0.358
	LW × TC	3	0.388
	TVR × HA	4	0.404
	All		0.610
Day versus night			
Day	TVR × CS	1	0.307
	TVR × TC	2	0.354
	T × TVR	3	0.374
	TVR × HA	4	0.384
	All		0.588
Night	$(TVR)^2$	1	0.258
	TVR × TC	2	0.300
	TVR × CS	3	0.327
	LW × TC	4	0.347
	All		0.537
Dry versus wet road			
Dry	$(TVR)^2$	1	0.303
	TVR × TC	2	0.352
	TVR × CS	3	0.378
	TVR	4	0.402
	All		0.615
Wet	TVR × CS	1	0.291
	T × TVR	2	0.331
	$(T)^2$	3	0.337
	TVR	4	0.352
	All		0.487

Note: CS cross slope; HA horizontal alignment; LW lane width; CO continuous obstructions; T trucks; SW shoulder width; TC traffic conflicts; TVR traffic volume ratio; VA vertical alignment

more likely it will experience a greater accident rate. The analyses also show that the most important geometric features affecting accident rates are pavement cross slope and traffic conflicts per mile.

Mathematical Model

Although mathematical models were prepared for all 7 analyses, the authors consider the analysis for total accidents to be the most important one. The mathematical model for this analysis is given in the Appendix.

In statistical terms, the product of correlation, X100, is defined as "the percent of variation in the data that is accounted for by the mathematical formula." Accordingly, the product of correlation, R^2, value obtained for the total accident model was 0.46, which indicates that the model explains 46 percent of the variation in the data. What does this really say? It says the 46 percent of the variation in the data is explained by the geometric factors that were included in the study. The remaining 54 percent is accounted for by variables not included in this study. Those are probably driver, environmental, and vehicle variables as well as other roadway variables not included and various interactions of all variables. The approximately 46 percent of the variation is in line with what authorities such as Baldwin (2) have suggested. In 1942 De Silva (37) said:

> The roadway may contribute directly to accidents, or an accident may result from a number of causes one of which may be traced to an inherent deficiency in the road. Accidents which are directly caused by "defects" in the roadway are believed to number from 3 to 10 percent of all accidents. If we include all accidents in which a "fault" may have been one of the contributing causes it is conceivable that 15 to 40 percent of accidents may be traceable directly or indirectly to the road.

More important than the model itself is its application and use as a possible tool for determining the probable accident rate for a proposed highway design. An example of the use of this model is given in the Appendix.

CONCLUSIONS

A criticism of the type of study undertaken here is that one obtains a lot of information that is intuitively obvious. That is, everyone knows that hills, curves, and a great deal of traffic will indicate high-accident experiences. This, of course, is true, and the fact that many of these factors do indeed come to the front comforts the researcher that his methodology is correct. How much do each of these factors contribute? This study has quantified the answer to this question. What about the occasional surprises? For instance, the relative importance of horizontal alignment over vertical alignment seems significant. The relative importance of cross slope seems to indicate that further study in this direction might prove fruitful. So the occasional surprises probably will more than justify the effort of the study.

This study is all-inclusive so that, of necessity, later studies must be concerned with specific design features that are here shown to be of importance.

Based on multiple regression analyses of data for Louisiana rural highways only, the following conclusions seem valid relating accidents to highway geometry:

1. Only 46 percent of the variation in accident rates is explained by the geometric factors that were included in this study. This indicates that 54 percent of the variation in accidents was caused by the driver, the vehicle, or by other variables that were not included in this study.
2. The methodology used and the results obtained appear to be valid based on limited comparisons with results of previously conducted research.
3. The 2 geometric variables appearing to have the most important effect on accident rates, based on their interactions with traffic volume, are pavement cross slope and the number of traffic conflicts per mile.

4. Of the geometric variables investigated, continuous obstructions, marginal obstructions, and their interaction terms had the least effect on accidents. This indicates that the variation in accidents was least caused by obstructions off of the right-of-way.

5. The project has been successful in relating several geometric variables to accident rates and has produced the following hierarchy of importance, all first-level interactions: traffic volume and pavement cross slope, traffic conflicts and traffic volume, lane width and traffic conflicts, traffic volume and horizontal alignment, shoulder width and horizontal alignment, and traffic volume and percentage of trucks.

6. A mathematical model has been developed that will help predict the accident picture likely to be experienced on a roadway, given the characteristics of that roadway.

RECOMMENDATIONS

Based on the results of this project, recommendations are as follows:

1. That the utilization of the mathematical models derived from this research be considered to determine how to allocate funds for highway accident reduction;
2. That particular attention be focused on the geometric variables of traffic conflicts and pavement cross slope in any highway improvement project;
3. That additional research be considered to investigate further the effect of pavement cross slope on vehicle performance and accident rate; and
4. That additional research be considered to ascertain ways to reduce accidents resulting from sources of traffic conflicts.

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Appendix

MATHEMATICAL MODEL

The following is the model for total accidents per 100 million vehicle miles. The R^2 is 0.46 and all of the F-ratios are significant at the 0.05 level.

$$y = 41.32 - 1.23X_1 - 0.54X_2 - 0.67X_6 + 0.03X_1X_2 + 0.03X_2X_6 - 0.0009X_2X_9$$
$$+ 0.026X_2X_{11} - 0.12X_4X_{11} + 0.009X_5X_9$$

where

y = total accidents per 100 million vehicle miles,
X_1 = percentage of trucks,
X_2 = traffic volume ratio,
X_6 = cross slope,
X_1X_2 = (percentage of trucks) (traffic volume ratio),
X_2X_6 = (traffic volume ratio) (cross slope),
X_2X_9 = (traffic volume ratio) (horizontal alignment),
X_2X_{11} = (traffic volume ratio) (traffic conflicts),
X_4X_{11} = (lane width) (traffic conflicts), and
X_5X_9 = (shoulder width) (horizontal alignment).

The mathematical models can be used to predict the number of accidents per 100 million vehicle miles, given the various geometric characteristics (independent variables) included in the model. The model for total accidents is used in this example.

The calculated value for the accident ratio can be obtained by using the equation for y in the mathematical model and substituting numbers in the correct units for the various X's. Average values were selected for the independent variables and were coded to be compatible with the computer program. They are as follows (their more familiar forms are in parentheses):

X_1 = percentage of trucks = 13,
X_2 = traffic = 55 (0.55),
X_4 = lane width = 11,
X_5 = shoulder width = 8,
X_6 = cross slope = 12.8 (0.0128 ft/ft),
X_7 = percentage of continuous obstructions = 66,
X_8 = marginal obstructions per mile = 7.5,
X_9 = horizontal alignment = 57 (57 percent of section > 3 deg),
X_{10} = vertical alignment = 50 (50 percent of section > 3 percent), and
X_{11} = traffic access points per mile = 9.

When these values are substituted in the equation for total accidents per 100 million vehicle miles, the following is obtained:

$$y = 28.65 \text{ accidents per 100 million vehicle miles}$$

With a given confidence value, the range of y can be determined. This equation is

$$Y' = y \pm t_{a,N} S_y$$

where

Y' = range of y,
$t_{a,N}$ = distribution that is a function of a and N,
a = confidence limits,
N = degrees of freedom, and
S_y = standard deviation of y.

The t value for the 50 percent confidence level and 224 degrees of freedom is 0.675. The standard deviation obtained from the MRP49 Multiple Regression Program was 25.78. Therefore, the range of y is

$$
\begin{aligned}
Y' &= y \pm t_{a,N} S_y \\
&= 28.65 \pm (0.675)\ (25.78) \\
&= 28.65 \pm 17.35
\end{aligned}
$$

Thus, 50 percent of the time y, the accident ratio, will lie between 11.30 and 46.00 accidents per 100 million vehicle miles. This is a large range, but it includes the accidents caused by geometric variables as well as by other factors. If it were possible to segregate those accidents caused by geometric factors alone, the product of correlation would increase, the standard deviation of the accident ratio would decrease, and the range in these calculations would be smaller.

The sign of the coefficient will enable the reader to determine how a change in magnitude of any term will affect the accident ratio. Terms with positive coefficient add to the accident ratio. A word of caution is appropriate concerning the magnitude of the coefficients. The term with the largest coefficient (absolute value) does not necessarily make the largest contribution in affecting the accident ratio.

The Relationship of Accidents to Length of Speed-Change Lanes and Weaving Areas on Interstate Highways

JULIE ANNA CIRILLO, Bureau of Public Roads, Federal Highway Administration, U.S. Department of Transportation.

This analysis was conducted on data collected for the Interstate System Accident Research, Study II. This study, which was initiated in 1961, has as its objective the determination of the relationship between the geometrics of the Interstate Highway System and its accident experience. The analysis reported here is concerned with the relationship between length of weaving areas, acceleration lanes, and deceleration lanes and the accident experience of these types of units. Also considered in the analysis was the number of vehicles utilizing these units and their relationship to the total number of vehicles using the Interstate. Results of this analysis indicate that increasing the length of weaving areas will reduce the accident rate and that increasing the length of acceleration lanes will reduce accident rates if the percentage of merging vehicles is greater than 6 percent of the mainline volume. Increased length of deceleration lanes will also reduce accident rates but to a lesser degree than the comparable increase in length of acceleration lanes. A smaller decrease in accident rate becomes apparent after the diverging traffic exceeds 6 percent of the mainline traffic. Only those variables specified in the report were considered, and therefore application of the results must be limited only to those variables.

•THE CONCEPTS OF ADEQUATE FACILITIES for handling traffic have evolved from many observations and analyses of traffic performance and driver behavior. As traffic volumes have increased over the years, the design of highway facilities has changed to accommodate this increase and to provide for the safe and efficient movement of people and goods. Initially roads were unpaved paths used by horses and horse-drawn vehicles. The advent of the automobile necessitated paving and making other improvements to provide safe, comfortable travel.

Among those improvements were additional lanes to provide better vehicular flow and barriers (either median, median barrier, or guardrail) to separate lanes of opposing traffic. However, vehicles still entered and exited all roadways by means of at-grade intersections. The parkway, designed in the late 1920's, was the forerunner of the freeway, i.e., a high-speed facility with access to and from the facility controlled by a grade-separated interchange. This drastic design change also introduced ramps to connect the crossing roadway. Initially these ramps were attached directly to the through traffic lanes, necessitating speed changes either on the main roadway or on the ramp. Subsequent research has shown (2, p. 73), that the variations in speed among the vehicles on the main roadway cause hazardous situations.

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In addition, ramps were neither designed for high speeds nor with sufficient length to allow a gradual speed transition. As the design of freeways improved and increased speed was possible, it became apparent that a modification to the existing design was necessary to aid the driver in executing the change in speed from the high-speed facility to the exit ramp or from the entrance ramp to the high-speed facility.

Thus the speed-change lane was developed and added to the end of the ramp adjacent to the main roadway to facilitate the speed changes. Many times these lanes were rather short, and some of the standards in use in the design of the Interstate System cause some questions as to the proper length of these units (1, Fig. VIII-18, p. 356). This is a report on an analysis of only one aspect of speed-change lanes: the effect of length on accidents. These lanes also have an operational aspect that, although related to safety, is not specifically considered in this analysis. This analysis was designed to determine the effect of providing additional length for both acceleration and deceleration lanes and for combined acceleration-deceleration lanes (weaving areas) on reducing the accident rate in merging, diverging, and weaving areas of an interchange.

Data used in this analysis were collected for the Interstate System Accident Research, Study II. This study, which was initiated in 1961, has as its objective the investigation of the relationship between the geometrics and the accident experience of the Interstate Highway System. Data were collected for this study by 24 state highway departments and were analyzed by the Bureau of Public Roads.

DATA BASE

Study sections, which are based at an interchange, were selected by each participating state. Each study section consisted of the interchange plus one-half the distance to

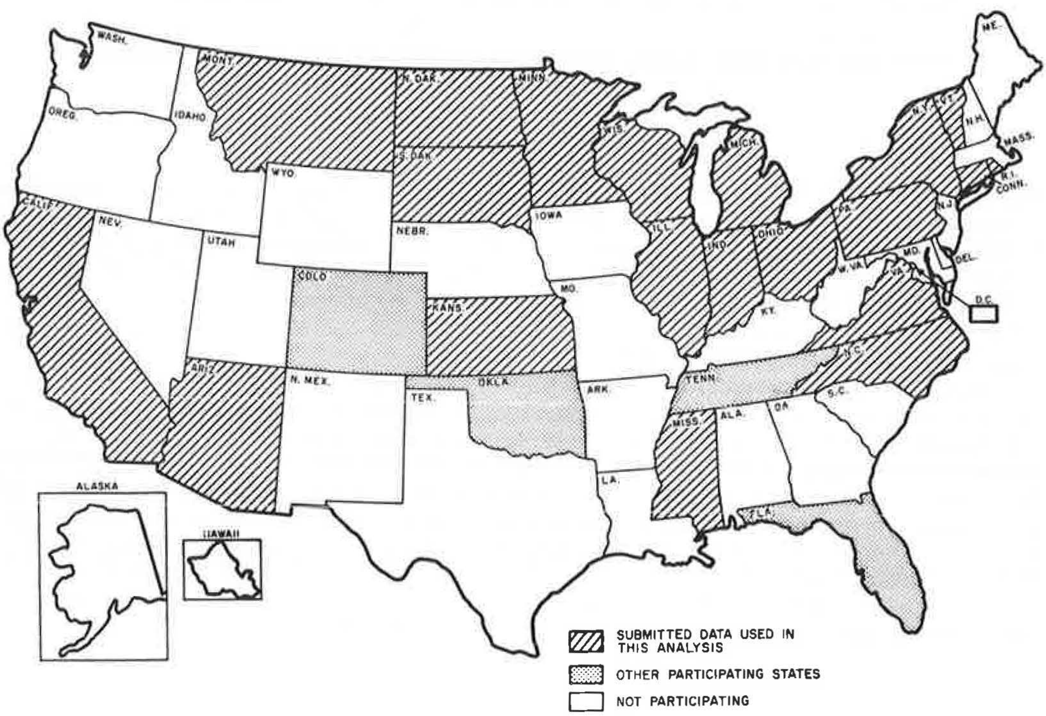


Figure 1. States participating in Interstate System Accident Research, Study II.

TABLE 1

NUMBER OF STUDY SECTIONS BY STATE AND YEAR

State	1959	1960	1961	1962	1963	1964	1965	Total	Percent
Ariz.	0	0	0	12	12	12	0	36	1.6
Calif.	0	0	0	49	58	57	0	164	7.2
Conn.	0	0	53	0[a]	53	0	0	106	4.6
Ill.	0	0	64	116	132	157	157	626	27.4
Ind.	0	1	28	37	40	41	0	147	3.4
Kan.	0	0	29	46	46	0	0	121	5.3
Mich.	0	0	0	65	0	0	0	65	2.8
Minn.	0	0	0	0	0	13	3	16	0.7
Miss.	0	0	0	0	0	6	6	12	0.5
Mont.	0	0	0	0	16	23	0	39	1.7
N.Y.	0	0	0	7	7	7	7	28	1.2
N.C.	34	40	59	74	124	128	127	586	25.6
N.D.	0	0	0	0	0	15	0	15	0.7
Ohio	0	3	10	9	10	11	0	43	1.9
Penn.	0	0	15	0	0	0	0	15	0.7
R.I.	0	0	4	4	4	6	8	26	1.1
S.D.	0	0	0	0	13	13	13	39	1.7
Vt.	0	0	0	6	0	0	0	6	0.3
Va.	0	2	11	14	27	48	48	150	6.5
Wisc.	0	16	16	16	0	0	0	48	2.1
Total	34	62	289	455	542	537	369	2,288	
Percent	1.4	2.6	12.6	19.8	23.6	23.4	16.1		

[a]The tape containing 1962 data for Connecticut was misplaced, and the omission was not discovered until after the analysis had commenced.

the preceding and succeeding interchange. Each study section was then divided into study units. Any portion of the study section that has a unique function or feature such as a ramp, speed-change lane, or underpass was defined as a study unit. Study units on the main roadway between interchanges that did not fit into one of these categories were determined according to the following definition: "...not more than 9,999 feet in length and homogeneous throughout with respect to its geometric characteristics" (3).

For each study unit, extensive information was collected on highway characteristics, traffic characteristics, and accidents that occurred on the unit. The 20 states whose highway departments submitted the data used in this analysis are shown in Figure 1. Data from the four remaining highway departments were not available at the time of analysis. The data used in this analysis represent over 9,000 year-miles of data. Mileage was computed for both directions of travel. In addition, no distinction was made among the different years of data. The chronological distribution of the data is given in Table 1.

Because of the rules devised for coding the study units, each ramp could be associated with the speed-change lane, either merging or diverging, to which it was connected. A distinction could also be made between merging and diverging study units. In this way, the effects of merging and diverging traffic as well as through traffic would be analyzed.

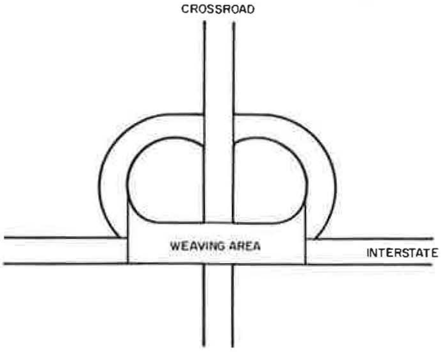


Figure 2. Weaving area.

ANALYSIS

The objective of this analysis was to determine if the length of weaving areas, acceleration lanes, and deceleration lanes had any effect on the accident experience of these types of units. Only length of speed-change lanes, specified traffic characteristics, and accident rate were considered. All traffic volumes reported are one-way, average daily traffic (ADT).

TABLE 2

ACCIDENT RATE BY LENGTH OF WEAVING AREA AND ONE-WAY TRAFFIC VOLUMES

Length of Weaving Area (ft)	One-Way Traffic Volumes (avg. no. vehicles per day)	Number of Units	Accident Rate[a]
400 to 449	<10,000	17	85
450 to 499	<10,000	9	105
	10,000 to 19,999	20	39
	20,000 to 29,999	18	525
500 to 549	<10,000	9	97
	10,000 to 19,999	23	241
	20,000 to 29,999	19	239
	30,000 to 39,999	11	206
	>40,000	20	446
550 to 599	<10,000	14	60
	10,000 to 19,999	31	195
	20,000 to 29,999	16	233
	30,000 to 39,999	21	337
	>40,000	26	328
600 to 649	<10,000	6	161
	10,000 to 19,999	33	94
	20,000 to 29,999	26	155
	30,000 to 39,999	20	230
	>40,000	15	300
650 to 699	<10,000	18	52
	10,000 to 19,999	27	82
	20,000 to 29,999	—	—
	30,000 to 39,999	20	154
	>40,000	17	245
700 to 749	<10,000	16	152
	10,000 to 19,999	38	49
	20,000 to 29,999	36	63
	30,000 to 39,999	25	128
	>40,000	16	209
750 to 799	<10,000	11	77
	10,000 to 19,999	31	48
	20,000 to 29,999	—	—
	30,000 to 39,999	22	128
	>40,000	15	228

[a]Number of accidents per 100 million merging and diverging vehicles.

Combined Acceleration-Deceleration Lanes—Weaving Areas

There were approximately 700 weaving areas in the data base utilized. (Weaving areas consisted of the acceleration-deceleration lane plus the adjacent main roadway. The length was measured as shown in Figure 2 and reported to the nearest 10 ft.) In this study, weaving areas only appeared on full cloverleaf and some types of partial cloverleaf interchanges. In the data base almost all weaving areas were classified as existing in urban areas. Thus, a stratification of the data by type of area was not possible.

When investigating the safety aspects of weaving areas, we thought it reasonable to assume that the number of conflicts or potential accidents has some relationship to the amount of merging and diverging traffic. This relationship was investigated and the results are given in Table 2. As expected, as the average daily traffic in one direction increases in a weaving area of a specific length, the accident rate exhibits a general upward trend. The information in Table 2 was also plotted and is shown in Figure 3. For one-way ADT greater than 10,000 vehicles, as the length of the weaving area increases, the accident rate decreases. For some volumes, this decrease in accident rate is quite severe; but for other categories, the reduction is of lesser magnitude. There appears to be no discernible trend for ADT under 10,000 vehicles, probably because the sample in this volume category was limited or because the length of weaving

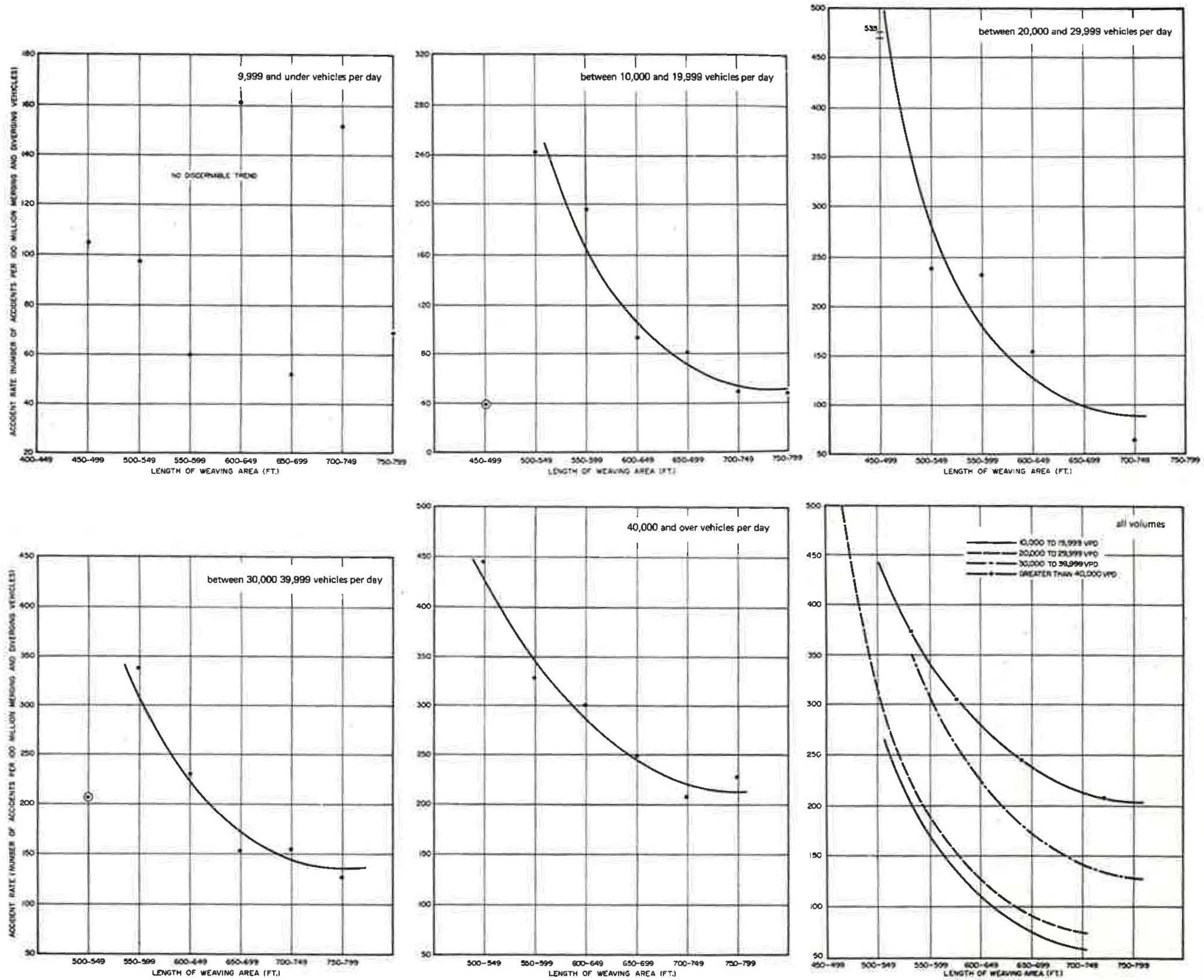


Figure 3. Accident rate by length of weaving area and one-way traffic volumes.

areas provided in areas of low volume is adequate. Thus it appears that the provision of longer weaving areas will effectively reduce the accident rate.

Acceleration and Deceleration Lanes

The arrangement of the data base allowed the investigation of individual acceleration and deceleration lanes to be conducted in a slightly different manner. (Acceleration or deceleration lanes are defined as the acceleration or deceleration lane plus the adjacent main roadway. The length of these units is measured as shown in Figure 4 and reported to the nearest 10 ft.) Of major interest here was the relationship of accident rate to the ratio of merging or diverging

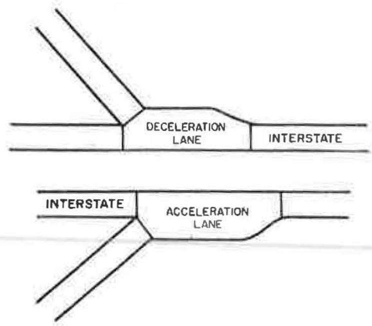


Figure 4. Acceleration and deceleration lanes.

TABLE 3

ACCIDENT RATE FOR ACCELERATION LANES OF VARIOUS LENGTHS

Length of Acceleration Lane (ft)	Percent Merging Traffic	Number of Units	Number of Accidents	Accident Rate[a]
<200	<2	27	14	81
	2.0 to 3.9	17	12	161
	4.0 to 5.9	21	14	157
	6.0 to 7.9	16	20	252
	8.0 to 9.9	18	33	350
	>10	27	30	448
200 to 299	<2	17	9	77
	2.0 to 3.9	16	11	98
	4.0 to 5.9	14	8	141
	6.0 to 7.9	19	24	170
	8.0 to 9.9	20	31	275
	>10	42	49	376
300 to 399	<2	15	10	70
	2.0 to 3.9	55	21	116
	4.0 to 5.9	30	17	141
	6.0 to 7.9	49	39	129
	8.0 to 9.9	35	67	200
	>10	70	121	292
400 to 499	<2	19	8	69
	2.0 to 3.9	25	31	103
	4.0 to 5.9	38	58	130
	6.0 to 7.9	37	58	178
	8.0 to 9.9	90	50	200
	>10	178	123	300
500 to 599	<2	5	4	68
	2.0 to 3.9	50	10	92
	4.0 to 5.9	82	35	118
	6.0 to 7.9	58	68	161
	8.0 to 9.9	116	76	199
	>10	205	99	250
600 to 699	<2	18	11	64
	2.0 to 3.9	13	8	91
	4.0 to 5.9	65	25	128
	6.0 to 7.9	102	62	100
	8.0 to 9.9	143	87	153
	>10	214	151	193
>700	<2	17	8	61
	2.0 to 3.9	12	6	90
	4.0 to 5.9	10	16	100
	6.0 to 7.9	212	118	123
	8.0 to 9.9	761	473	137
	>10	538	498	168

[a]Number of accidents per 100 million vehicles.

traffic to mainline volumes. This ratio was investigated in conjunction with the length of the individual speed-change lane.

The results of this analysis are given in Tables 3 and 4 and shown in Figure 5. As the percentage of merging or diverging traffic increases, the accident rate also increases regardless of the length of the speed-change lanes. The increase in accident rate, however, is substantially greater on acceleration lanes than on deceleration lanes. In addition, the shorter the length of the speed-change lane, either acceleration or deceleration lane, the higher the accident rate is regardless of the percentage of merging or diverging traffic.

The range of accident rates for the various lengths of acceleration and deceleration lanes is shown in Figure 6. There is little difference in accident rates for acceleration lanes of various lengths when the percentage of merging traffic is less than 6 percent of the mainline traffic. Thus when merging traffic is less than 6 percent of the mainline traffic, the savings in accidents for the additional length of speed-change lanes probably would not offset the cost of the additional length. However, the wide spread

TABLE 4

ACCIDENT RATE FOR DECELERATION LANES OF VARIOUS LENGTHS

Length of Deceleration Lane (ft)	Percent Diverging Traffic	Number of Units	Number of Accidents	Accident Rate[a]
<200	<2	21	10	62
	2.0 to 3.9	17	14	65
	4.0 to 5.9	13	17	119
	6.0 to 7.9	18	23	151
	8.0 to 9.9	10	15	196
	>10	27	43	259
200 to 299	<2	13	10	58
	2.0 to 3.9	19	19	69
	4.0 to 5.9	46	35	125
	6.0 to 7.9	28	56	140
	8.0 to 9.9	63	65	178
	>10	139	216	227
300 to 399	<2	30	16	39
	2.0 to 3.9	52	120	60
	4.0 to 5.9	41	28	123
	6.0 to 7.9	193	105	124
	8.0 to 9.9	33	30	172
	>10	217	56	200
400 to 499	<2	12	9	33
	2.0 to 3.9	38	24	60
	4.0 to 5.9	146	71	109
	6.0 to 7.9	97	40	129
	8.0 to 9.9	122	47	151
	>10	231	190	176
500 to 599	<2	16	7	29
	2.0 to 3.9	20	8	58
	4.0 to 5.9	30	20	127
	6.0 to 7.9	42	23	105
	8.0 to 9.9	348	190	129
	>10	230	171	200
600 to 699	<2	14	8	25
	2.0 to 3.9	41	16	41
	4.0 to 5.9	100	18	88
	6.0 to 7.9	57	31	120
	8.0 to 9.9	47	41	118
	>10	158	78	149
>700	<2	17	6	39
	2.0 to 3.9	11	5	48
	4.0 to 5.9	37	12	79
	6.0 to 7.9	36	18	111
	8.0 to 9.9	44	31	112
	>10	177	166	148

[a]Number of accidents per 100 million vehicles.

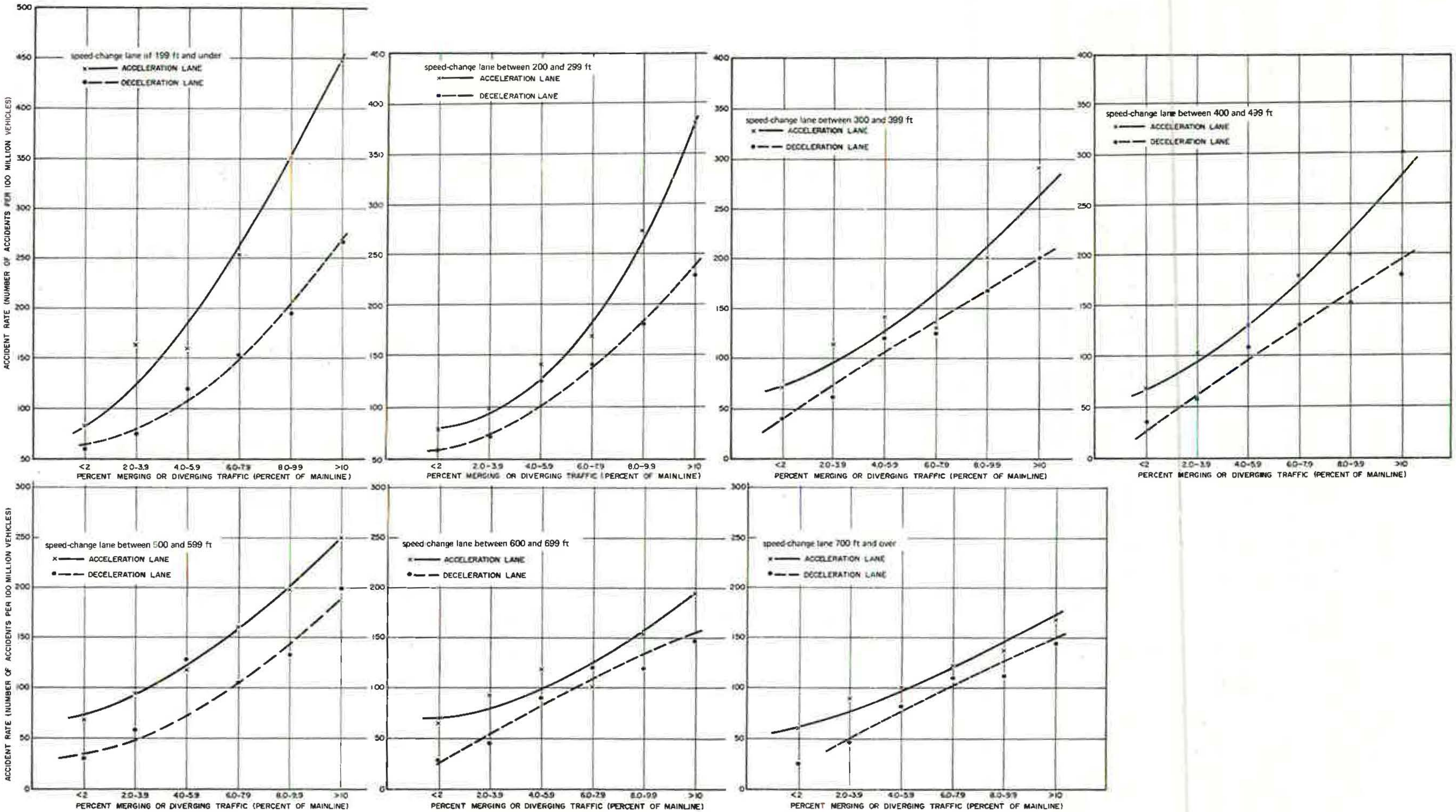


Figure 5. Accident rate by percentage of merging or diverging traffic and length of speed-change lanes.

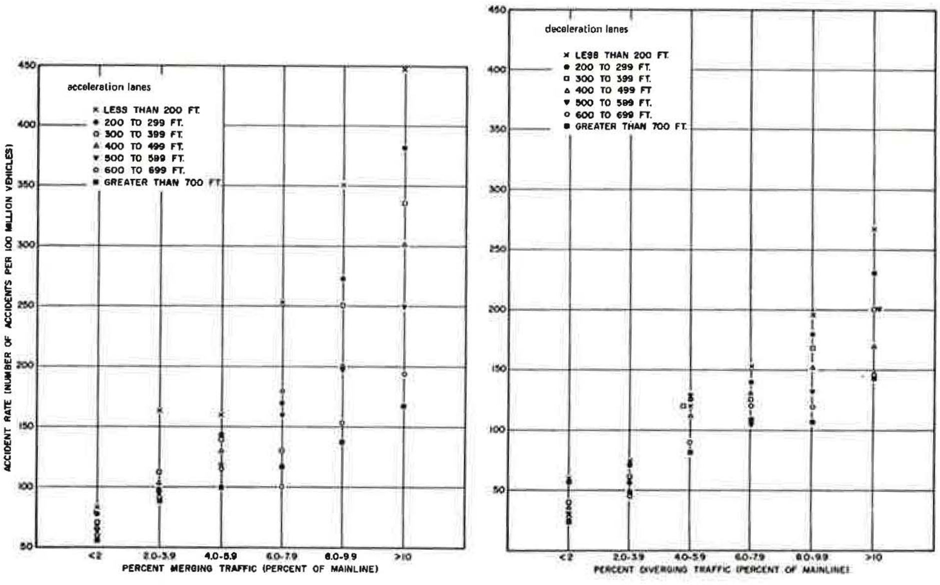


Figure 6. Accident rate by percentage of merging or diverging traffic and acceleration and deceleration lanes of various lengths.

of points for lanes between 6 and 7.9, between 8 and 9.9, and greater than 10 percent indicates that, as the percentage of merging traffic increases beyond 6 percent, the additional length on acceleration lanes provides a significant savings in accidents. In addition, increased length on acceleration lanes should improve the operation of the facility if properly used by vehicle drivers.

Relatively little benefit is obtained from additional length of deceleration lanes when the percentage of diverging traffic is less than 6 percent of the mainline volume. When the percentage of diverging traffic is over 6 percent, the length of the speed-change lane does provide some benefit in terms of decreased accident rate; but this decrease is not of the same magnitude as was apparent on acceleration lanes as the closeness of the points indicates.

The indication here is that additional length on speed-change lanes will be beneficial in terms of reduced accident rates if the percentage of merging or diverging traffic is above 6 percent. Increased length on acceleration lanes appears to be more beneficial than increased length on deceleration lanes, but deceleration lanes are, in general, safer than acceleration lanes regardless of the length of the lane and the percentage of merging or diverging traffic.

CONCLUSIONS

Results of this analysis indicate that increasing the length of weaving areas will decrease the accident rate. Accident rates on individual speed-change lanes exhibited a similar relationship with length. The effect of increasing the length of acceleration lanes appears to be substantial when the percentage of merging traffic is above 6 percent. The effect of increasing the length of deceleration lanes is not as great. Thus the relative benefit (i.e., savings in terms of lower accident rates) is greater from longer acceleration lanes than from longer deceleration lanes. For a given length of acceleration or deceleration lane, the accident rate increases respectively as the percentage of merging or diverging traffic increases.

REFERENCES

1. A Policy on Geometric Design of Rural Highways. American Association of State Highway Officials, Washington, D.C., 1965.
2. Cirillo, J. A. Interstate System Accident Research Study II, Interim Report II. Public Roads, Vol. 35, No. 3, Aug. 1968, pp. 71-74.
3. Interstate System Accident Research Revised Procedure Manual. Traffic Systems Division, U.S. Bureau of Public Roads, Aug. 1961.

Discussion

W. R. BELLIS, Manasquan, N.J.—This paper is on a very important subject and helps to validate experienced judgment. Similar analysis should be undertaken for all design features. It is often said that good highways are safe highways, and there is no doubt about this; but what constitutes a good highway? Each generation prides itself with building good highways, and they justify expenditures (which far outstrip the cost of the old) on promised reductions in accidents, injuries, and fatalities.

Road design of today is only better than that of yesterday when we can differentiate conclusively between the good and the bad as proven by the severe test of public use. The overall problem is very complex. We make improvements here and there within our ability, but after due reflection we note that the accidents, injuries, and fatalities have continued to increase unchecked no matter how we analyze the accident experience. Yes, we can draw attention to a few design features or even a few sections of modern highways that are relatively safer than selected old samples. This we do repeatedly; but it is also possible to select modern designs that have a record much worse than certain selected old designs. This we do not do. The cases where we can demonstrate improvement must be very insignificant compared to the overall picture; otherwise, their impact would be recognizable in the overall record or the improvement at one location has only aggravated the condition at another location.

Let us assume that a section of road or a system of roads is divided into 2 equal parts AB and BC; the overall accident rate for AC is 1,000 accidents per 100 million vehicle miles and the rate for AB and BC each is also 1,000. The BC half is modernized, and the expectation is that the accident rate will be reduced by 30 percent from 1,000 to 700. After the modernization, the accident rate for AC is 1,200 or an increase of 20 percent. Although the expected accident reduction did materialize on BC, in order to have a rate of 1,200 on AC, the rate on AB increased 70 percent to 1,700 after the modernization of BC.

This is an explanation of what is going on in every state in the country. We are making improvements within our ability, but we do not have the ability to reduce the accident rate; instead we are witnessing a continuing increase in the rate. The conclusion can only be that we must not be doing the right thing.

The definition of terms is very often a problem for the reader. Cirillo states "the effects of merging and diverging as well as through traffic could be analyzed." She also states "the number of conflicts or potential accidents has some relationship to the amount of merging and diverging traffic." And then, "as the average daily traffic increases...the accident rate exhibits a general upward trend."

Therefore, by deduction, merging traffic is that traffic that enters the main roadway; diverging traffic is that traffic that leaves the main roadway; and the through traffic is that traffic that stays on the main roadway. For example, assume that 30,000 vehicles approach a weaving area, 10,000 enter the main roadway from a ramp or side road, and then downstream from this point 8,000 leave the main roadway to enter a ramp or side road. Cirillo apparently calls the 10,000 cars "merging" and the 8,000 cars "diverging," and by her definition of accident rate "traffic volume" or "ADT" means the average daily merging or diverging traffic and does not include the through

traffic. In the discussion of speed-change lanes, it is not clear what is meant by "mainline." In the preceding example, is 30,000 or 40,000 the mainline traffic for the merging condition, and is 40,000 or 32,000 the mainline traffic for the diverging condition?

I would prefer that merging or diverging traffic meant the sum of the 2 approaching traffic streams or the sum of the 2 separating traffic streams. In my example, there would be 40,000 merging cars. Cirillo states that, for a weaving area of specific length, the accident rate increases as the average daily traffic entering and leaving the main roadway increases. In the preceding example, if the 10,000 vehicles should become 15,000, the 8,000 should become 12,000 (a 50 percent increase), and the 30,000 should become 12,000, I am sure that the accident rate as defined would decrease. If the 10,000 and the 8,000 should remain constant and the 30,000 should increase to 60,000, I believe the accident rate as defined would increase. I am sure that the accident rate is a function of the through traffic as well as the turning traffic. If the analysis were made similarly but the volumes of through traffic were held as constants, the general conclusions would be the same but much more valid.

I am very surprised at the large volume categories for the traffic entering and leaving the weaving areas from and to the ramps or side roads.

It should be made clear which accidents are included. Are all accidents in the weaving area and the immediate approaches to and departures from the weaving areas included? Or are only those accidents included that are interpreted as involving only vehicles making either an entering or a leaving movement?

Cirillo states that, as the percentage of merging or diverging traffic increases, the accident rate increases. Assume that there is a merging condition where 10,000 cars on the main roadway are joined by 800 cars from the side. This is 8 or 7.4 percent merging traffic (as defined) depending on whether the mainline traffic is 10,000 or 10,800. Now, if the 10,000 should reduce to 8,000 and the 800 should not change, the percentage of merging traffic would be 10 or 9.1. I fail to see how there would be an increase in the accident rate, especially as the accident rate is defined. Also, if the 10,000 should increase and the 800 remain constant, the percentage of vehicles merging would decrease; therefore, according to Cirillo, the accident rate would decrease. Certainly the number of accidents would increase and, because the number of merging vehicles stays constant, the rate would increase.

If the 10,000 is taken as mainline traffic and the 800 increases, then the percentage of merging traffic increases and the paper shows that the accident rate increases. In this case, however, the accident rate increases as the number of merging (as defined) vehicles increases. Then why not use number of vehicles instead of converting to a percentage? The same comments apply for diverging (decelerating) speed-change lanes as for the merging (accelerating) speed-change lanes.

Cirillo states that the longer the acceleration or deceleration lane, the lower the accident rate will be. This is a very important observation. I think such curves would be very helpful if illustrated. She also finds that increasing the length of acceleration lanes is more beneficial than increasing the length of deceleration lanes. To me, increasing the length of both is very beneficial. From her plottings I deduce that, with the 8 to 9.9 percent category, the accident rate is 74 percent greater for merging and 46 percent greater for diverging on a speed-change lane from 200 to 299 feet than on a speed-change lane from 600 to 699 feet. Both are of sufficient benefit to justify additional lengths.

I hope that Cirillo and others having extensive data available to them will continue and even accelerate such studies and make the results available to others so that ultimately conclusive evidence is available to justify the cost of improvements that will really reduce the accident rate. Apparently such improvements will be radically different from those made by current practices.

S. R. BYINGTON, Bureau of Public Roads, Federal Highway Administration, U.S. Department of Transportation—It is frequently observed by research personnel and administrators that the "true" value of a piece of research work may not be known until a long time, perhaps years, after the work is completed. But, the "true" value of a piece of research is never totally known for as long as there are people; and as long as people are characterized by change, the value of things, including research, will be subject to change. Still, value being person-space-time related, there is always the need to know how scientific output can be immediately applied by those on the operational side of a profession. Accordingly, the following discussion is devoted to the possible application of Cirillo's research results.

Based on the independent variables examined—speed-change lane lengths and ramp and mainline volumes—there are potentially 2 possible applications of the research results: (a) establishment of refined design criteria for minimum speed-change lane lengths, and (b) establishment of gross safety guidelines that should be considered in the ramp metering of traffic.

Only the first potential application is studied here with respect to deceleration, acceleration, and weaving area speed-change lanes. The examination of these geometric elements includes an abbreviated review of existing design criteria and their comparison with the findings in the reviewed paper. The second potential application is not studied because of the apparent need for more refined analysis of the existing data base.

Deceleration Lanes

Currently, desirable lengths of deceleration lanes are determined as a function of the following 3 factors: "(a) the speed at which drivers maneuver onto the auxiliary lane; (b) the speed at which drivers turn after traversing the deceleration lane; and (c) the manner of decelerating or the deceleration factor" (1, p. 348). From these assumptions, desirable lengths of deceleration lanes are calculated for various combinations of highway and exit curve (ramp) design speeds. The calculated lengths for those design speed combinations prevalent in Cirillo's data base and kindly supplied by her on request are given in Table 5 (1, p. 351).

TABLE 5

RECOMMENDED LENGTHS FOR DECELERATION LANES

Location	Highway Design Speed (mph)	Exit Curve Design Speed (mph)	Deceleration Lane Length	Comments
Rural	70	25	550	Sixty-five percent of Cirillo's data base involved deceleration lanes in rural areas for which the mainline design speeds were all 70 mph or greater. Exit ramps (assumed to be predominantly diamond) had design speeds of 30 mph or greater in 90 percent of the cases.
	70	35	525	
	80	25	650	
	80	30	600	
Urban	50	25	375	Thirty-five percent of Cirillo's data base involved deceleration lanes in urban areas for which the mainline design speeds were about evenly split between 50 and 60 mph. Exit ramps (assumed to consist of loops, outer connections, and the like) had design speeds of 20 and 25 mph in 80 to 90 percent of the cases depending on the exact type of ramp.
	50	25	350	
	60	20	475	
	60	25	450	

Figure 6 offers a comparison with the deceleration lane lengths given in Table 5 and demonstrates quite clearly the safety advantages of a minimum 600 to 700 ft deceleration lane length. Figure 6 shows that, regardless of the percentage of diverging traffic except for one grouping, the lowest accident rate occurred on deceleration lanes of 600 ft or greater. Even for the one exception, when the percentage of diverging traffic was 6.0 to 7.9 percent, the second lowest accident rate occurred when the deceleration lane length was 700 ft or greater. Such results indicate that the existing recommended minimum deceleration lane lengths are too low.

However, before such a change is recommended, the study's data should be reanalyzed with rural and urban deceleration lanes separated. Such an analysis might reveal different optimum lengths for rural and urban deceleration lanes. If the sample size permits, an even finer analysis should be made that would consider the effect of varying combinations of mainline and ramp design speeds on accident rates. The results could then be directly compared with the existing design standards and, when combined with a cost analysis, used in determining how or if the existing standards should be modified. The importance of such an analysis is emphasized by the large percentage, 65 percent, of short deceleration lanes, 500 ft or less, found in the study's sample of deceleration lanes.

Other refinements might include grouping by geometric type of ramp, such as diamond or clover, or total length of ramp because portions of the ramp itself can function as an acceleration or deceleration lane.

Acceleration Lanes

Factors similar to those employed for determining deceleration lane lengths are used presently in establishing minimum acceleration lane lengths. If design speed combinations similar to those in Table 5 are used, the recommended lengths for rural and urban areas are respectively in the ranges of 1,300 to 1,400 ft and 600 to 1,000 ft (1, p.351).

Figure 6 shows that, regardless of percentage of merging traffic, the optimum lane length is 700 ft or greater. Thus, there is fairly good agreement in desirable lane lengths between existing standards and that supported by accident analysis. The one question remaining is whether additional refinement of the present study data, in terms of acceleration lane length groupings, would have shown increased accident savings as the lane length increased. As shown in Figure 6, this may be particularly important where the percentage of merging traffic exceeds 6 percent.

Weaving Areas

Weaving area length, weaving volume, and quality of flow are the basic variables incorporated in the weaving volume determination chart presented in the Highway Capacity Manual (4, p. 166). Thus, to determine the recommended length of weaving section requires knowing both the total weaving traffic and quality of flow. Because this information was not available in the reviewed study, no comparison could be made between existing design standards and the results obtained by Cirillo. Nevertheless, the results are still of interest, for Figure 3, all volumes, shows that, regardless of the weaving section traffic volume, the accident rate continually decreases as the weaving area length increases up to a length of around 700 to 750 ft. Consideration of accident and construction costs might then dictate that the length of a combined acceleration-deceleration lane be 700 ft. Additional increases in the length of such lanes is questionable because the accident rate levels off at this distance.

Reference

4. Highway Capacity Manual—1965. HRB Spec. Rept. 87, 1965.

JOHN A. DEACON, University of Kentucky—In this study of accident rates at speed-change lanes and weaving areas, Cirillo has limited her evaluation to 2 independent variables, namely, length of lane or area and traffic volumes. Some of the unexplained variability in the reported data is doubtlessly due to the omission of other significant independent variables. However, some portion of the unexplained variability is probably due to an improper selection of the pertinent volume variables.

Consider for illustrative purposes the case of merging traffic at an on-ramp. It is hypothesized that the accident rate is a function of (a) the probability of conflict between merging and through vehicles, (b) the ability of drivers to detect potential conflicts, and (c) the ability and willingness of drivers to make adjustments necessary to avoid such conflicts. The probability of conflict is in turn a function of factors such as relative speeds and the volumes in each of the merging and through lanes. The drivers' ability to detect potential conflicts is a function of factors such as sight distance and angle of merge. Finally, their ability and willingness to make required adjustments are functions of factors such as acceleration lane length, gradient, and vehicular accelerative characteristics.

For merging areas, Cirillo has identified the pertinent volume variable as the percentage of merging traffic. It is a simple matter to show, however, that the expected number of potential conflicts is a function of volume on the through facility as well as the percentage of merging traffic. Figure 7, which is based on certain simplifying assumptions concerning peak-hour fractions, directional and lane distributions, gap distribution and critical size, and the like, supports this contention. Might it not be reasonable to assume, therefore, that traffic volume on the through facility is equally as important in its effect on accident rate as percentage of merging traffic? Similar analyses would indicate the importance of extending the number of volume variables for both weaving areas and deceleration lanes as well as for acceleration lanes.

In order to evaluate the effect of length of speed-change lane on accident rates, a plot such as that shown in Figure 8 is preferred to Figure 6. Figure 8 is based on

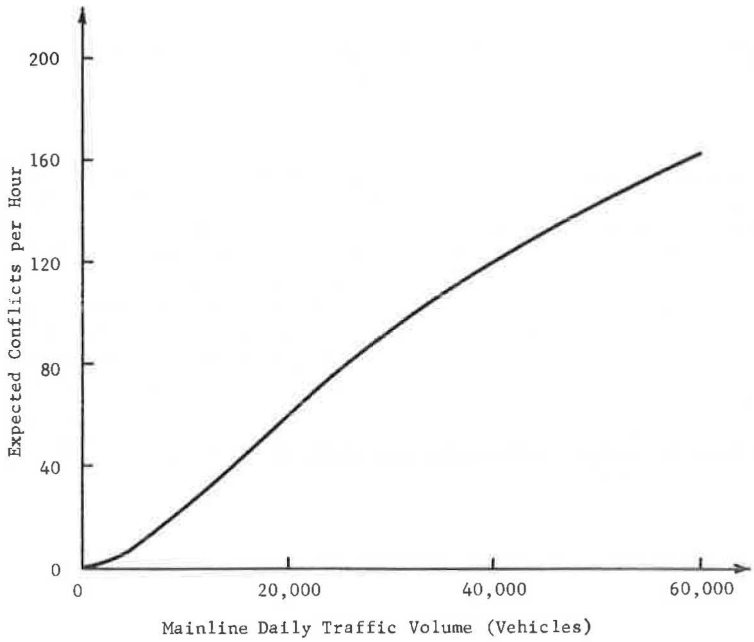


Figure 7. Merging conflicts for 7 percent merge volume.

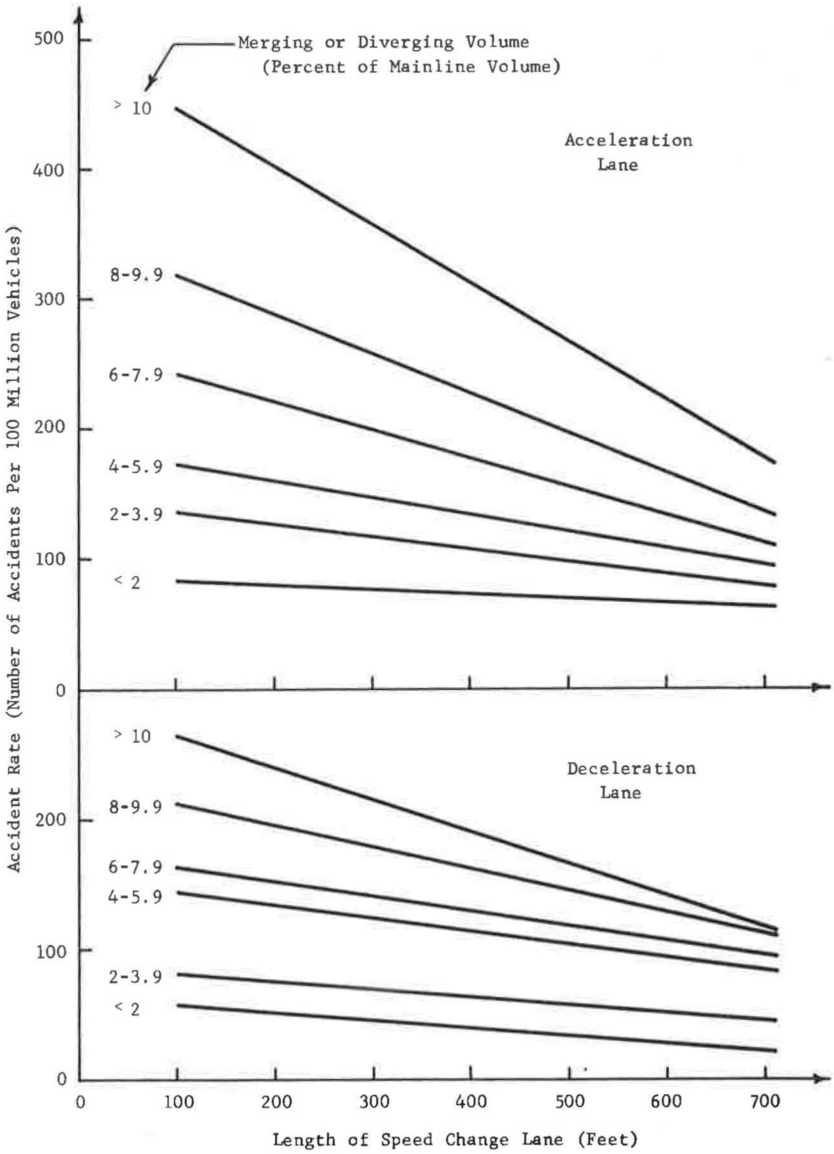


Figure 8. Effect of length of speed-change lanes on accident rates.

Cirillo's data as shown in Figures 3 and 5 with individual points being omitted for clarity. It shows that the reduction in accident rate (reduction in accidents per 100 million vehicles) for each 100-ft increase in length of speed-change lane varies from about 6 to 25 for deceleration lanes and from about 3 to 45 for acceleration lanes, depending on the percentage of merging or diverging traffic.

Cirillo implies that, when merging or diverging volumes are less than 6 percent of the mainline volumes, increased length of a speed-change lane is not justifiable from the viewpoint of accident reduction. Let the reader be cautioned to avoid considering this 6 percent level as invariant because (a) there is no abrupt change in accident rate at this level of merging or diverging, (b) an explicit economic analysis was not reported, and (c) factors other than safety are important.

The extensive accident data of this study have potentially significant implications for the proper design of speed-change lanes and weaving areas because safety together with capacity, speed, operational flexibility, cost, and service constitute fundamental design criteria. However, it should be noted that Cirillo has not chosen to fully examine the design implications of her findings. To enable the reader to do this for himself would require minimal additional information such as the number of study units in each of the various data categories as well as the corresponding number of accidents and traffic volumes.

A Safety Evaluation of Current Design Criteria for Stopping Sight Distance

JOHN C. GLENNON, Texas Transportation Institute, Texas A&M University

This paper presents a review of the current AASHO design standards and an evaluation of these standards based on existing practices. The evaluation considered the criteria that were employed in developing the standards and that include driver perception-reaction time, design friction factors, assumed speeds for design, driver's eye height, and object height. In addition, the report proposes a new philosophy for sight distance design, a philosophy that considers the visual requirements for safety depending on operational conditions.

•ABILITY TO SEE THE ROADWAY ahead is of the utmost importance in the safe and efficient operation of a highway. The path and speed of vehicles on the highway are subject to the control of drivers whose training is largely elementary. If safety is to be built into highways, the design must provide sight distance of sufficient length to permit drivers enough time and distance to control the path and speed of their vehicle, in order to avoid unforeseen collision circumstances.

There has been an increasing concern by highway and traffic engineers regarding the validity of the basic criteria that are fundamental to geometric design standards. The design standards for stopping sight distance employed by most state highway departments are taken from "A Policy on Geometric Design of Rural Highways" published by the American Association of State Highway Officials (1). The design criteria for stopping sight distance presented by AASHO are based on studies conducted between 1934 and 1953. As such, they may no longer be representative because vehicle, roadway, and driver characteristics have changed. In addition, there are uncertainties regarding the assumptions employed in establishing the safe stopping distance design standards.

This research study was addressed to an evaluation of the validity of the AASHO standards for safe stopping sight distance. The method of study employed a comprehensive review of current stopping sight distance standards and an evaluation of their validity, based on an analysis of existing practices.

STOPPING SIGHT DISTANCE DESIGN STANDARDS

A comprehensive description of the AASHO highway design standards on stopping sight distance is offered as a basis for an evaluation of their validity (1, pp. 134-140, 147-149).

Stopping Sight Distance Defined

The AASHO Policy defines sight distance as "the length of highway ahead visible to the driver." The minimum sight distances available should be sufficiently long to enable a vehicle traveling at or near the likely top speed to stop before reaching an object in its path. Although greater length is desirable, sight distance at every point along the highway should be at least that required for a below-average driver or vehicle to stop. Minimum stopping sight distance is the sum of 2 distances: (a) the distance traveled by the vehicle during the period of perception and brake reaction and (b) the distance required to brake the vehicle to a stop.

Paper sponsored by Committee on Operational Effects of Geometrics and presented at the 49th Annual Meeting.

Brake Reaction Time

Many studies have been conducted to determine the brake reaction time of drivers. These studies show that the brake reaction time for most people is from 0.5 to 0.7 sec (1). Some drivers react in a shorter time and some require a full second or more. One of the primary variables is age of the driver; as the driver becomes older, his reaction time becomes greater (2). The AASHO Policy states: "For safety, a reaction time that is sufficient for most operators, rather than for the average operator, should be used in any determination of minimum sight distance. A brake reaction time of a full second is assumed herein."

Perception Time

Perception time, as considered here, is "the time required for a driver to perceive the need for brake application." It is the time lapse from the instant an object is visible to the driver to the instant he realizes that the object is in his path and that a stop is required. Little is known about the exact time required for driver perception. It varies with the ability of the driver, his emotional and physical condition, and the visibility of the object. At high speeds, perception time may be less than at low speeds because the driver is more alert; however, the longer distances associated with higher speeds may require more time because of the degradation in visual acuity associated with higher speeds (3).

Perception-Reaction Time

Research data on perception time are very limited. Most available data combine perception time with brake reaction time. One study (4) conducted with alerted drivers determined an average combined value of 0.64 sec, with 5 percent of the drivers requiring over 1 sec. Under such conditions, perception time can be expected to be a small portion of the total perception-reaction time. The study concluded that the driver requiring 0.2 to 0.3 sec of perception time would require 1.5 sec for normal highway conditions. In another study (5) with alerted drivers, combined values ranged from 0.4 to 1.7 sec. Supplemental unpublished data in a study (6) of passing maneuvers showed that a perception time of approximately 1 sec was required for drivers to analyze and begin a passing maneuver.

The AASHO Policy considers the data from these studies in arriving at a value for perception-reaction time for use in stopping sight distance design. It states:

> A significant feature of these comparative tests is that the total perception and brake reaction time for highway conditions may be several times that for laboratory conditions, and it is evident that perception time is greater than brake reaction time. In determination of sight distance for design, the perception time value should be larger than the average for all drivers under normal conditions. It should be large enough to include the time taken by nearly all drivers under most highway conditions. For such use herein it is assumed that the perception time value is 15 seconds, and the total of perception and brake reaction time is 2.5 seconds. Available references do not justify distinction over the range in design speed.

Braking Distance

The approximate braking distance of a vehicle on a level roadway may be determined by using the standard formula

$$d = V^2/30f$$

where

d = braking distance, ft,
V = initial speed, mph, and
f = coefficient of friction between tires and roadway.

In this formula the coefficient of friction, f, is an equivalent constant value representing the entire speed-change interval from V to zero mph. Measurements show that f is not the same for all speeds (7). It decreases as initial speed increases. It varies

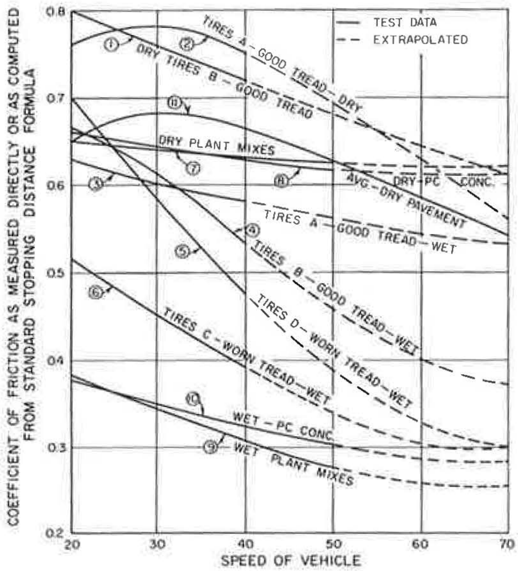


Figure 1. Relationship between friction factor and speed for several conditions.

because of several physical elements such as tire pressure, tire type, tire tread depth, type and condition of pavement, and the presence of water, snow, ice, or mud. These variables are accounted for if the coefficient of friction, f, is computed for each test from the standard formula $d = V^2/30f$. It thus represents the equivalent constant friction factor.

Design Friction Factors

In developing design values for the friction factor, f, AASHO considered the results of several investigators (5, 8, 9). Figure 1 shows the curves relating friction factor, f, to vehicle speed (1, Fig. III-1). For several of these curves, the friction factor, f, was calculated by using the standard formula because 2 of the investigators recorded speed and stopping distance only (5, 8).

Curves 1 to 6 in Figure 1 are from a study (8) in which more than 1,000 measurements of forward stopping distance were made on 32 pavements, both in wet and dry conditions. Several types of tires were used. Curves 7 and 8 are representative of several curves developed in a study (9) that recorded friction factors on 50 surfaces tested when dry by using 3 different methods and 3 types of tires. Curves 9 and 10 from the same study are representative of wet conditions. Curve 11 is the calculated equivalent friction factor for stopping distances, measured (5) on a new high-quality pavement; these were the only tests that included stops from 60 and 70 mph. This curve represents an average of all stops measured.

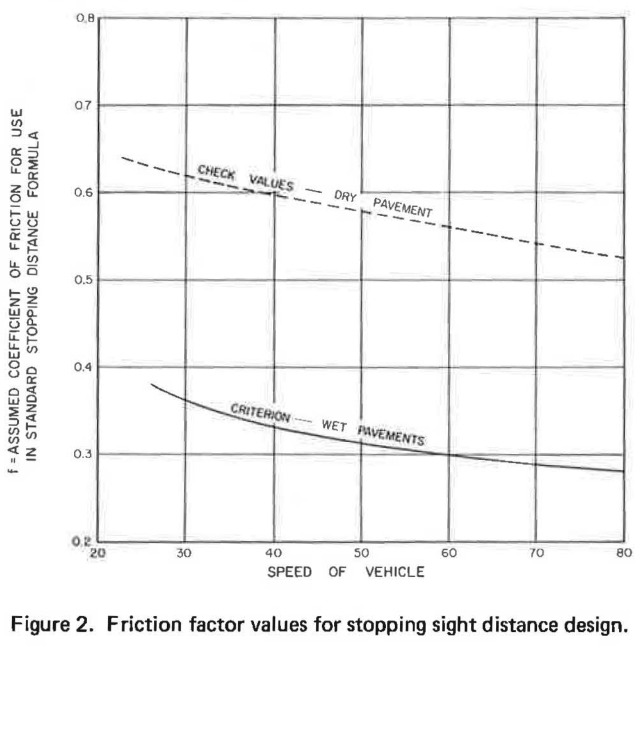


Figure 2. Friction factor values for stopping sight distance design.

The AASHO Policy in concluding its establishment of design values for the friction factor makes the following remarks:

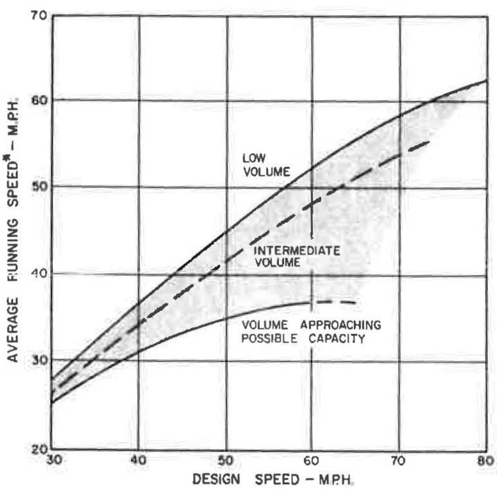
> Because of lower coefficients of friction on wet pavements as compared to dry, the wet condition governs in determining stopping distances for use in design. The coefficients of friction used for design criteria should not only represent wet pavements in good condition but also surfaces throughout their useful life. The values should encompass nearly all significant pavement surface types and the likely field conditions. They should be such as to be safe for worn tires, as well as for new tires, and for nearly all types of treads and tire composition. And, the friction factor should safely encompass the differences in vehicle and driver braking from different speeds. On the other hand, the values need not be so low as to be suitable for obsolescent or bleeding surfaces or for pavements under icy conditions. Preferably, the f values for design should be nearly all inclusive, rather than average; available data are not fully detailed over the range for all these variables, and conclusions must be made in terms of the safest reported average values. The lower curve in Figure III-1B gives the f values assumed for calculation of design stopping distances, recognizing these factors. Comparison with the curves of Figure III-1A shows them to be both practical and conservative.

Figure 2 shows the friction factor values for design referred to in this quotation (1, Fig. III-1B).

Assumed Speed for Conditions

The AASHO Policy states that it is not realistic to assume that travel will occur at full design speed when conditions are wet. It states:

> While the degree to which speeds are lower in inclement weather is not known precisely, it is definite that top speeds will be somewhat lower on wet pavements than on the same pavements in dry weather. For use herein the speed for wet conditions is considered to approximate 80 to 93 percent of design speed which, as previously explained, is indicative of the top speeds when pavements are dry. These speeds are the same as average running speeds for low volume conditions as shown in Figure II-16.



RUNNING SPEED IS THE SPEED (OF AN INDIVIDUAL VEHICLE) OVER A SPECIFIED SECTION OF HIGHWAY, BEING DIVIDED BY RUNNING TIME.

*AVERAGE RUNNING SPEED IS THE AVERAGE FOR ALL TRAFFIC OR COMPONENT OF TRAFFIC, BEING THE SUMMATION OF DISTANCES DIVIDED BY THE SUMMATION OF RUNNING TIMES. IT IS APPROXIMATELY EQUAL TO THE AVERAGE OF THE RUNNING SPEEDS OF ALL VEHICLES BEING CONSIDERED.

Figure 3. Relationship between average running speed and design speed.

Figure 3 shows the relationship between average running speed and design speed (1, Fig. II-16). The AASHO Policy refers to data collected to establish these curves but does not give a source, either published or unpublished. The relationship was supposedly established from data that related average spot speed to design speed on horizontal curves.

Minimum Stopping Sight Distance Design Values

The sum of the distance traveled during the perception and brake reaction time and the distance required to stop the vehicle, is the minimum stopping sight distance. Table 1 gives the AASHO Policy design values (and their bases of computation) for minimum stopping sight distance. Comparative check values for dry pavements are shown in the lower part of Table 1. These are computed by use of full design speed and f-values for dry pavements as shown in the upper part of Figure 2.

Effect of Grades on Stopping

When a highway is on a grade, the standard formula for braking distance is

$$d = \frac{V^2}{30\,(f + G)}$$

in which G is the percentage of grade divided by 100, and the other terms are as previously stated. Table 2 gives the extent of the grade corrections of the AASHO design values for stopping sight distance.

Criteria for Measuring Sight Distance

All the material presented previously deals with the design level required for adequate stopping sight distance based on driver and vehicle performance levels. To apply these minimum stopping sight distances in the design procedure requires geometric considerations of the highway alignment (both horizontal and vertical), the height of the

TABLE 1

DESIGN STANDARD FOR MINIMUM STOPPING SIGHT DISTANCE

Design Speed (mph)	Assumed Speed for Condition (mph)	Perception-Reaction		Coefficient of Friction (ft)	Braking Distance on Level (ft)	Stopping Sight Distance	
		Time (sec)	Distance (ft)			Computed (ft)	Rounded for Design (ft)
				Wet Pavements			
30	28	2.5	103	0.36	73	176	200
40	36	2.5	132	0.33	131	263	275
50	44	2.5	161	0.31	208	369	350
60	52	2.5	191	0.30	300	491	475
65	55	2.5	202	0.30	336	538	550
70	58	2.5	213	0.29	387	600	600
75[a]	61	2.5	224	0.28	443	667	675
80[a]	64	2.5	235	0.27	506	741	750
				Comparative Values—Dry Pavements			
30	30	2.5	110	0.62	48	158	
40	40	2.5	147	0.60	89	236	
50	50	2.5	183	0.58	144	327	
60	60	2.5	220	0.56	214	434	
65	65	2.5	238	0.56	251	489	
70	70	2.5	257	0.55	297	554	
75	75	2.5	275	0.54	347	622	
80	80	2.5	293	0.53	403	696	

[a]Design speeds of 75 and 80 mph are applicable only to highways with full control of access or where such control is planned in the future.

TABLE 2

EFFECT OF GRADE ON STOPPING SIGHT DISTANCE FOR WET CONDITIONS

Design Speed (mph)	Assumed Speed for Condition (mph)	Correction in Stopping Distance (ft)					
		Percent Decrease for Upgrades			Percent Increase for Downgrades		
		3	6	9	3	6	9
30	28	—	10	20	10	20	30
40	36	10	20	30	10	30	50
50	44	20	30	—	20	50	—
60	52	30	50	—	30	80	—
65	55	30	60	—	40	90	—
70	58	40	70	—	50	100	—
75[a]	61	50	80	—	60	120	—
80[a]	64	60	90	—	70	150	—

[a]Design speeds of 75 and 80 mph are applicable only to highways with full control of access or where such control is planned in the future.

driver's eye, and the height of the object. Sight distance along a highway is measured from the top of an object on the traveled way when it first comes into view. The general equations used in the design for stopping sight distance over crest vertical curves are

$$L = 2S - \frac{200\,(\sqrt{H_1} + \sqrt{H_2})^2}{A} \quad \text{for } S > L$$

and

$$L = \frac{AS^2}{200\,(\sqrt{H_1} + \sqrt{H_2})^2} \quad \text{for } S < L$$

where

L = length of vertical curve,
S = sight distance, ft,
A = algebraic difference in grade over the crest,
H_1 = height of driver's eye, ft, and
H_2 = height of object, ft.

The equation used in the design for stopping sight distance on horizontal highway curves is

$$S = \frac{R}{28.65} \cos^{-1} \frac{R - m}{R}$$

where

S = sight distance, ft,
R = radius of highway curve, ft, and
m = distance in between obstruction and the centerline of the inside lane.

For more than 20 years, the AASHO design policies (1, 10) based stopping sight distance criteria on a driver's eye height of 4.5 ft above the ground. In 1965, the AASHO Policy adopted a height of 3.75 ft. A study by Stonex (11) probably influenced this decision. Percentile distributions of "average" driver eye heights are shown for automobile models of various years in Figure 4. Median driver eye height has decreased from 56.5 in. in 1936 to 47.5 in. in 1960. Stonex surmised that average driver eye heights would not fall below about 42 in. because of the need for automobiles to conform with sight distance constraints of existing highways.

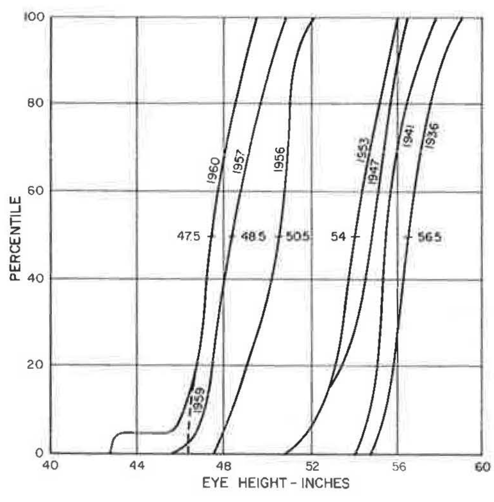


Figure 4. Percentile distribution of driver's eye height for automobile models of various years.

Minimum stopping sight distance is based on the distance required to stop safely from the instant a stationary object in the same lane becomes visible. On crest vertical curves, this point is limited by some point on the road surface. For horizontal curves, it is limited by a lateral obstruction beyond the roadway on the inside of the curve. The height of object that should be used to measure stopping sight distance on crest vertical curves has been a controversial subject (1,10).

The safest height of object would be zero (i.e., the surface of the roadway would be visible to the driver for the full length of the minimum stopping sight distance). Using this criterion, however, could result in long vertical curves requiring considerable excavation cost. The height should not be more than the approximate 2-ft height of vehicle taillights. On the other hand, this height would allow questionably short vertical curves, because lower objects on the road, such as small animals, merchandise dropped from a truck, or rocks rolled from a side cut, may have to be seen to avoid a collision.

Examination of the required lengths of vertical curves for the minimum stopping sight distance (AASHO design values) in conjunction with various heights of objects indicates a significant relationship. The AASHO Policy states:

> Plottings (not shown) of lengths of vertical curves with respect to height of object, for any one condition of sight distance and algebraic difference in grades, reveal that the required length of vertical curve diminishes very rapidly as the height of object is increased from 0 to about 6 inches; for greater object heights, the reduction in length of vertical curve is progressively less significant. Substantial economy in construction (as reflected in the depth and volume of excavation due to shortening of vertical curve) is effected by using a 6-inch object instead of the desirable zero value, yet the ability to see or appraise a hazardous situation is not materially altered. A height of 6 inches is assumed for measuring stopping sight distance on crest profiles.

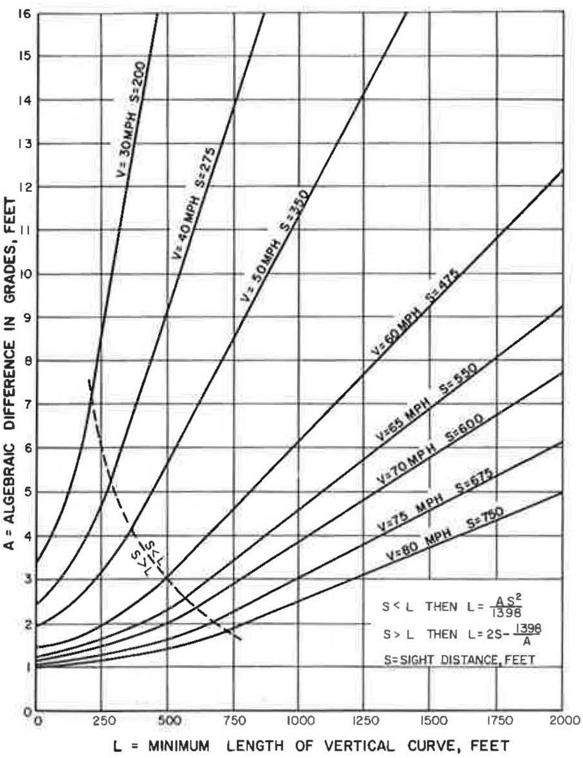


Figure 5. Relationship between stopping sight distance and length of crest vertical curve for eye height of 3.75 ft and object height of 0.5 ft.

These criteria for height of eye and height of object represent a departure from the 1954 AASHO Policy. For the 1954 Policy, the height of eye was assumed to be 4.5 ft. The same analysis as described earlier was used in the 1954 Policy to conclude that a 4-in. object height should be used in design. If 3.75 ft for the height of eye and 0.5 ft for height of object are used, the general equations for the length of vertical curve may be modified as follows:

$$L = 2S - \frac{1,398}{A} \quad \text{for } S > L$$

$$L = \frac{AS^2}{1,398} \quad \text{for } S < L$$

Figure 5 shows the relationship between length of vertical curve and sight distance for various algebraic differences in grade.

The height of object in determining stopping sight distance on horizontal highway curves is not as significant as that on vertical curves. Where the lateral sight obstruction is vertical, all heights of object may be seen at the same distance. Where the obstruction is an inclined cut slope, sight

distance is affected somewhat by the height of object, but the effect is not large. For consistency, the AASHO Policy uses the same height criteria on both horizontal and vertical curves, i.e., 3.75-ft eye height and 6-in. object height.

EVALUATION OF STOPPING SIGHT DISTANCE DESIGN STANDARDS

This discussion includes the evaluation of the validity of the following criteria employed in the AASHO stopping sight distance standards: perception and brake reaction times, braking distance equation, design values for the friction factor, assumed speeds for design, driver eye height, and object height.

Perception and Brake Reaction Time

The time interval of the stopping process, commonly called the perception-reaction time, is a very complex phenomenon. It is highly variable, dependent on the driver's psychological and physiological characteristics as well as the condition to be perceived. This may explain the lack of research to measure driver perception-reaction values in actual highway driving situations.

There are, however, conceptual explanations of the perception-reaction phenomenon. For example, Matson, Smith, and Hurd (12) describe the phenomenon as being composed of 4 elements: perception, intellection, emotion, and volition. Perception time is described as the time interval between the visibility of an object and the recognition of the object through visual sensation by the driver. The intellection time is that interval required for comparing, regrouping, and registering new sensations. Emotion is described as a time modifier of perception and intellection, dependent on the psychological makeup of the driver. Volition time is that interval necessary to exercise the decision to act. Another conceptual explanation of the perception-reaction process is offered by Baker and Stebbins (13) and is shown in Figure 6.

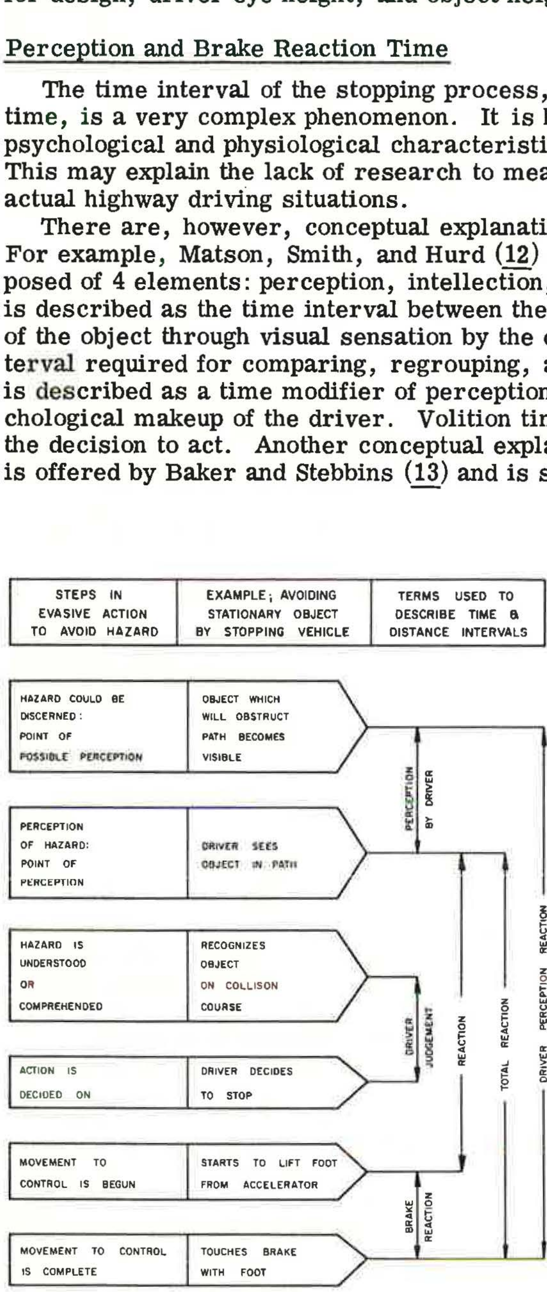


Figure 6. The perception-reaction process.

Matson, Smith, and Hurd (12) described many variables that affect perception and reaction time, including fatigue, physical disabilities, alcohol, drugs, climatic conditions, light conditions, and driver traits. Mullins (3) states that eye blinking occurs in intervals of 2.8 to 3.8 sec with a duration of 0.3 sec or more. He concludes that vision is unreliable for a short time before and after a blink and the modified blackout period caused by blinking may vary from 14 to 20 percent of all seeing time.

The most important element in the perception-reaction pheonomenon is perhaps the perception of form. Perception of form depends mainly on a sharp difference of brightness between an object and its background. Color difference is not as perceptible as brightness. Surfaces that differ in hue, but not in brightness, may be difficult to distinguish. In addition, highway design criteria assume that, as an object first comes into view on a crest vertical curve, the driver will perceive it and take appropriate action. Surely the driver has to see more than the first fraction of an object before he can perceive it. At 70 mph on a vertical

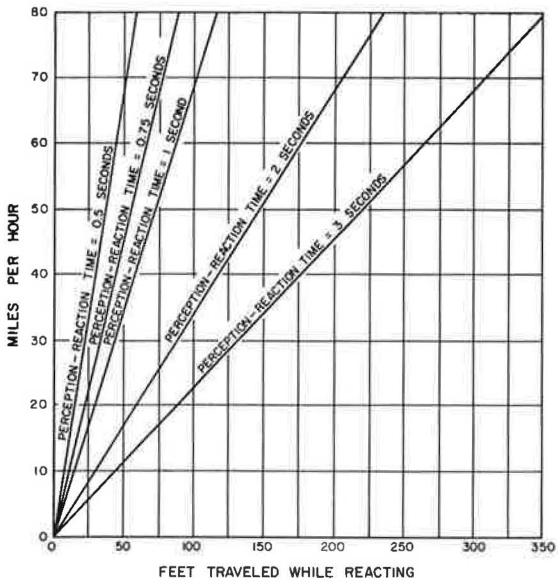


Figure 7. Distance traveled during various perception-reaction times for various speeds.

curve with a 600-ft stopping sight distance (i.e., the driver sees the top of a 6-in. object at 600 ft), the driver will not see the whole object until he has traveled 225 ft further.

The laboratory tests (4, 5) previously discussed indicated driver perception-reaction times in the range of 0.4 to 1.7 sec. As stated in the AASHO Policy, however, these times may be significantly greater for actual highway driving conditions. The AASHO Policy value of 2.5 sec, however, was hypothesized based on these studies. The perception-reaction time is highly variable, however, and, under critical conditions, could be higher than the 2.5-sec value.

Figure 7 shows how the distance traveled during perception-reaction time varies with speed. The required stopping sight distance would not be significantly changed for the lower speeds by increasing the required perception-reaction time by 1 sec. For the higher speeds, however, the distance is significantly increased for a 1-sec increase. Because of the trend of increased speeds on highways and because of degradation in visual acuity with higher speeds, the 2.5-sec perception time is questionably low for use in designing stopping sight distance for high-speed roadways.

Friction Factor Design Values

The AASHO Policy considered several studies (5, 8, 9) in determining the design values for friction factors (Fig. 2 and Table 1). The friction factor versus speed relationship employed is representative of wet pavements measured in the referenced studies. Because wet values are considerably lower than dry values, the wet values are a rational basis for design. The question remaining, however, is, Does this friction factor versus speed relationship represent a typical critical condition? Figure 8 shows a percentile distribution of skid numbers (skid numbers are considered equivalent to 100f) at various speeds measured on 500 pavements (when wet) randomly dispersed throughout one state (14). These measurements were made in 1964 by using a modified ASTM skid trailer with standard ASTM test tires. Figure 2 shows that the curve that AASHO considers typical represents about the 35th percentile pavement in Figure 8. In other words, 35 percent of the pavements have friction factors lower than the design values. As such, the AASHO values are somewhat high. A more appropriate measure might be the 15th percentile pavement. Table 3 gives the friction factor versus speed relationship for 15th percentile pavement.

Stopping Distance Equation

If the relationship (shown in Fig. 2) of friction factor versus speed were indeed a typical critical curve, there would be no need for a verification of the validity of the stopping distance equation. The reason is that the AASHO Policy employed the equation to compute friction factors from the referenced stopping distance measurements and used these friction factors, with a safety margin applied, to recompute the stopping distance for design. In reality, this simply amounted to applying a safety factor to the original measured stopping distances.

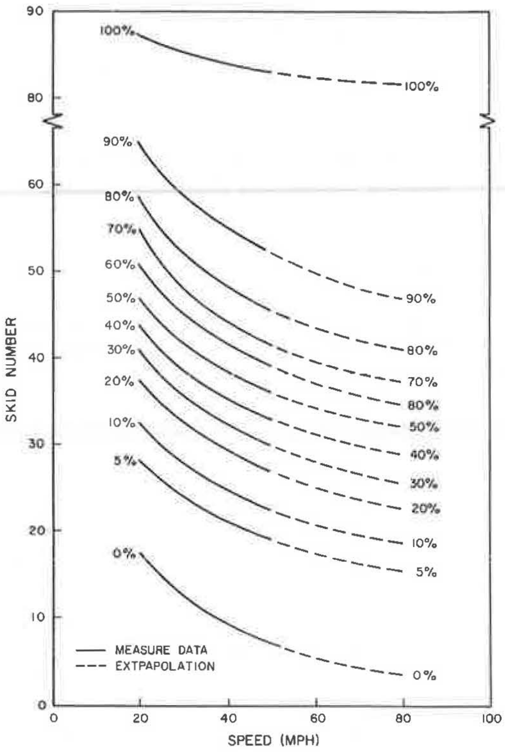


Figure 8. Percentile distribution of the skid number versus speed relationship for 500 pavements in one state.

TABLE 3

FRICTION FACTOR VALUES FOR THE 15TH PERCENTILE PAVEMENT IN ONE STATE

Design Speed	Friction Factor	Design Speed	Friction Factor
30	0.30	60	0.23
40	0.26	70	0.22
50	0.24	80	0.21

Because of the apparent need to consider lower friction factors (discussed previously), it is necessary to validate the equation for use of skid trailer measurements to compute stopping distance capability. Fortunately, tire quality test measurements (15) conducted by the Texas Transportation Institute in 1968 were available for analysis. The results of these measurements are given in the Appendix to this report.

The analysis considered 3,900 measurements of stopping distances of several tire brands and tire types on 5 test pavements of varying friction factors. Friction factors were measured (using the Texas skid trailer) on all 5 pavements periodically during the 6 months of testing. The analysis compared measured stopping distances with the stopping distances computed by using the appropriate friction factor in the standard equation. Although there was considerable variation between measured and computed distances, the analysis illustrated that only 10 percent of the trials involved stopping distances greater than those predicted by the equation.

In many of the trials, the vehicle stopped much shorter than predicted. This may be attributed to many variables such as experimental error, tire temperature, tire type, or pavement macrotexture. A considerable portion of the variation was attributed to measurements made on the pavement with the lowest friction factor (pad 4, approximate f-value of 0.20). On this pavement, the stopping distances in many trials were much shorter than those predicted by the equation. On the other 4 test pavements (f-values ranging between 0.44 and 0.64), the variation was considerably less than that on pad 4. The explanation for this phenomenon is apparently related to the accuracy of skid trailer measurements on lower coefficient pavements.

In summary, it appears that employing skid trailer values in the stopping distance equation yields reasonably conservative friction factor values for design purposes. The friction factors thus obtained should provide values of stopping distances that will be adequate for design of almost all conditions.

Assumed Speed for Conditions

The AASHO Policy states that it is not realistic to assume that travel will occur at full design speed when conditions are wet. Therefore, the Policy subjectively determined that the critical speeds for wet conditions should approximate 80 to 93 percent of design speed, as represented by average running speed for low-volume, or dry, conditions as shown in Figure 3. The AASHO Policy stated that this curve was developed from field data that related average spot speed to the design speed on horizontal curves. How ironic that the speeds taken from the curve in Figure 3 were used for

stopping sight distance design but not for horizontal curve design (for which full design speed is used for calculations).

It may be true that a critical speed (such as the 85th percentile) for wet pavements on horizontal curves is approximated by the average speed for dry conditions. Stopping sight distance, however, is also a design consideration on tangent alignment. It is reasonable to assume that low-volume average and 85th percentile speeds will be higher on tangent sections than on horizontal curves, especially for the lower design speeds. For free-flowing conditions the horizontal curvature is actually the only feature that limits speed (with the exception of very steep grades). No matter what the overall design speed of a highway may be, the operating speed of long level tangent sections on that highway is not limited by the geometry.

There are many variables that affect the spot-speed distribution of a highway section, including traffic volumes, percentage of commercial vehicles, contiguous design speed, type of facility, amount of roadside development, weather conditions, wet pavements, and posted speed limits (16). Many studies have been conducted to relate traffic speeds to posted speed limits, but no references are available that have measured the relationship between critical wet weather speeds and design speed.

The Texas Highway Department (17) collected wet and dry weather speed data at 16 sites in 1968. Various speed distribution parameters taken from these measurements are given in Table 4. The speed stations employed in this study were essentially level-tangent sections to limit the variation due to geometric design. Also, measurements were made of free-flowing vehicles only to eliminate variation due to traffic friction. In most cases, the overall design speeds of the highways were at or below the posted speed. From the data given in Table 4 and the description of site and measurement conditions, the following observations have been made pertaining to wet weather speeds:

1. There appears to be a leveling off of the higher speeds as posted speed increases. This is illustrated by the 85th percentile speed variation for 50, 60, and 70 mph posted speeds.

2. There is more variation in the distribution parameters for wet weather conditions than for dry weather conditions. This may be due to variations in rainfall intensity.

3. If it may be assumed that posted speeds approximate design speed, then it appears that the 85th percentile wet weather speed closely approximates the design speed for lower design speed highways (less than 50 mph).

4. The wet weather 85th percentile speed averages about 3 mph higher than the dry weather average speed. This variation has an indicated regression that would vary from 0 mph at lower speeds to 5 mph at the higher speeds.

TABLE 4

SPOT-SPEED DISTRIBUTION PARAMETERS FOR WET AND DRY WEATHER CONDITIONS

Site	Site Conditions		Wet Weather Speed Percentile			Dry Weather Speed Percentile			
	No. of Lanes	Posted Speed	50th	85th	90th	Average	50th	85th	90th
1	4	50	45	51	53	51	50	58	60
2	4	60	47	55	57	52	51	58	60
3	4	60	50	58	60	63	61	70	72
4	4	60	55	62	64	59	58	63	66
5	4	60	56	63	65	60	59	65	67
6	4	60	58	68	70	60	60	70	72
7	4	60	60	67	69	66	65	71	74
8	4	60	61	68	69	64	64	70	71
9	4	60	64	71	73	67	66	73	75
10	4	70	52	59	62	61	61	68	70
11	4	70	52	62	64	57	56	63	65
12	4	70	54	61	63	58	57	67	68
13	2	70	54	64	65	59	59	69	70
14	4	70	56	64	65	62	61	70	72
15	4	70	55	64	65	60	60	70	71
16	2	70	56	64	66	61	61	70	71

TABLE 5

A DERIVATION OF CRITICAL WET SPEEDS FOR DESIGN

Design Speed (mph)	AASHO Average Dry Speeds on Curves (mph)	Assumed Difference Between Average Dry Speeds on Tangent and Curves (mph)	Derived Average Dry Speeds on Tangents (mph)	Assumed Difference Between Average Dry Speed and 85th Percentile Wet Speeds on Tangents (mph)	Derived 85th Percentile Wet Speeds (mph)
30	28	6	34	0	34
40	36	6	42	0	42
50	44	6	50	0	50
60	52	5	57	2	59
65	55	4	59	3	62
70	58	3	61	3	64
75	61	2	63	4	67
80	64	1	65	5	70

The AASHO Policy's "assumed speeds for condition" have no objective basis. The question then is, What speeds should be used for design? One argument holds that the design speed should be used as the critical design speed for stopping sight distance, just as it is used for horizontal curve design. This could be justified on the basis that, at some time, a highway might be expected to have a posted speed limit either at or above the design speed. In addition, it might be expected that some drivers will exceed the posted limit, regardless of weather conditions.

Another basis for critical speeds to use in stopping sight distance design could be derived from the low volume curve in Figure 3 and from the Texas speed data. This would require that assumptions be made regarding the relationship between average dry speeds on tangents and average dry speeds on horizontal curves. These assumed differences are given in Table 5, along with the AASHO low-volume average dry speeds on horizontal curves, the derived low-volume average speeds on tangents, the differences between average dry speeds and 85th percentile wet speeds on tangents (discussed previously), and the derived critical speeds assumed for design. With this method, the critical speeds are somewhat higher than those in the AASHO policy.

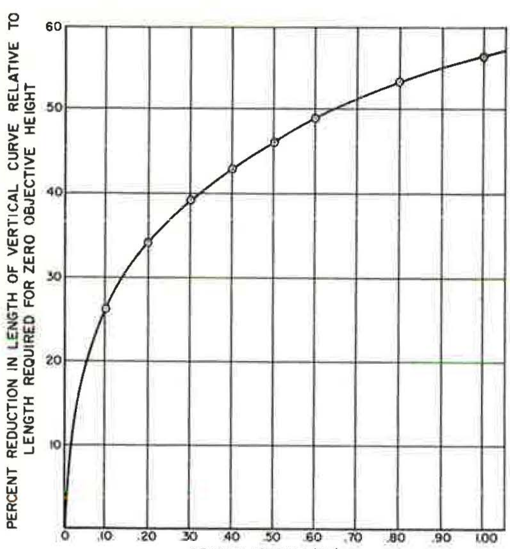


Figure 9. Relationship between object height and required length of crest vertical curve.

Driver Eye Height

It appears that the driver's eye height has not been significantly lowered since the 1960 automobile models. Therefore, the 3.75-ft eye height may be a reasonably valid criterion for the design of stopping sight distance; however, the percentile distributions shown in Figure 4 are for models and are not distributions experienced on the highway. It is possible that a considerable percentage of driver eye heights on the highway are lower than 3.75 ft because of the introduction and high-volume sales of automobiles such as the Ford Mustang and the Chevrolet Camaro.

Object Height

The AASHO Policy considers that a zero object height would provide for the safest sight distance design. The 6-in. object height, however, was selected because it supposedly represented a point of diminishing returns in terms of the cost of excavation, considering the relationship between the object height and the length of

TABLE 6

SAFE STOPPING SIGHT DISTANCES FOR HEAD-ON COLLISION CRITERION

Design Speed (mph)	Assumed Speed for Condition (mph)	Perception-Reaction Time (sec)	Perception-Reaction Distance (ft)	Friction Factor (f)	Braking Distance on Level (ft)	Stopping Sight Distance: 1 Vehicle (ft)	Stopping Sight Distance: 2 Vehicle (ft)	Stopping Sight Distance: Rounded for Design (ft)
30	32	2.5	117	0.29	118	235	470	470
40	40	2.5	147	0.26	205	352	704	700
50	48	2.5	176	0.24	320	496	992	1,000
60	57	2.5	209	0.23	471	680	1,260	1,260
65	60	2.5	220	0.23	522	742	1,484	1,480
70	62	2.5	228	0.23	557	785	1,570	1,570
75	65	2.5	239	0.22	640	879	1,758	1,760
80	68	2.5	250	0.22	700	950	1,900	1,900

vertical curve required to provide stopping sight distance for various object heights. This relationship between object height and length of vertical curve is shown in Figure 9 (1). The point of diminishing returns appears to be more nearly in the range between 0.1 and 0.3 ft.

This basis for designing vertical curvature may provide safety for most operational conditions; however, whether it is entirely adequate for providing relatively high overall safety of operation is questionable. There appear to be many operational situations that would require a zero object height for safety, such as either a horizontal curve or an intersection hidden by a crest vertical curve. Therefore, the present criterion for object height bears no relation to many of the operational requirements for safe stopping sight distance.

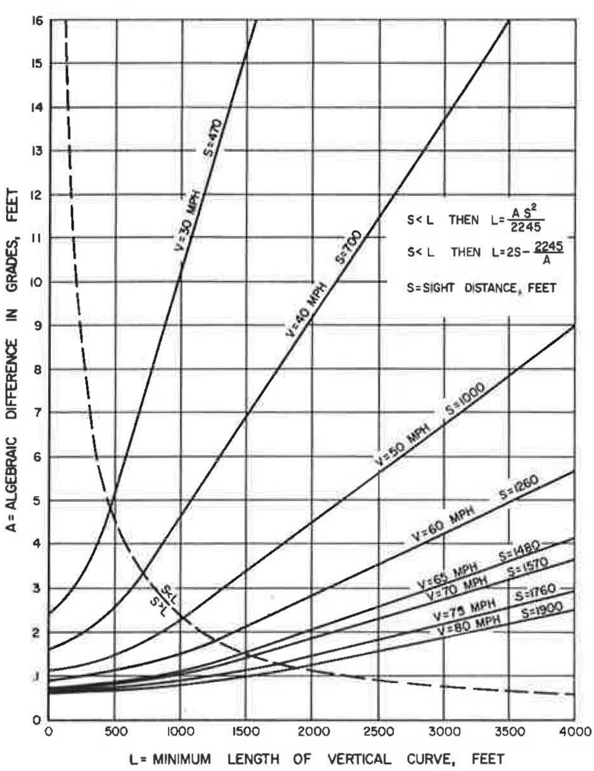


Figure 10. Relationship between stopping sight distance and length of vertical curve for head-on collision criterion.

Head-on Collision Criterion for Stopping Sight Distance

At this point, an entirely new philosophy of stopping sight distance design will be offered. On 2-lane highways, a vehicle must travel in the opposing lane to pass a slower moving vehicle. Legally, this is only possible where adequate passing sight distance is available and no restrictions are placed on passing. Traffic engineers and traffic law enforcement officers, however, well know that no-passing zones are often violated. Considering those drivers who violate no-passing zones and also drivers who wander into the opposing lane because of drowsiness or intoxication (2 characteristics that are all too common, especially at night), we might appropriately design for nighttime stopping sight distance to opposing headlights, allowing for the closing rate of the 2 vehicles. It is true that a driver can see the beams of opposing headlights well before he can see the headlights themselves. He cannot always perceive, however, that the opposing vehicle is in his lane until

TABLE 7

SAFE STOPPING SIGHT DISTANCES FOR STATIONARY OBJECT CRITERIA THAT EMPLOY DESIGN SPEEDS FOR WET CRITICAL SPEEDS

Design Speed (mph)	Assumed Speed for Condition (mph)	Perception-Reaction		Friction Factor (f)	Braking Distance on Level (ft)	Stopping Sight Distance	
		Time (sec)	Distance (ft)			Computed (ft)	Rounded for Design (ft)
30	30	2.5	110	0.30	100	210	210
40	40	2.5	147	0.26	205	352	350
50	50	2.5	184	0.24	347	531	530
60	60	2.5	221	0.23	522	743	750
65	65	3.0	287	0.22	644	931	930
70	70	3.0	309	0.22	742	1,051	1,050
75	75	3.5	386	0.21	893	1,279	1,280
80	80	3.5	412	0.21	1,016	1,428	1,430

TABLE 8

SAFE STOPPING SIGHT DISTANCES FOR STATIONARY OBJECT CRITERIA THAT EMPLOY DESIGN SPEEDS FOR WET CRITICAL SPEEDS

Design Speed (mph)	Assumed Speed for Conditions (mph)	Perception-Reaction		Friction Factor (f)	Braking Distance on Level (ft)	Stopping Sight Distance	
		Time (sec)	Distance (ft)			Computed (ft)	Rounded for Design (ft)
30	34	2.5	125	0.28	138	263	260
40	42	2.5	154	0.26	226	380	380
50	50	2.5	184	0.24	347	531	530
60	59	2.5	217	0.23	504	721	720
65	62	3.0	273	0.23	557	830	830
70	64	3.0	282	0.23	593	875	880
75	67	3.5	345	0.22	680	1,025	1,030
80	70	3.5	360	0.22	742	1,102	1,100

the headlights are visible. The AASHO Policy states that headlight beam height is approximately 2 ft from the ground level.

For minimum safety requirements, the safe stopping sight distances for the head-on situation should be doubled. Although seeming to be more than enough, this is probably conservative because it assumes that the opposing vehicle will stop, and it may not if the driver is either intoxicated or asleep. In designing for the head-on situation, one is again faced with the problem of establishing critical speeds for design. In this case, the critical speed should be, for example, the 85th percentile night wet weather speed. The 1968 speed survey (18) by the Texas Highway Department shows that the nighttime 85th percentile speed is about 2 mph lower than the daytime 85th percentile speed. If the assumptions in Table 5 are accepted, then the 85th percentile night wet weather speed might be derived as 2 mph lower than the 85th percentile day wet weather speeds.

The 2.5-sec perception-reaction time (assumed by AASHO) is probably low for the higher travel speeds, but that evaluation was based on a stationary object without its own source of illumination. For stopping sight distance design to an opposing vehicle, the 2.5-sec perception-reaction time is probably adequate. Table 6 gives the proposed safe stopping sight distances for the head-on collision criterion employing the assumptions discussed in this subsection. The design friction factors represent the 15th percentile pavement as previously discussed (Table 3). Figure 10 shows the relationship between stopping sight distance and length of vertical curve for the head-on collision criterion.

Stationary Object Collision Criterion for Safe Stopping Sight Distance

For multilane, divided, and possibly undivided highways, there is no apparent need for the head-on collision criterion. There are, no doubt, other operational conditions

that have respective stopping sight distance requirements. Until these operational requirements can be defined, the stationary object criterion is all that is available.

Table 7 gives stopping sight distances based on the assumption that the critical speed is equivalent to the design speed. Table 8 gives stopping sight distances based on the critical speeds derived in Table 5. In both Tables 7 and 8, the 15th percentile pavement given in Table 3 is employed for friction factor values. In addition, the perception-reaction times for the higher speeds have been adjusted upward, in light of the earlier discussion in this section.

CONCLUSIONS

1. The commonly used criterion of a 2.5-sec perception-reaction time for the braking maneuver was based on a subjective extrapolation from laboratory studies. This evaluation indicates that the perception-reaction time is highly variable and, under critical conditions, could be higher than 2.5 sec. Because of the trend toward higher speeds and the concomitant degradation of a driver's visual acuity with higher speed, the 2.5-sec criterion is questionable for use in designing stopping distance for high-speed roadways.
2. Based on skid trailer measurement of 500 pavements randomly dispersed throughout one state, the AASHO design friction factor values do not represent a critical level of stopping capability. The AASHO values are representative of the 35th percentile pavement in the one state. In other words, 35 percent of these pavements could not provide adequate stopping distance to meet minimum stopping sight distance standards.
3. The use of skid trailer values (which are related to design speed) in the standard stopping distance equation will yield reasonably conservative stopping distance values for use in design. This evaluation was based on 3,900 stopping distance tests, which were related to computed stopping distances derived from test speed and a representative friction factor versus speed relationships for the test pavements.
4. The AASHO Policy's "assumed speed for conditions" has no objective basis. Because of lower friction factors when pavements are wet, the wet condition is the rational basis for design. The AASHO Policy states that it is not realistic to assume that travel will occur at full design speed when conditions are wet; however, the "assumed speed for conditions" is arrived at by employing average speeds for dry conditions on horizontal curves (of given design speed) as the critical wet speeds (related to design speed) for stopping sight distance design. Other bases for determining the "assumed speed for conditions" are presented.
5. A 3.75-ft driver's eye height is employed in the measurement of stopping sight distance design. This height is representative of the distribution of 1960-model automobiles. Although no data are available, this eye height is reasonably representative of current production automobiles. An eye height representative of vehicles on the roadway could be lower, however, because of the introduction and high-volume sales of automobiles such as the Ford Mustang, the Chevrolet Camaro, and the Volkswagen.
6. Theoretically, a zero object height would provide the safest sight distance design. The 6-in. object height used for the measurement of stopping sight distance design was supposedly selected on the basis of diminishing returns, in terms of the cost of excavation for crest vertical curves. Because excavation is no longer the major cost of highway construction, a more appropriate object height would be in the range of 0.1 to 0.3 ft.
7. The present criterion for object height bears no relation to many of the operational requirements for safe stopping sight distance. There appear to be many operational conditions that require a zero object height for maximum safety, such as a horizontal curve or an intersection hidden by a crest vertical curve.
8. There is a need to design nighttime stopping sight distance for opposing vehicle headlights on 2-lane highways. It is not uncommon for vehicles to be in the opposing lane even if there are legal restrictions. The opposing driver may be asleep, intoxicated, or otherwise openly violating the restriction.

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Appendix

TIRE TEST RESULTS

The 1966 National Traffic and Motor Safety Vehicle Act provided for the development of a uniform-quality grading system for pneumatic passenger-vehicle tires. In order to develop this system, the National Bureau of Standards conducted tests on tires currently in production to provide the necessary data base. Under contract to the National Bureau of Standards, the Texas Transportation Institute undertook the testing of 95 sets of tires during the period from March through November 1968 (15). The various sets of tires included in this program are given in Table 9. Each set of tires was tested to provide data on tractional characteristics when stopping with locked wheels and to determine loss of traction when driving through curves.

The pavements used in this test program were specially designed to achieve predetermined coefficients of friction. They included 4 different asphalt pavements and 1 portland cement concrete pavement. Each stopping pad was 24 ft wide and 600 ft long, having a cross slope of 2 in. in 24 ft.

Test Vehicle

The automobile used in this test program was a 1968 4-door Bel Air Chevrolet. Modification was made to the suspension system, including a change to heavy-duty coil

TABLE 9

TIRE SETS TESTED

Tire	New	Mileage Worn	Random Rerun (new and mileage worn)
Bias ply	20	13	5
Radial	8	8	4
Wide oval	12	0	2
Snow	11	0	2
Police	2	0	1
SAE	4	0	2
Wide slicks	1	0	0
Total	58	21	16

springs and heavy-duty shock absorbers. Prior to each day of testing, the vehicle height was determined by measuring the height of marks placed on the bumper at each corner of the car. This procedure was established to determine if deterioration was occurring in the suspension system. Air pressure for the automobile tires tested was 24 psi cold.

The tire-test vehicle was equipped to indicate and record the following information: distance traveled as a function of time, velocity of the vehicle as a function of time, rear-wheel lock-up point, and lateral forces (transverse accelerations). Distance and velocity data were obtained from a track-test fifth-wheel assembly attached to the rear bumper. Data were recorded on a Honeywell Visicorder. The ac power required for the Visicorder was supplied by a gasoline engine generator mounted in the trunk.

Test Surfaces

The location of the Texas A&M Research Annex on property that had previously been a jet trainer airfield permitted a wide choice in the specific location of the various test pavements. The study called for the design and construction of 4 different surface textures produced from selected aggregate and a single grade and type of asphalt cement. The program also required a fifth surface that consisted of selected portions of the existing portland cement concrete runways. These different surfaces were expected to have particular coefficients of friction that would remain constant for the duration of the study.

The preparation of the existing portland cement concrete pad consisted of a thorough cleaning. The other surfaces were to be designed and constructed to provide a range of friction coefficients between 0.20 to 0.60. Pavements were produced that covered a range of 0.18 to 0.64 (as measured with a skid at 40 mph) at the beginning of testing.

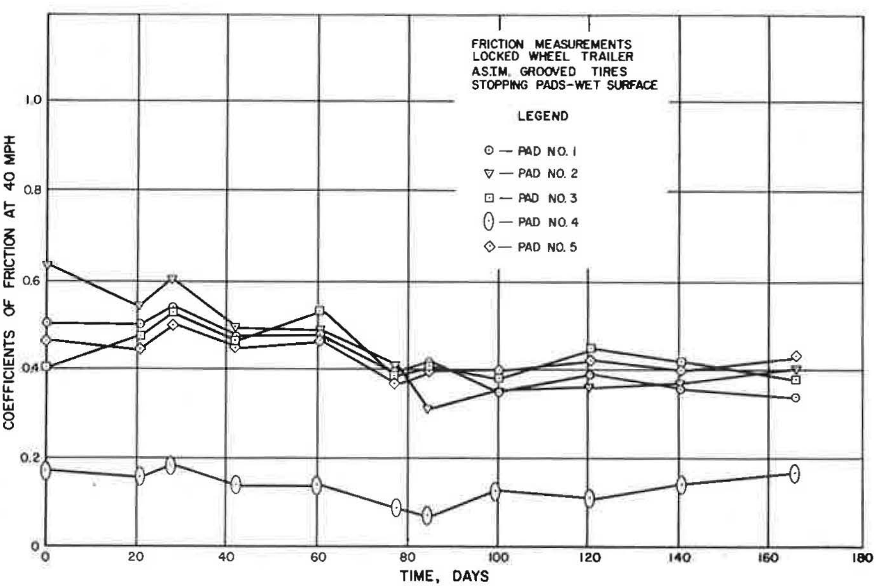

Figure 11. Variation of test pad coefficients over period of testing.

TABLE 10

LOCK-UP VEHICLE SPEEDS FOR EACH TEST PAD

Pad	Lock-Up Speeds (mph)			Pad	Lock-Up Speeds (mph)		
1	30	40	50	4	15	25	35
2	35	45	55	5	30	40	50
3	30	40	50				

The spread of these coefficients was reduced during the course of the project to a range of 0.18 to 0.44. The friction coefficients over the period of testing are shown in Figure 11 for each test pavement.

Skid Trailer Measurements

The friction values shown in Figure 11 were obtained with the Texas Highway Department skid trailer run with standard ASTM grooved test tires. The source of water for wetting the pavements was a 4,000-gal water truck complete with spray bars and a controlled pumping system capable of producing a uniform flow and distribution of water. On each pad, 1 pass of the watering truck preceded 2 passes with the skid trailer (one in each direction). The skid trailer's self-watering system was not used.

Friction determinations were made at 20, 30, and 40 mph. For each speed, 6 locked-wheel skids were made on each pad (3 in each direction). The points plotted in Figure 11 represent averages of these 6 measurements.

Stopping Distance Tests

The test vehicle was subjected to 4-wheel lock-up 4 times for each of the speeds given in Table 10. Two test trials were conducted in each direction for each pad, tire set, and speed. The speeds given in Table 10 were determined at the beginning of the test program, so the driver would not be subjected to extremely unsafe conditions.

On pads 1, 3, 4 and 5, 1 pass of the watering truck preceded each 2 passes of the test vehicle (once in each direction). On pad 2, the same procedure was followed, except that, prior to the 55-mph test run, 1 pass of the watering truck preceded each trial run. This was necessary because of the nature of the pavement and time consumed by the driver in accelerating to the 55-mph speed.

Approximately 3,900 total stopping distance measurements were made on the following tire set types: new bias-ply, worn bias-ply, new radial, worn radial, and new wide oval.

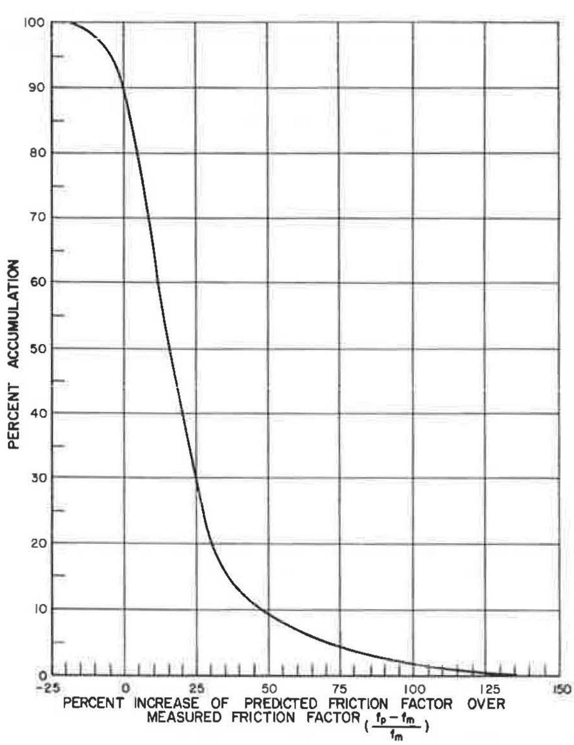

Figure 12. Percentile distribution of percentage deviation between measured and computed friction factors for all tests.

Test Results

For each stopping distance test run, it was possible to associate a computed friction factor with the test speed, based on the skid trailer measurements made at 20, 30, and 40 mph. The computed friction value and the test speed were used to compute the predicted stopping distance by using the standard equation $d = V^2/30f$. The prediction reliability of the equation could then be analyzed by comparing the predicted values with the measured values.

Figures 12 and 13 show percentile distributions of test runs related to

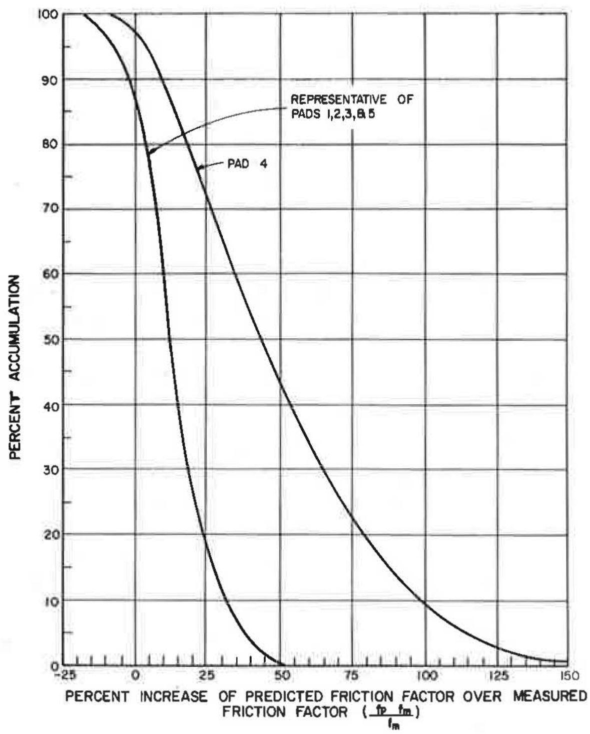

Figure 13. Percentile distribution of percentage deviation between measured and computed friction factors by test pad.

the percentage difference between the measured friction factor and the computed friction factor, (fc - fm)/fm. In 90 percent of the 3,900 test trials, the vehicle stopped in a shorter distance than that predicted by the standard stopping distance equation (Fig. 12). Figure 13 shows that, for the lowest coefficient pavement (pad 4), the vehicle stopped considerably shorter than predicted. No explanation can be offered for this phenomenon.

Additional analyses of these data indicate that (a) the equation is somewhat more reliable for lower stopping speeds; (b) new radial and new wide oval tires have a somewhat better stopping capability than new bias-ply tires; and (c) new tires have a somewhat better stopping capability than worn tires (10,000 high-speed miles of wear).

Discussion

D. W. LOUTZENHEISER, Highway Standards and Design Division, Bureau of Public Roads, Federal Highway Administration, U. S. Department of Transportation—Glennon's evaluation of sight distance design criteria is very timely because the AASHO Committee on Planning and Design Policies, which prepared the document examined, is now engaged in revising the AASHO policy on stopping sight distances.

In the AASHO concept for highway design, the minimum sight distance available on a highway is made sufficiently long to enable a vehicle traveling at or near the likely top speed to stop before reaching an object in its path. Although greater length is desirable, sight distance at every point along the highway should be at least that required for a below-average operator or vehicle to stop.

Minimum stopping sight distance for a stated speed is derived as the sum of 2 distances. One is the distance traversed by a vehicle from the instant the driver sights

an object for which a stop is necessary until he decides to act. The other is the distance required to stop the vehicle after the brake application begins. The design control is a minimum value for each design speed, to be used for that speed on any highways and for all conditions. Derivation of these values requires determination of 3 factors over the range of speeds used in design, namely, perception and reaction time, coefficient of friction, and vehicle speed. The first 2 factors lend themselves to scientific measurement under controlled laboratory conditions, but evaluation of such measurement for general highway conditions is difficult because of variations in physical properties of tires and roadway surfacing materials, physiological differences among drivers, weather, and the highway geometric details. Judgment, therefore, plays a major role in the determination of representative values for needed length of sight distance to ensure reasonably safe operation and at the same time be economically attainable.

Glennon lists 8 conclusions from his evaluation. Each is worthy of comment.

1. He questions the adequacy of 2.5 sec for perception-reaction. Admittedly, there is little factual support for 2.5 sec, and this value as well as any other that might be proposed is subject to conjecture. Results of actual investigations would be welcomed. To simply question the 2.5 sec offers little guidance as to a correct value. There are no plans to alter the 2.5-sec value in the revision of the AASHO sight distance standards.

2. He points out that the friction factor used in the stopping distance equation is higher than the skid numbers for 35 percent of pavements tested in one state. He suggests that a more appropriate measure might be the 15th percentile pavement. This is very useful information, and his suggestion seems to be a sound one. Each state, and perhaps each highway district within a state, could utilize the results of friction tests to determine sight distance requirements.

Two additional comments seem appropriate. First, the friction factor in the equation is the average value over the entire length of stop. Because friction increases as speed decreases, the average friction is considerably higher than the skid number measured at uniform speed. Thus, a friction factor (average) of 0.33 as used in the AASHO Policy for a stop from 40 mph may not be out of line with a factor of 0.26 measured at a uniform speed of 40. There is need to develop better correlation between actual stopping distance and calculated stopping distance based on friction factors measured with skid trailers. The former involves variables such as brake fade and tire-pavement relations that are not measured by the uniform speed method (see also item 4).

The second comment is that a long sight distance cannot ensure that a slippery pavement is a safe pavement, although a long sight distance may help. It might be wiser to devote a larger share of the highway dollar to improving skid resistance than to providing long sight distances. Such antiskid improvements could be programmed on a selective basis, if desirable, so that pavements would be treated first where sight distances are near minimum.

3. He shows that the use of skid numbers obtained with skid trailers will yield reasonably conservative stopping distance values for use in design. In many of the trials the vehicle stopped much shorter than predicted. This reassuring bit of information is welcome news. The reason for the conservatively low stopping distances may be as suggested in the first comment for item 2.

4. He concludes that the AASHO Policy's "assumed speed for conditions" has no objective basis. Interestingly enough, the results given in Table 4 support the Policy's position that speeds are somewhat lower when pavements are wet than when dry. The attempt by the author to relate measured speeds to design speed on the assumption that posted speed limits are approximately equal to design speeds is debatable. The AASHO Committee is proposing that the vehicular speed to be used in the stopping sight distance equation should be the same as the design speed of the highway, even with wet pavements. These speeds are, of course, considerably higher than those utilized in the development of current AASHO standards and somewhat higher than those proposed by Glennon.

5. He observes that the driver's eye height could be lower now than when data were compiled for the AASHO Policy. This is true; however, the eye height of 3.75 ft is

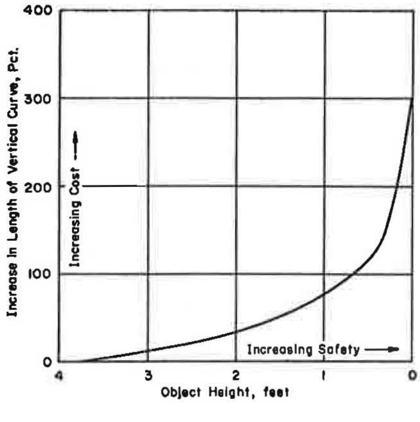

Figure 14. Relationship between object height and length of vertical curve needed to provide safe stopping distance (based on eye height of 3.75 ft and maximum object height of 3.75 ft).

lower than that found for the Corvair, Falcon, Valiant, and other compact cars that were in production at the time the standards were last revised. It is well below that for the Volkswagen and is believed to be on the conservative side for all but a very few foreign-made sports cars.

6 and 7. He proposes an object height in the range of 0.1 to 0.3 ft. Of all the variables that enter into the determination of minimum lengths of vertical curves, this one is least adaptable to scientific derivation. The value of 0.5 ft is admittedly a compromise that is vulnerable to challenge. Glennon arrived at his suggested values by what he refers to as the point of diminishing returns. However, his application of the economic law of diminishing returns is somewhat at variance with normal procedures. Usually, the point of diminishing returns is said to be reached when a relatively large expenditure is needed to purchase a relatively small benefit. In this case the length of vertical curve is a measure of expenditure and the benefit, which is the increase in safety, is measured by a lowering of the object height. Viewed in this sense, the point of diminishing returns appears to be reached at an object height of about 0.5 ft (Fig. 14). It is actually somewhat higher than this, however, because earthwork quantities increase almost exponentially with increasing lengths of vertical curves. Consequently, costs rise more rapidly than do the lengths of curves.

Glennon also suggests that a zero object height should be applied at horizontal curves. The AASHO sight distance design bases is that of a "single" minimum value for each design speed, applicable to govern design of both crest vertical curves and horizontal curves. In the original derivations (about 1940) it was concluded both practical and desirable to avoid a double sight distance standard. Instead the criteria for height of driver's eye and height of object were established as logical values that compromise the 2 curve conditions. Thus the design sight distance value derivation involves a "package" of interwoven factors—the 3 factors for stopping distance and the 2 factors for height—all of which must be jointly considered in any change. The intricacies of the interrelations could be more fully explained at some length, but further review is not made here.

8. He develops an entirely new philosophy for needed stopping distance for 2-lane roads. This is both refreshing and challenging. It is refreshing for the reason that the need for a new philosophy for both 2-lane and multilane highways is becoming increasingly apparent, and it is good that Glennon is willing to take the first step. Perhaps instead of stopping distance, the need is for maneuver distance or maneuver time or room. Possibly the most likely object to be dealt with then is another automobile. Even on 2-lane highways, a stopped vehicle in the travel lane may be a more likely hazard than an oncoming one. Regardless of what type of object is chosen, there is need to know how large the target must be before the driver can detect that a stop (or other maneuver) is called for. If, for example, a car is the critical object, must the driver see all above the headlights, even in daylight, before he can detect that evasive action is necessary? It seems reasonable that he might.

Glennon's philosophy is challenging in that some of his assumptions are controversial, and necessarily so. For example, he assumes that the minimum sight distance, driver eye to headlight height, should be double the stopping distance. Presumably this is based on the theory that both the "safe" driver and the errant driver will detect that an emergency stop is necessary and that their bumpers will just touch as they come to a halt. If each is traveling 65 mph, they can be no closer than $1/3$ mile apart when they commence this action. In the glare of oncoming headlights, or on a slight horizontal

curve, the drivers are a long distance away to detect a hazardous condition calling for immediate action. It hardly seems logical to expect that the driver who is at fault will remain in the wrong lane until the 2 cars meet, and even less likely that he will attempt to stop in that lane. If, however, he should continue on the wrong side of the centerline at a speed of, say, 60 mph and if the safe driver attempts to stop from, say, 60 mph, he will still be traveling at 16 mph when the 2 vehicles meet.

I hope that this report will stimulate action to further explore Glennon's philosophy and to develop new ones supported by research findings. Information relating accidents to sight distances would be a great help in deciding on minimum sight distance standards, but such information is scarce. A recent report is, however, rather enlightening in this regard and is worthy of mention (19). Although this report does not show a definite cut-off point or point of diminishing returns beyond which accidents show a sharp increase with a further decrease in sight distance, it does make clear that the greater the minimum sight distance (up to 2,600 ft, which was the limiting value used in the study) the safer the highway becomes. This should impress upon everyone that above-minimum values should be used whenever feasible. It should also lend support to the higher sight distance standards under consideration by AASHO that, if adopted, will result in sight distances up to 40 percent longer than those now in use.

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Accidents at Median Crossovers

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The purpose of this study was to examine the necessity for and the value of median crossovers on limited-access facilities. An accident analysis provided information on the frequency and nature of accidents at median crossovers. The type of and necessity for crossover usage by authorized personnel was determined from interviews with highway maintenance engineers and questionnaires completed by state police who patrol limited-access facilities. It was found that crossovers do cause frequent accidents in some locations, thus negating the positive benefits of having crossovers. Undesirable locations for crossovers include urban areas, interchange areas, and any location where use by the general public is likely. Applying more stringent controls to the location and use of median crossovers than now employed may result in a 5 percent reduction in accidents on limited-access facilities.

•SAFETY IMPROVEMENTS are often controversial subjects, especially when the subject is highways. Judgments must be made weighing lives and injuries against the hard realities of financing the construction and maintaining the highway systems. Median crossovers on rural and urban freeways and expressways are controversial design features. State police and maintenance forces claim that median crossovers are necessary and essential for their work and that more frequent location of crossovers is desirable. Engineers involved with highway safety maintain that crossovers create accidents, are not necessary, and should be eliminated.

Crossovers are located on divided roadways so that emergency and maintenance vehicles can cross the median to change their direction of travel. However, motorists also find crossovers convenient for their use, even though the maneuver is illegal. This creates an accident-producing situation. Accidents at median crossovers involving U-turning vehicles accounted for nearly 25 percent of the total accidents on several road sections during some years investigated in this study. During a 4-year period an average of 5 percent of all accidents on the majority of toll and Interstate roads in Kentucky were caused by vehicles using median crossovers.

The purpose of this study was to analyze existing crossover locations, usage, and accidents and to develop criteria for use in determining the necessity for and the location of median crossovers. Primarily, 3 sources of information were used. An inventory of existing crossover locations was taken to determine the prevailing policy, if any, on crossover design and location. A comprehensive analysis of U-turn accidents at median crossovers was performed. Interviews were conducted with district highway engineers and questionnaires were given to all state police who patrol Interstate or toll roads in Kentucky. The questionnaire also provided an opportunity for the state police to express their opinions concerning the location and necessity for crossovers.

INVENTORY OF EXISTING CROSSOVER LOCATIONS

An inventory of existing crossover locations was performed by 2-man teams who traversed the roads selected for study. Crossovers, interchanges, or other features of significance were logged to the nearest tenth of a mile. These loggings were then plotted to

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TABLE 1

CROSSOVER SPACING AND LOCATION

Road	Average Distance Between Crossovers and Interchanges	Average Distance of Crossovers from Interchanges	Average Distance Between Interchanges	Average Number of Crossovers Between Interchanges
I-64 (Lexington to Morehead)	2.6	2.3	5.5	1.3
I-64 (Frankfort to Louisville)	2.1	1.8	5.0	1.4
I-65 (Cave City to Elizabethtown)	2.4	2.0	5.7	1.4
Kentucky Turnpike	2.2	1.5	18.4	3.7
Blue Grass Parkway	2.3	1.9	14.8	5.4
Western Kentucky Parkway	2.2	0.6	14.4	6.4
Mountain Parkway	1.7	0.6	8.4	3.3
US-41	1.0	0.7	2.7	1.5

scale and examined for similarities to see if any patterns in design governed. Kentucky's traffic guidance manual (2) provides no recommendations for crossover location. The Kentucky Department of Highways Standard Drawing 14.04c states: "Maintenance Cross-Overs shall be constructed one half to one mile from the end of the acceleration lane taper or de-acceleration lane taper whichever provides for the furthest spacing from the interchange." Although not stated, the locating of crossovers this close to interchange areas is assumed to be strictly a convenience for maintenance forces to conduct snow removal work. This philosophy is also reflected in the following statement that appeared in an ASCE publication (1): "Usually crossovers are needed at each end of an interchange area so snow and ice equipment may reverse direction quickly to clear all entrance and exit ramps."

The suggested pattern of crossover location is not consistently followed on any of the Interstate or toll roads in the state. Crossovers are commonly found as near as 0.1 mile and as far as 5.0 miles from interchange areas. The average distance between crossovers and interchanges varies from 0.6 mile on the Western Kentucky Parkway and Mountain Parkway to 2.3 miles on I-64 (Lexington to Morehead) (Table 1). Generally there is at least one crossover between exits when interchange spacing exceeds 3 miles. The average distance between crossovers and interchanges is about 2.2 miles, with the exception of US-41 in Hopkins County where no consistent spacing between consecutive crossovers or between an interchange and a crossover was found.

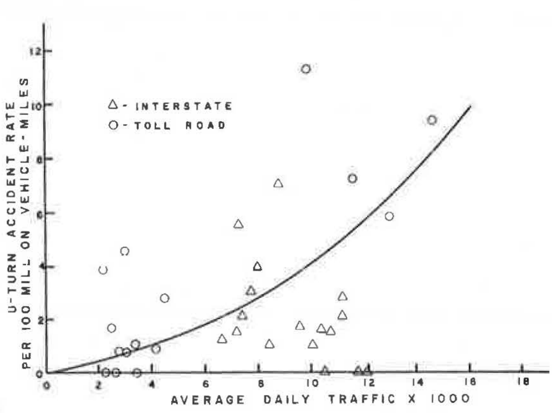

Figure 1. U-turn accident rate versus traffic volume.

ACCIDENT ANALYSIS

Accident reports for a 4-year period were copied from original state police records for the following controlled-access roads: I-64, I-65, Bluegrass Parkway (3-year period), Western Kentucky Parkway, Mountain Parkway, Kentucky Turnpike, and US-41 (Madisonville Bypass, Hopkins County). All U-turn accidents at median crossovers were counted and analyzed. The

following variables were found to affect the number of U-turn accidents on a given road:

1. Volume of traffic on the road,
2. Proximity to urban areas,
3. Presence of major interchanges between controlled-access facilities,
4. Number of crossovers,
5. Nearness of crossovers to interchanges,
6. Composition of the traffic stream,
7. Interchange spacing,
8. Width and type of median, and
9. All other roadway, weather, driver, and vehicle variables.

Of these variables, the first three are the most significant. Logically, as traffic volume increases, the probability of a U-turning vehicle coming in conflict with another vehicle increases (Fig. 1).

It is reasonable to assume that the drivers of U-turning vehicles are lost or confused, i. e., they may have made a wrong turn or missed a turn. More persons are likely to get confused in urban areas and in major interchange areas. Therefore, more U-turn accidents are likely to occur at such locations. This is verified by the U-turn accidents on the Kentucky Turnpike and US-41. There have been 34 U-turn accidents in a 4-year period involving southbound vehicles on the Kentucky Turnpike (Fig. 2). There were 16 at the first crossover south of Louisville, 8 at the second crossover south of Louisville, 4 at the third crossover south of Louisville, 2 at the fourth crossover south of Louisville, and 2 at other locations. In addition, 2 drivers involved in an accident admitted they were driving too slowly because they were looking for a crossover.

Ten accidents involved northbound vehicles on the Kentucky Turnpike (Fig. 3). All of these occurred after the opening of the Bluegrass Parkway in November 1965. There were 5 at the first crossover north of Elizabethtown, 2 at the second crossover north

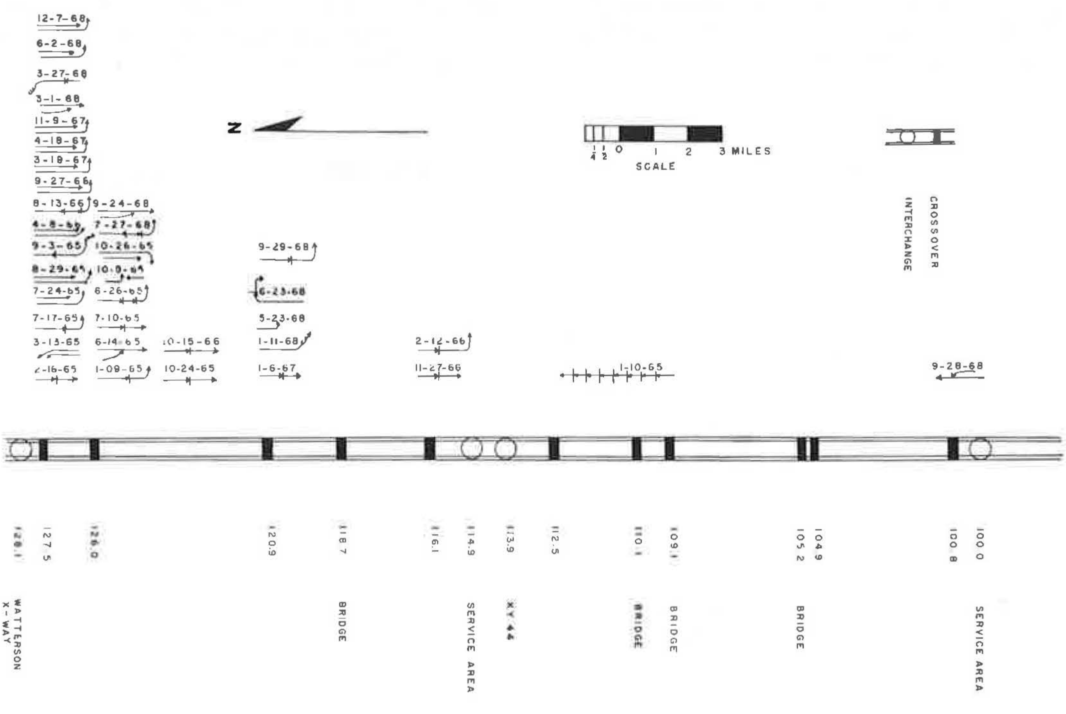

Figure 2. U-turn accidents on northern half of the Kentucky Turnpike.

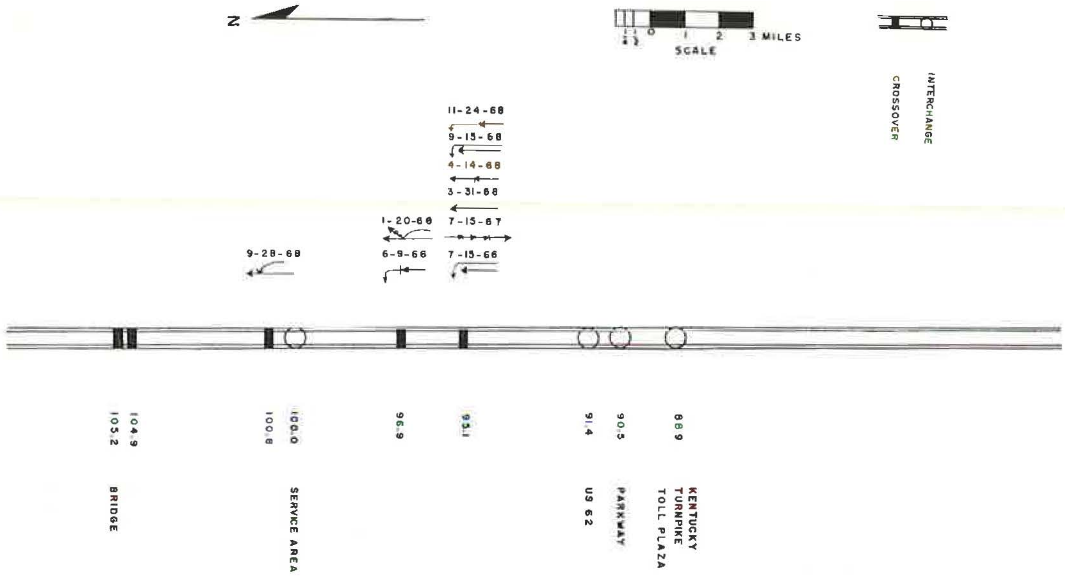

Figure 3. U-turn accidents on southern half of the Kentucky Turnpike.

of Elizabethtown, 1 at the third crossover north of Elizabethtown, and 2 at other locations. Many of these accidents were apparently caused by drivers who became lost or confused at the west end of the Bluegrass Parkway and were going north on the Kentucky Turnpike when they wanted to go south. Other situations where U-turn accidents occur near urban areas or major interchanges exist on I-65 and US-41 in Hopkins County. At the first crossover south of Elizabethtown on I-65, for example, there have been 13 accidents involving southbound U-turning vehicles in a 4-year period. The U-turn accidents

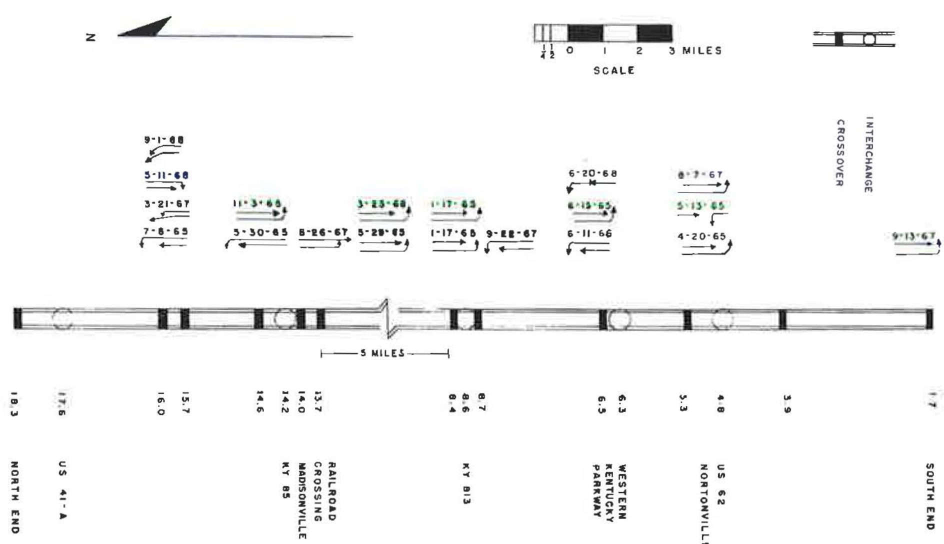

Figure 4. U-turn accidents on US-41, Madisonville Bypass.

TABLE 2

TYPE AND SEVERITY OF U-TURN ACCIDENTS

Type of Accident or Injury	Kentucky Turnpike		US-41		I-65 (South of Elizabethtown)		Total	
	No.	Percent	No.	Percent	No.	Percent	No.	Percent
Accident								
Right-angle	16	36	12	63	2	14	30	39
Oblique	17	38	7	37	8	57	32	41
Rear-end	11	25	0	0	3	21	14	18
Other	1	2	0	0	1	7	2	2
Injury[a]								
A	12	27	5	26	3	21	20	26
B	5	11	4	21	1	7	10	13
C	4	9	0	0	2	14	6	8
O	24	53	10	53	8	57	42	53

[a]Injury codes are as follows:
A—broken bones, visible cuts and lacerations, severe injuries, had to be carried from scene;
B—cuts and bruises of minor nature, need not be hospitalized;
C—complaint of injuries, none visible; and
O—no injuries.

on US-41 are clustered around the interchange with the Western Kentucky Parkway and the KY-85 interchange leading into Madisonville (Fig. 4).

Further evidence that driver uncertainty in traveling urban or interchange areas is a prime cause of U-turn accidents is supplied by the fact that 48 percent of the drivers of the vehicles making the U-turns were out-of-state drivers. Another 29 percent were in-state drivers but were out of their home county. Therefore, up to 80 percent of the drivers involved in accidents were probably unfamiliar with the roadway.

Other variables contribute to the accident problem in some locations. A study by Cribbins et al. (5) conclusively showed that for non-controlled-access facilities the accident rate increased with the number of openings in the median. An abnormally high number of crossovers on US-41 seems to contribute to the U-turn accident problem on that road.

The severity of U-turn accidents seems to depend roughly on the type of accident, i. e., right-angle, oblique, or rear-end collisions (Table 2). Right-angle collisions, which account for 39 percent of the total U-turn accidents, caused 59 percent of the severe injuries (Fig. 5). Less severe injuries resulted primarily from oblique and rear-end accidents. Overall, U-turn accidents are more prone to produce injury, as shown in Figure 6. In nearly all of the U-turn accidents studied, at least one innocent driver was involved.

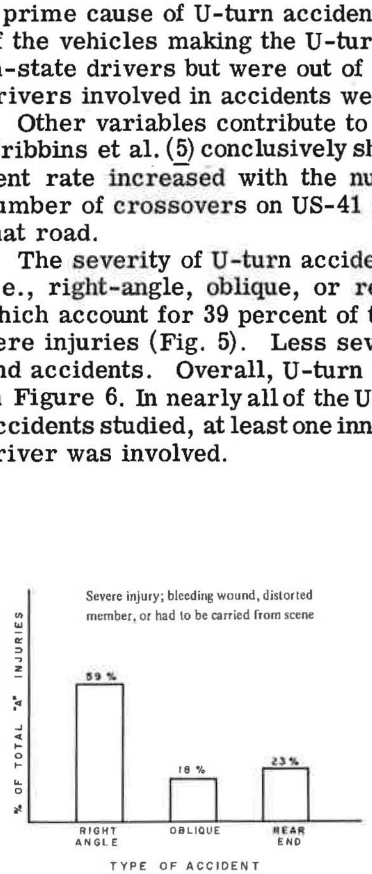

Figure 5. Severe injuries (type A) by type of U-turn accident.

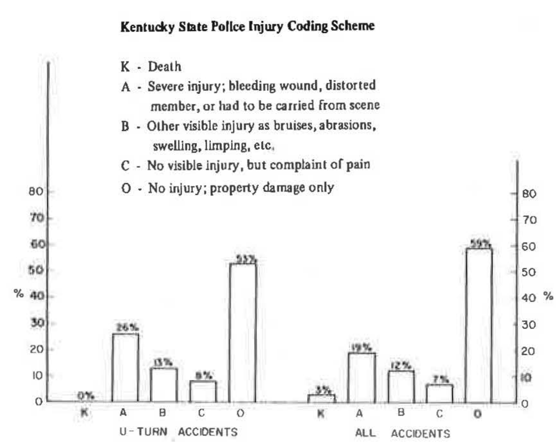

Figure 6. Injuries associated with U-turn accidents compared with injuries associated with all accidents.

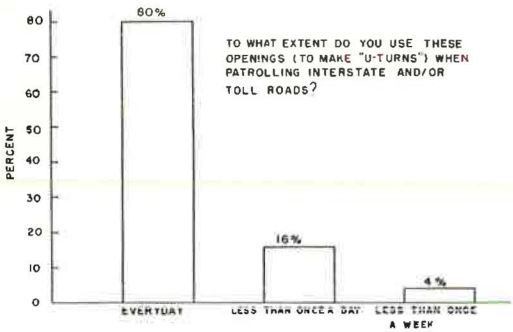

Figure 7. Frequency that state police cross median at crossovers.

MEDIAN CROSSOVER USAGE

An interview with the district engineers of several highway districts provided information on crossover usage by maintenance vehicles. Maintenance vehicles use crossovers primarily in winter during snow removal. The crossovers are convenient for clearing an interchange area and for turning around at county lines where maintenance responsibilities end. Other uses by maintenance vehicles are not readily predictable. As an example, when shoulder work is being performed, crossovers are used to lessen the distance for hauling materials. There are also special situations such as separate rest areas serving travelers in either direction of travel where usually one crew maintains both facilities. Crossovers at each end of the rest areas enable the maintenance personnel to service both without undue inconvenience.

State troopers who patrol Interstate and toll roads completed a questionnaire and provided much information on state police use of and attitude toward median crossovers. The questionnaire was designed by the Division of Research in cooperation with the Kentucky State Police. A representative from the Division of Research visited each post and explained the questionnaire. Cooperation from state police personnel was excellent. There were 132 completed from approximately 95 percent of the troopers solicited.

State troopers use crossovers on a regular basis (Fig. 7). Eighty percent of the respondents reported using crossovers at least once a day. Eighty-two percent of the troopers admitted crossing the median at nondesignated locations (Fig. 8). Some 19 percent of these cross the medians at other locations more than they do at regular crossovers. When responding to an emergency and not being near a crossover, troopers will cut across the median wherever they happen to be. The only time this is not feasible is during periods of snowfall or heavy rain or where the median is difficult to cross, as for example on the Mountain Parkway.

There were several questions designed to evaluate trooper attitudes toward crossovers. When asked if crossovers were absolutely necessary for state police activities, 84 percent replied that they were necessary. Of the 16 percent who thought they were not absolutely necessary, the majority were troopers who patrol the Kentucky Turnpike and I-65 where U-turn accidents are more prevalent. Further evidence of a difference in attitude is reflected by responses given in Table 3. Among all troopers, there is an obvious majority who favor more frequent spacing of crossovers. On the Kentucky Turnpike and I-65, where there is an accident problem, attitudes shift toward more stringent control of crossover usage. The majority of troopers on the Kentucky Turnpike feel that crossovers should be eliminated entirely. This would seem to indicate a general attitude that, where crossovers frequently cause accidents, their necessity is to be questioned. A notable exception to this is US-41 in Hopkins County, which has a deeply depressed median. Here the troopers have difficulty crossing the median, and they take a more forceful stand for the necessity of median crossovers.

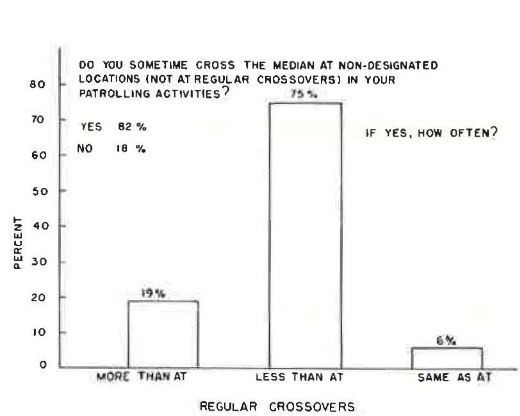

Figure 8. Frequency that state police cross median at locations other than crossovers.

TABLE 3

ATTITUDES OF STATE TROOPERS TOWARD LOCATION OF CROSSOVERS

Response[a]	Percent of All Troopers	Percent of Troopers on I-65	Percent of Troopers on Kentucky Turnpike
Located more frequently	65	60	0
Located less frequently	2	4	10
Eliminated entirely	7	12	50
Eliminated near interchanges and located very sparingly between interchanges	19	20	30
Other	7	4	10

[a]The responses were multiple choices to the question, If criteria were to be developed for the location of median openings or crossovers, do you feel that they should be . . .?

MEDIAN CROSSOVER LOCATION

All evidence indicates that guidelines and restrictive measures on the location and use of median crossovers are in order. A summary of the reasons for this conclusion follows:

1. Median crossovers are prone to cause accidents.
2. Crossovers seem to be a convenience, not a necessity, for maintenance activities.
3. There is no consistent policy being followed for crossover locations.
4. Crossovers are a necessity for state police activities only during inclement weather and where the median is difficult to cross (82 percent of troopers cross the median at nondesignated locations).
5. When the accident-producing aspect of crossovers is obvious, state police tend to be opposed to crossovers.

Furthermore, the AASHO traffic safety committee concluded (3): "Any openings in the median can be the scene of unsafe driving, and should, therefore, be kept to a minimum." On several roads, accidents at median crossovers pose a special problem. These roads warrant separate discussion.

Kentucky Turnpike

The combination of narrow medians, relatively high traffic volumes, and confusing junctions makes the Kentucky Turnpike especially susceptible to U-turn accidents. Because of the accident problem and the corresponding negative attitude of the state police toward median crossovers, crossovers on this facility might well be permanently closed. Crossovers located between bridge piers (Fig. 9) may be an exception. There have not been any accidents at these crossovers. Retaining these crossovers on a conditional basis and noting whether they result in accidents could produce evidence on whether the hidden crossovers may be a solution to the problem in some locations.

Figure 9. Crossover located between bridge piers.

I-65 (South of Elizabethtown)

Most crossover accidents on this road occur at one crossover. The first crossover south of Elizabethtown at milepost 87.3 has been the site of 13 U-turn accidents between 1965 and 1968. If this

crossover were eliminated, drivers would have to travel a few more miles and turn around at an interchange.

US-41 (Hopkins County)

The occurrences of U-turn accidents on this road have been clustered around the Western Kentucky Parkway and KY-85 (at Madisonville) interchanges. Figure 4 shows that the crossovers are generally located very close to the interchanges. This close spacing contributes to the problem by requiring quick decisions from the driver making the U-turn. Sixty-three percent of the U-turn accidents are right-angle accidents caused by drivers turning from the outside lane into the path of another vehicle. To eliminate the abnormal number of U-turn accidents on this road, it would be desirable to eliminate all the crossovers. With interchanges spaced on the average only 2.75 miles apart, elimination of crossovers might be acceptable if it were not for the deeply depressed median that troopers find difficult to cross. An alternate solution would be to eliminate crossovers near interchanges and have only one crossover, at most, between any two interchanges.

Interstate Roads

In general, Interstate roads do not need many crossovers. Interchanges are spaced on the average about 5.5 miles apart, and the median can be easily crossed, if necessary, The present spacing is adequate for most purposes. There are, however, some exceptions. Crossovers near interchanges might be eliminated or at least moved. Interchanges and crossovers should be spaced so that there is a fairly uniform distance between two crossovers or an interchange and a crossover. Because crossovers are designed for convenience, the somewhat erratic spacing now found on many road sections can hardly be justified.

Toll Roads

The present spacing of crossovers on most toll roads, with the exception of the Kentucky Turnpike, presents few problems. Where the median can be easily crossed, crossovers spaced 5 miles apart would suffice. On toll roads with deeply depressed medians that cannot be easily crossed, closer spacing may be desriable.

Other Considerations

Among those who favor the use of median crossovers, there seem to be 2 different theories as to crossover locations. The first group maintains that crossovers are going to be used by the general public in any event. Therefore, crossovers should be located in prominent locations, have adequate sight distance, and be conspicuously signed. The second group says that crossovers should be hidden from the public eye and not signed. An article on operational problems on controlled-access facilities (1) stated: "Crossovers should be as inconspicuous as possible to prevent use by the public.... For enforcement purposes signs prohibiting public use are required, such as NO U-TURN—FOR OFFICIAL USE ONLY."

The policy to place crossovers in inconspicuous locations and then sign them seems contradictory. At the present little effort is made to make crossovers inconspicuous. However, on the Kentucky Turnpike, there have been no accidents at the crossovers located between the bridge piers during the 4 years of the study. This would indicate the desirability of using hidden crossovers.

Responses on the questionnaire indicated that there is some question as to the wording of the sign FOR EMERGENCY AND MAINTENANCE VEHICLES USE ONLY. Many troopers questioned the length and the message of the sign. It may be in order to study the contrasting effects at crossovers having the present sign, no sign at all, and a sign with a negative connotation like NO U-TURNS or U-TURNS ARE ILLEGAL. It is doubtful that signing changes are a solution, but the possibility should be investigated.

CONCLUSIONS

The purpose of this study was to examine median crossovers on controlled-access facilities in an attempt to determine the necessity for them. Although crossovers are

desirable and worthwhile for state police and maintenance uses, crossovers can only be considered as a convenience, not a necessity, and should be eliminated if an accident problem arises. Accident prevention was the foremost consideration in the development of the following criteria that appear to be warranted with respect to crossover location:

1. Median crossovers should not be located in or near urban areas, i.e., cities having populations of 10,000 or greater.
2. Median crossovers should not be located near major interchanges, i.e., the intersection of 2 controlled-access facilities. There should be no crossover between the interchange area and the next interchange on all connections, except on some toll roads where distances between interchanges may be extremely great.
3. Median crossovers should not be located within 2 or 3 miles of an interchange.
4. Any median crossover located so that the general public may be tempted to use it will cause accidents and should be eliminated.

Applying more stringent controls to the location and use of median crossovers than now employed may result in a 5 percent reduction in accidents on Interstate and toll roads.

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Relationship Between Curvature and Accident Experience on Loop and Outer Connection Ramps

JOHN G. YATES, Westat Research, Inc.

This paper examines the relationship between accident rate and curvature on loop and outer connection ramps of the Interstate Highway System. Ramp characteristics considered include maximum curvature, location of ramp, enter or exit direction with respect to the main-line unit, and average daily traffic. Accidents per million vehicles are used as the primary indication of accident experience. Results of the investigation indicate that accident rates increase with maximum curvature for all right entering or exiting loops and outer connections except rural loops. Accident rates are lower on right entering or exiting outer connections with no curvature (less than 1 deg) than on those with curvature for all but one urban ADT category. All urban right entering or exiting loops and outer connections, except urban loops, show a positive correlation between ADT and accident rate. The effect of traffic volume on rural ramp safety for various curvatures exhibited mixed relationships.

•THE PURPOSE OF THIS PAPER is to examine the influence of maximum curvature and average daily traffic (ADT) on the accident rate of loop ramps and outer connection ramps. This study grew directly out of the first general analysis (1) of the Interstate System Accident Research, Study II (ISAR-II), and is similar to a recent ramp study conducted in part by the author (2).

This analysis is restricted to a study of right entering or exiting (REE) and left entering or exiting (LEE) loop ramps and outer connection ramps in urban and rural areas as defined in the ISAR-II data base. (Terms used in this paper are defined in a glossary at the end of the paper.) These 2 types of ramps are integral parts of cloverleaf interchanges, and all ramps in this study are part of the Interstate Highway System. These 2 types of ramps were not compared in this study, though various figures are available if such a comparison is desired.

This study examines the geometric variable, maximum curvature, the traffic variable, ADT, and several characteristics of ramp accidents. Maximum curvature, as measured in this data base, is not a continuous variable but a discrete variable with 10 possible categories of curvature varying from 0 to 36 deg and above. Maximum curvature as well as ADT data were each stratified into 3 groups. Also, accident rate comparisons were made between no-curvature and curvature outer connections as well as between low- and high-curvature loop ramps. Accident characteristics, such as time of accident and manner of collision, were also examined.

FINDINGS

1. Except for ramps located in rural areas, all REE ramps show an increase in accident rate with increasing curvature.
2. All rural LEE or REE outer connections as well as REE loops show a mixed relationship between accident rate and increasing ADT, whereas all urban REE or LEE outer connections show increasing accident rate with increasing ADT.

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3. Outer connection ramps without curvature have accident rates lower than those on outer connection ramps with curvature in urban and rural areas for all ADT levels except 0 to 500 for urban outer connections.

4. Urban loop ramps with high curvature have accident rates higher than those on urban loop ramps with low curvature for most ADT categories. But, rural loop ramps with low curvature have accident rates higher than those on rural loop ramps with high curvature for most ADT categories.

5. Urban loops and outer connections, regardless of left turning or right turning, dramatically differ from rural loops and outer connections in ADT levels (higher in urban), curvature (higher in urban), accident rates (higher in urban), manner of collision of accidents (mostly rear-end in urban), vehicles per accidents (greater than 1.5 vehicles per accident in urban), accidents involving commercial traffic (lower in urban), and in the period of occurrence of accidents (mostly between 7 a.m. and 6 p.m. in urban).

6. For urban REE, loops and outer connections, the percentage of rear-end collisions and the percentage of daytime (between 7 a.m. and 6 p.m.) accidents increase with maximum curvature.

7. For rural REE outer connections, the percentage of noncollision accidents increases with maximum curvature.

ANALYSIS

Data Characteristics

The data were obtained from the Interstate System Accident Research, Study II, data base. The number of data points available are as follows:

Ramp	REE Urban	REE Rural	LEE Urban	LEE Rural
Outer connections	1,548	1,364	122	101
Loops	1,084	852	10	7

Because of the insufficient number, LEE loops were excluded from this analysis.

Because there were so few 2-lane ramps, 1-lane and 2-lane ramps were not separated, but a distinction was made for analysis purposes between urban and rural and between REE and LEE ramps. Thus, there are 6 individual study groups in this analysis:

1. Urban loops, right enter or exit
2. Rural loops, right enter or exit
3. Urban outer connections, right enter or exit
4. Rural outer connections, right enter or exit
5. Urban outer connections, left enter or exit
6. Rural outer connections, left enter or exit

The first four are referred to as the primary study groups, and the last two referred to as the secondary study groups. Table 1 gives the contribution of data points by state for these 6 groups. Urban areas are

	Maximum Curvature			
Traffic Volume (Veh /Day)	<20	20-30	>30	All Curvature
0 - 1000				
1000 - 4000				
Over 4000				
All Volume				

The table has 40 cells each containing the following accident data:
1. Number of accidents a year/total traffic volume for year of vehicles.
2. Number of injuries a year/total traffic volume for year of vehicles.
3. Number of fatalities a year/total traffic volume for year of vehicles.
4. Property damage a year/total traffic volume for year of vehicles.
5. Number of accidents a year.
6. Accidents to injuries ratio.
7. Number of units.

Figure 1. Sample output table generated by computer program for each model.

TABLE 1

DATA POINTS BY CONTRIBUTING STATE

State	REE Loops				REE Outer Connections				LEE Outer Connections			
	Urban		Rural		Urban		Rural		Urban		Rural	
	No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent
Arizona	6	0.6	30	3.5	4	0.2	45	3.3	—	—	11	10.9
California	37	3.4	24	2.8	158	10.2	44	3.2	5	4.1	1	9.9
Connecticut	146	13.5	75	8.8	247	15.9	148	10.8	—	—	—	—
Florida	—	—	3	0.4	—	—	—	—	—	—	—	—
Illinois	487	44.9	230	27.0	550	35.5	304	22.3	85	69.7	15	14.9
Indiana	—	—	3	0.4	6	0.4	—	—	—	—	—	—
Kansas	110	10.1	—	—	179	11.5	—	—	11	9.0	—	—
Michigan	14	1.3	24	2.8	39	2.5	53	3.9	4	3.0	1	9.9
Minnesota	28	2.6	—	—	27	1.7	—	—	—	—	—	—
Mississippi	6	0.6	—	—	18	1.2	—	—	3	2.5	—	—
Montana	—	—	21	2.5	—	—	54	4.0	1	0.8	—	—
New York	35	3.2	10	1.2	40	2.6	25	1.8	—	—	—	—
North Carolina	125	11.5	172	20.2	133	8.6	285	20.9	5	4.1	21	20.8
North Dakota	5	0.6	4	0.5	4	0.3	10	0.7	—	—	9	8.9
Ohio	—	—	46	5.4	—	—	54	4.0	—	—	—	—
Oklahoma	2	0.2	5	0.6	2	0.2	15	1.1	—	—	2	2.0
Pennsylvania	10	0.9	8	0.9	13	0.8	14	1.0	—	—	—	—
Rhode Island	3	0.3	—	—	26	1.7	—	—	6	4.9	—	—
South Dakota	—	—	—	—	—	—	—	—	—	—	16	15.8
Vermont	—	—	28	3.3	—	—	70	5.1	—	—	13	12.9
Virginia	62	5.7	77	9.0	94	6.1	123	9.0	2	1.6	7	6.9
Wisconsin	8	0.7	92	10.8	8	0.5	115	8.4	—	—	5	5.0
Total	1,084	100	852	100	1,549	100	1,364	100	122	100	101	100

dominated by Illinois data while rural areas are dominated by both Illinois and North Carolina data.

Analytical Approach

The accident data for the 2 ramp types studied were tabulated by a computer program that was written by using a previously generated n-tuple tape for each ramp as input data. (In order to speed processing time, the n-tuple tape took only a limited number of variables from the original number in the ISAR-II data base.) A typical computer table of the accident data is shown in Figure 1. The tables are labeled at the top as to ramp type and location. The first 3 columns represent 3 maximum curvature categories. The fourth column sums the preceding 3 columns. Similarly, the first 3 rows provide a 3-way breakdown for ADT, and the fourth sums the preceding 3 rows. The lower right cell gives accident data for all the data points in that table. The various measures of safety and descriptive statistics tabulated for each cell of the tables are also shown in Figure 1. Computer programs were also used to tabulate accident charactertistics and frequency distributions of data by ADT, curvature, and state.

Urban Versus Rural and REE Versus LEE

Data given in Tables 2, 3, and 4 show the marked differences among the 3 urban study groups and the 3 rural study groups. Table 2 gives ADT frequency distributions for each of the 6 study groups. Obviously, the ADT frequency distributions for the 3 rural study groups differ from the distributions for the 3 urban study groups. ADT distributions peak at lower volumes and drop off more rapidly in the rural than in the urban distributions.

Table 3 gives maximum curvature frequency distributions for each of the 6 study groups. Curvature distributions peak at a lower maximum curvature in the 3 rural than in the 3 urban distributions. Of course, this is expected because land in urban areas is usually more difficult to obtain for an Interstate Highway than in rural areas.

Table 4 gives accident characteristics for each of the 6 study groups. Because the LEE outer connections in rural areas have only 14 accident samples, the accident

TABLE 2

AVERAGE DAILY TRAFFIC FREQUENCY DISTRIBUTION

Average Daily Traffic	REE Loops				REE Outer Connections				LEE Outer Connections			
	Urban		Rural		Urban		Rural		Urban		Rural	
	No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent
0 to 499	228	21.0	599	70.3	288	18.6	809	59.3	3	2.5	27	26.7
500 to 999	226	20.8	177	20.7	317	20.4	353	25.9	15	12.3	33	32.7
1,000 to 1,499	141	13.0	54	6.3	239	15.4	111	8.1	21	17.2	10	9.9
1,500 to 1,999	158	14.6	8	0.9	202	13.0	38	2.8	17	13.9	7	6.9
2,000 to 2,499	94	8.7	5	0.6	122	7.9	36	2.6	9	7.4	3	3.0
2,500 to 2,999	46	4.2	6	0.7	90	5.8	13	1.0	6	4.9	3	3.0
3,000 to 3,499	55	5.1	0	0	53	3.4	2	0.1	3	2.5	6	5.9
3,500 to 3,999	42	3.9	1	0.1	67	4.3	1	0.07	2	1.6	4	4.0
4,000 to 4,499	35	3.2	0	0	46	3.0	0	0	2	1.6	4	4.0
4,500 to 4,999	13	1.2	1	0.1	23	1.5	1	0.07	1	0.8	1	1.0
5,000 to 5,499	12	1.1	0	0	31	2.0	0	0	4	3.3	1	1.0
5,500 to 5,999	4	0.4	0	0	12	0.8	0	0	2	1.6	1	1.0
6,000 to 6,499	4	0.4	1	0.1	12	0.8	0	0	6	4.9	1	1.0
6,500 to 6,999	8	0.7	0	0	12	0.8	0	0	3	2.5	0	0
7,000 and over	18	1.7	0	0	34	2.2	0	0	28	22.9	0	0
Total	1,084	100	852	100	1,548	100	1,364	100	122	100	101	100

characteristics of this study group probably are not representative of all LEE outer connections. In general, accidents of the 3 urban study groups occur mostly between 7 a.m. and 6 p.m.; most are 2-vehicle accidents, usually rear-end collisions. Accidents of the 3 rural study groups are much more likely to occur in the evening or at night, and to be single-vehicle accidents in which vehicles leave the road with or without hitting a structure. The 3 urban study groups have fewer accidents involving commercial vehicles (less than 10 percent) than the 3 rural study groups (more than 13 percent). This figure though, may only indicate that a greater percentage of rural traffic is commercial traffic. Unfortunately, the percentage of rural traffic that is commercial was not on the n-tuple tape, though it was in the original data base.

Figures 2 through 5 show accident, injury, fatality, and property damage rates respectively for each of 6 study groups for urban and rural areas. The accident rates are higher for ramps in urban areas than ramps in rural areas. The dotted lines indicate the various rates when all truck and bus traffic has been removed. The influence

TABLE 3

MAXIMUM CURVATURE FREQUENCY DISTRIBUTION

Curvature	REE Loops				REE Outer Connections				LEE Outer Connections			
	Urban		Rural		Urban		Rural		Urban		Rural	
	No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent
Tangent	—	—	—	—	111	7.2	58	4.3	15	12.3	17	16.8
0 deg 29 min and under	—	—	—	—	4	0.3	3	0.2	—	—	5	5.0
0 deg 30 min to 1 deg 29 min	—	—	—	—	11	0.7	4	0.3	11	9.0	2	2.0
1 deg 30 min to 2 deg 59 min	—	—	—	—	30	1.9	47	3.4	10	8.2	9	8.9
3 deg to 4 deg 59 min	3	0.3	2	0.2	48	3.1	65	4.8	21	17.2	36	35.6
5 deg to 8 deg 59 min	1	0.1	5	0.6	243	15.7	409	30.0	24	19.7	24	23.8
9 deg to 14 deg 59 min	42	3.9	35	4.1	351	22.6	373	27.3	14	11.5	4	4.0
15 deg to 23 deg 59 min	64	5.9	90	10.6	294	19.0	244	17.9	19	15.6	3	3.0
24 deg to 35 deg 59 min	271	25.0	287	45.4	191	12.3	113	8.3	6	4.9	—	—
36 deg and over	703	64.8	333	39.1	265	17.1	48	3.5	2	1.6	1	1.0
Total	1,084	100	852	100	1,548	100	1,364	100	122	100	101	100

TABLE 4

PERCENTAGE DISTRIBUTION OF ACCIDENT CHARACTERISTICS

Accident Characteristics	REE Loops		REE Outer Connections		LEE Outer Connections	
	Urban	Rural	Urban	Rural	Urban	Rural
Time of accident						
Night (12 to 7 a.m.)	15.8	34.1	12.6	23.6	12.6	28.6
Evening (6 to 12 p.m.)	16.0	23.0	17.3	20.0	24.3	28.6
Day (9 a.m. to 4 p.m.)	33.5	25.6	30.9	29.7	36.9	21.4
Rush (7 to 9 a.m. and 4 to 6 p.m.)	34.6	17.0	39.2	26.9	26.2	21.4
Manner of collision						
Rear-end	48.4	8.5	61.8	14.5	70.0	50.0
Head-on	0.3	2.4	0.1	0.7	1.0	0
Angle	1.3	1.2	2.7	2.1	1.0	14.0
Collision with pedestrian	0	0	0.2	0.7	0	0
Other collision	34.4	47.5	27.4	37.9	23.3	14.0
Noncollision	15.4	39.0	7.6	44.1	3.9	21.4
Not known	0	0	0	0	0	0
Vehicles per accident						
1 vehicle	49.3	87.8	35.1	82.7	27.2	35.7
2 vehicles	28.9	10.9	61.8	15.2	61.2	64.3
3 vehicles	1.3	2.4	3.2	2.1	10.7	0
4 vehicles	0.4	0	0	0	1.0	0
Ratio of vehicles to accidents	1.53	1.13	1.68	1.19	1.85	1.91
Type of vehicle involved						
Passenger cars	89.7	76.3	44.0	81.5	89.0	87.0
Pedestrian	0	0	0	0	0	0
Unknown or other	1.1	0	1.1	2.9	3.7	0
Trucks and buses	9.2	23.7	4.7	15.6	7.3	13.0
Total number of accidents	537	82	822	145	103	14

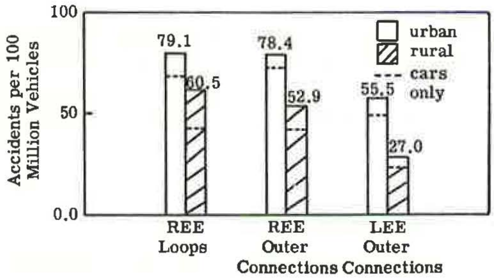

Figure 2. Accident rates.

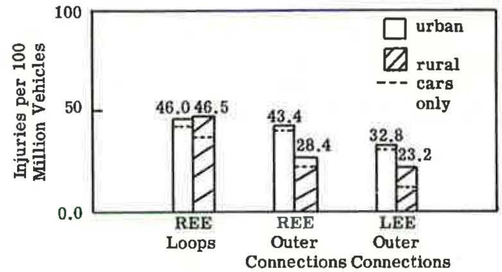

Figure 3. Injury rates.

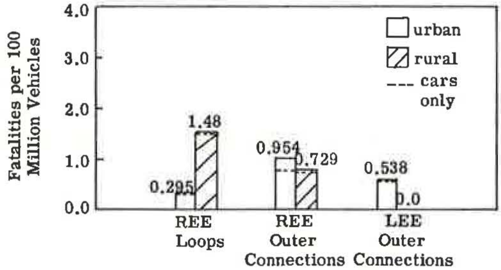

Figure 4. Fatality rates.

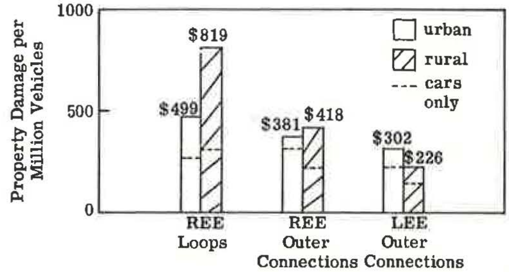

Figure 5. Property damage rates.

of trucks and buses on the fatality rates is very small, but on property damage rates, very large, especially on rural loops.

The importance of separating urban and rural data is that dividing lines for ADT and maximum curvature for the analyses to follow can be selected without bias. For example, if urban and rural data were not separated, the lower ADT level of 0 to 1,000 vehicles used in the analysis would contain mostly rural data points and the level of 1,001 to 4,000 would be mostly urban data points. Thus, a bias of rural data would exist in the lower ADT level of the example. Therefore, an urban-rural bias has been avoided by the realization of the difference in geometric variables, traffic variables, and accident characteristics between urban and rural ramps.

The data given in Tables 1 through 4 demonstrate and support the division of the ramp types into urban and rural. The data were also divided into left entering or exiting ramps and right entering or exiting ramps for the reason that there are so few left entering or exiting ramps that the ordinary driver has more difficulty negotiating them because he is less familiar with their traffic movement.

Separating the left and right entering or exiting ramps also allows each group to be more homogeneous and thus reduces possible bias. The only drawback from this separation is that it creates LEE categories for which there are few data points.

Maximum Curvature

Maximum curvature as defined in the data base is simply the greatest curvature that would be experienced by a driver as he drives his vehicle along the ramp. AASHO (3) defines maximum curvature as a "limiting value for a given design speed determined from the maximum rate of super-elevation and the maximum side friction factor." Thus, the 2 definitions of maximum curvature do not necessarily coincide.

Maximum curvature may have served as a proxy variable in this analysis for other important design variables such as minimum stopping sight distance or distance from approach nose to portion of the ramp where curvature exceeds 10 deg. These 2 particular variables were examined, in a similar manner to ADT, by using a computer program, but no correlation was found between their results and maximum curvature results. Also, other connections and loops with very high maximum curvature could not be isolated because of the nature of the data base that combines all ramps with curvatures above 36 deg. This is only a minor drawback of the data base, however, and does not affect these results.

Generally, each of the 6 study groups, except for REE rural loops, showed an increase in accident rate with an increase of maximum curvature (Fig. 6).

Outer connection ramps can be thought of as having curvature or no curvature. Table 5 gives a comparison of accident rates for various ADT levels for urban and rural outer connections with and without curvature. An outer connection was considered to have no curvature if its curvature was less than 1 deg. For each ADT level in urban and rural areas, except for 0 to 499 urban outer connections, the no-curvature ramps have smaller accident rates than those with curvature.

TABLE 5

ACCIDENT RATES ON OUTER CONNECTIONS BY CURVATURE AND ADT

ADT	Urban		Rural	
	Without Curvature (<1 deg)	With Curvature (>1 deg)	Without Curvature (<1 deg)	With Curvature (> 1 deg)
0 to 499	0.74	0.64	0	0.67
500 to 1,000	0.34	0.72	0.13	0.49
1,001 to 1,500	0.64	0.84	0	0.61
1,501 to 2,000	0.15	0.93	0[a]	0.20
2,001 and over	0.49	0.82	0[a]	0.72
All volumes	0.44	0.81	0.05	0.56

[a]Less than 10 study units.

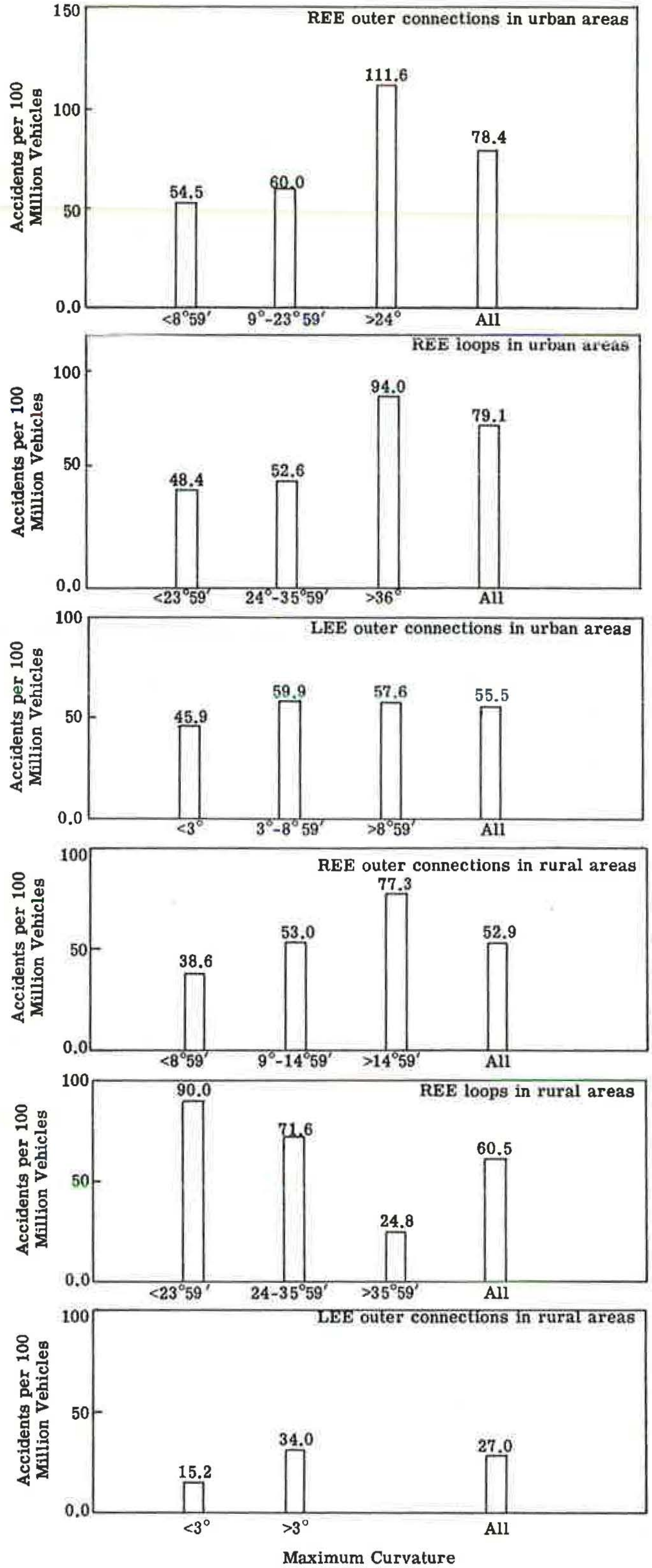

Figure 6. Accident rates for all traffic volumes by maximum curvature.

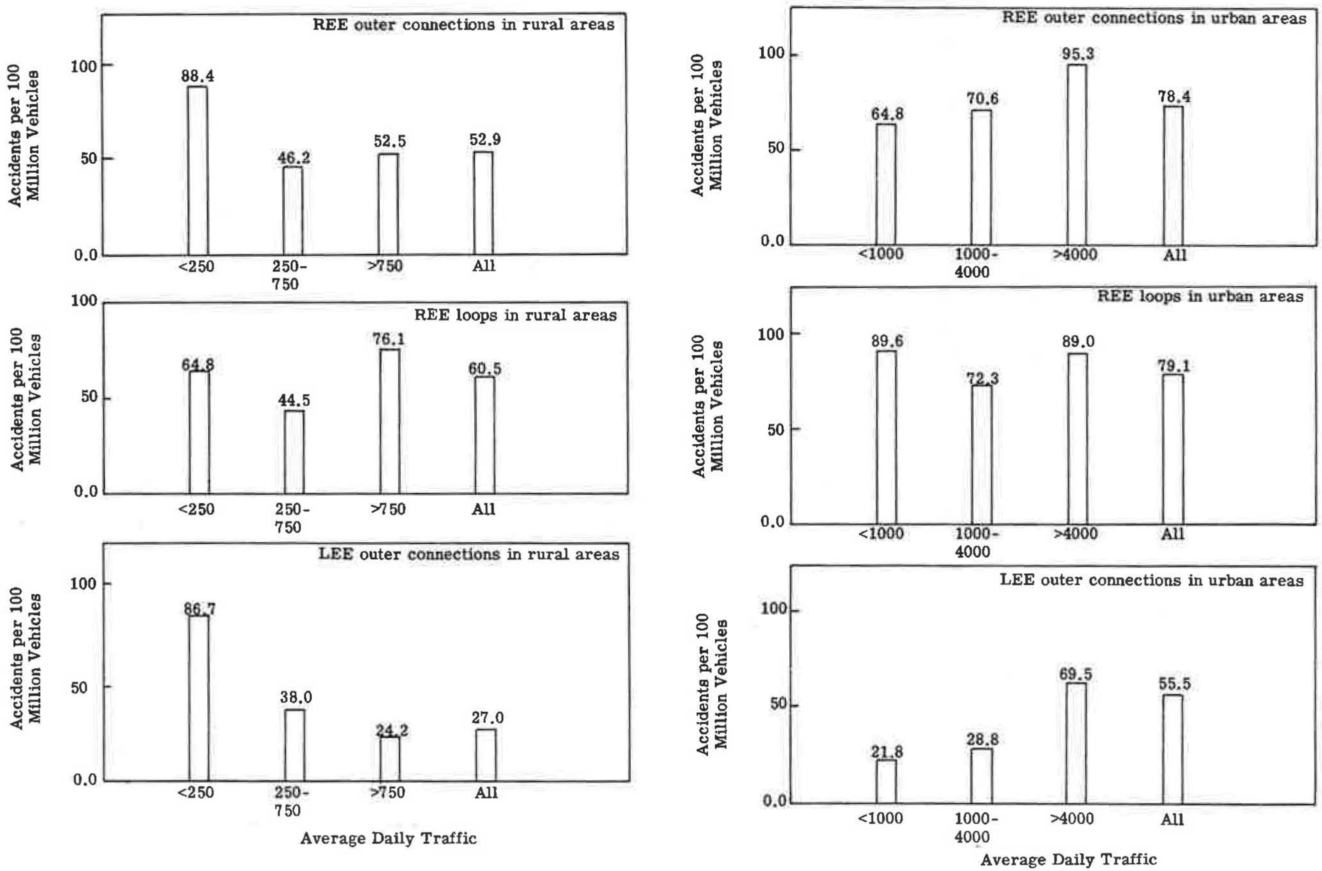

Figure 7. Accident rates for all degress of curvature by traffic volume.

TABLE 6

ACCIDENT RATES ON LOOPS BY CURVATURE AND ADT

ADT	Urban		Rural	
	Low Curvature (<12 deg)	High Curvature (>36 deg)	Low Curvature (<12 deg)	High Curvature (>36 deg)
0 to 499	0[a]	0.841	1.000	0.26
500 to 1,000	0[a]	0.960	0.810	0.37
1,001 to 1,500	1.320[a]	0.690	0[a]	0
1,501 to 2,000	0	0.720	0[a]	0
2,001 and over	0.141	1.000	—[a]	0
All volumes	0.200	0.940	0.631	0.25

[a]Less than 10 study units.

It is impossible to examine separately those ramps with extremely high curvature because of the design of the data base. This deficiency is especially felt for urban loop ramps for which 60 percent of the data are in the highest curvature category. Thus a comparison was made between accident rates on loops with low curvature (maximum

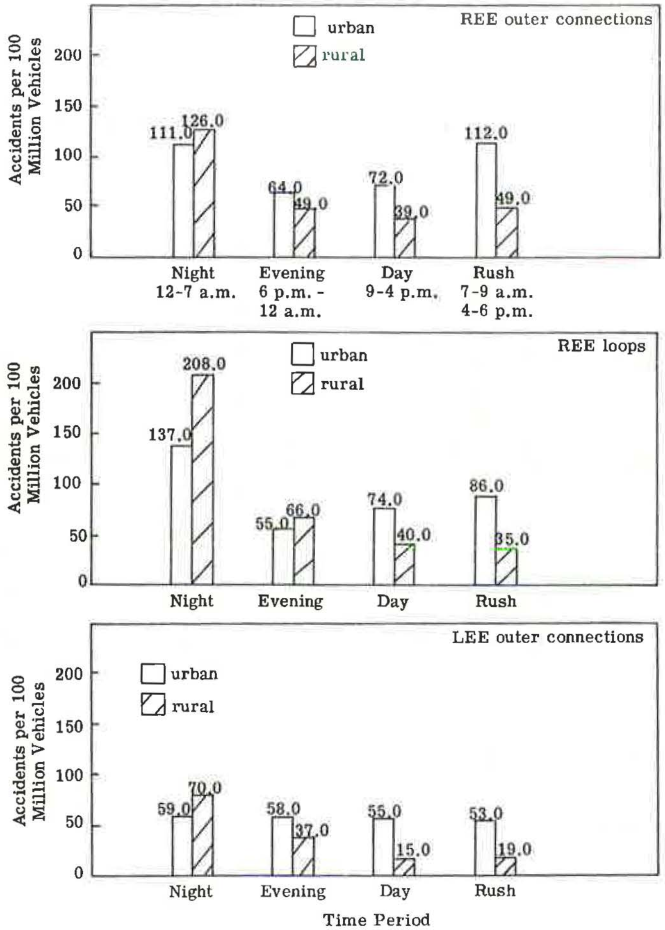

Figure 8. Accident rates by time period.

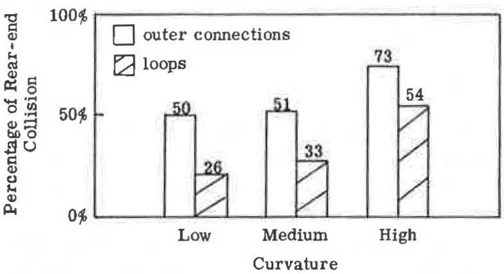

Figure 9. Percentage of rear-end collisions on REE outer connections and loops in urban areas by maximum curvature.

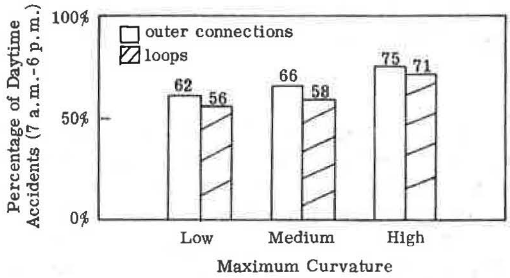

Figure 10. Percentage of daytime accidents on REE outer connections and loops in urban areas by maximum curvature.

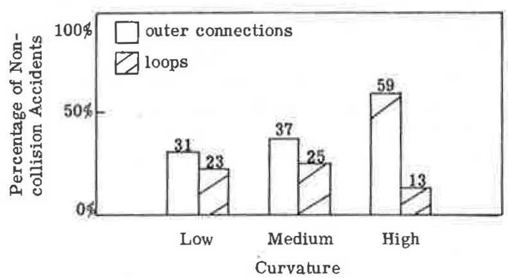

Figure 11. Percentage of noncollision accidents on REE outer connections and loops in rural areas by maximum curvature.

curvature of 12 deg or less) and loops with high curvature (maximum curvature of 36 deg or greater). Table 6 gives this comparison. In urban areas, high curvature loops have high accident rates for most ADT levels; in rural areas, low curvature loops have high accident rates for most ADT levels.

Average Daily Traffic

The 3 urban study groups are examined separately from the 3 rural study groups because of differences in ADT levels. The 3 urban study groups had ADT breakdowns of 0 to 999, 1,000 to 4,000, and 4,001 and over. The 3 rural study groups had ADT breakdowns of 0 to 249, 250 to 750, and 751 and over.

The assumption in the examination of ADT is that, as it increases, the accident rate also increases because of vehicle interaction. Thus, one would not expect the 3 rural study groups to demonstrate increasing accident rate with increasing ADT because the ADT levels are so very low that vehicle interaction is low even at high (751 and over) rural ADT levels. Figure 7 shows the relationship between ADT and accident rates for each of the 6 study groups. Because of low ADT levels, the 3 rural study groups show no relationship between accident rate and increasing ADT. Increasing ADT increases accident rates in urban areas for REE and LEE outer connections but not for loops.

Time Period

Accident rates for various time periods on the ramps in urban and rural areas are shown in Figure 8.

Accident Characteristics by Maximum Curvature

The accident characteristics of the primary study groups were examined to locate those characteristics that increased consistently with maximum curvature. In urban areas for all the primary study groups, the percentage of rear-end collisions and the percentage of day (7 a.m. to 6 p.m.) accidents increase with maximum curvature (Figs. 9 and 10). Similarly, in rural areas for REE outer connections, the percentage of noncollision accidents increases with increasing maximum curvature (Fig. 11). On

rural loops the percentage of noncollision accidents also increases for low and medium maximum curvatures, but decreases for high curvature.

CONCLUSION

In this examination of outer connection and loop ramps, data were separated into 8 groups: REE and LEE outer connections and loops in urban and rural areas. After the number of data points in each group were examined, 2 groups (urban and rural LEE loops) were discarded.

The remaining 6 groups were examined for relationships between accident rate and maximum curvature and ADT. At first, each group was inspected individually for optimum data groupings for the analysis. Results indicated that (a) except for loops, in rural areas, all REE ramps showed an increase in accident rate with increasing maximum curvature, and (b) all ramps in urban areas, except urban loops, showed an increasing accident rate with increasing ADT.

Curvature versus no curvature was examined for outer connections and low curvature versus high curvature for loops. In both analyses, comparable urban-rural ADT levels were used. Outer connections without curvature have smaller accident rates than those with curvature in both urban and rural areas for all ADT levels except 0 to 499 on outer connections in urban areas. Unfortunately, the design of the data base prevented a true comparison of loops with low curvature with loops with high curvature. But, results showed that rural loops with low curvature had higher accident rates than rural loops with high curvature whereas the reverse was true for urban loops. The percentage of rear-end collisions and the percentage of day (9 a.m. to 4 p.m.) accidents also increased with maximum curvature on REE urban loops and outer connections.

ACKNOWLEDGMENT

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GLOSSARY

accident rate. Accidents per 100 million vehicles.

average daily traffic. Total traffic volume during a stated period divided by the number of days in that period. Unless otherwise stated, the period is a year. Abbreviation ADT.

commercial vehicle. A motor vehicle designed primarily for transportation of goods or more than 10 passengers or both in connection with business, industry, agriculture, public service, or the exploitation of natural resources.

criterion variable. Variable used as a measure of relative safety. In this study, accident rate is used as the criterion variable. Also called response variable.

data point. An n-dimensional point whose coordinates represent the criterion variable and the various independent variables. A new data point is produced for each year that a particular unit is studied.

injury rate. Injuries per million vehicles unless specifically noted.

left entering or exiting ramp. Ramp that exits or enters on the left side of the mainline roadway looking in the direction of the traffic flow. A left entering or exiting outer connection or loop almost always requires a left-turning movement. Abbreviation LEE.

loop ramp. A one-way turning roadway that curves about 270 deg to the right to accommodate a left-turning movement. It may include provision for a left turn at a terminal to accommodate another turning movement (AASHO). Also called loop.

maximum curvature. The greatest degree of curvature that is experienced by a driver negotiating a study unit, which in this paper, are loop and outer connection ramps.

noncollision accident. An accident in which a vehicle leaves the road but does not strike any object.

other collision accident. An accident in which a vehicle leaves the road and strikes an object without having hit another vehicle or object on the roadway.

outer connection ramp. A one-way turning roadway primarily for a right-turning movement. It may include provision for a left turn at a terminal to accommodate another turning movement (AASHO). In this study, diamond interchange ramps that satisfy this AASHO definition were not included in this category. Also called outer connection.

primary study groups. The following 4 study groups: urban right entering or exiting loops, urban right entering or exiting outer connections, rural right entering or exiting loops, and rural right entering or exiting outer connections.

right entering or exiting ramp. Ramp that exits or enters on the right side of the mainline roadway looking in the direction of the traffic flow. A right entering or exiting outer connection or loop nearly always requires a right-turing movement. Abbreviation REE.

rural. Any area not classified as a suburban area or an urban place, except an urban place with a population less than 5,000.

study groups. The 6 individual groups of data studied. The 2 ramp types, loop and outer connection, divided into urban or rural and right or left entering or exiting make 8 possible study groups, but, because left entering and exiting loops lacked sufficient data, the number was reduced to six.

urban. Urban places with populations equal to or greater than 5,000 and suburban areas surrounding urban places.

Visibility Problems in Crest Vertical Curves

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The length of a crest vertical curve is governed by visibility considerations. The minimum length is based on the stopping sight distance; the maximum length is based on the passing sight distance, and overtaking is allowed throughout its length. The object of the present paper is theoretical determination of the zone of overtaking visibility in a curve designed on a below-maximum basis. The analysis covers 2 cases: (a) overtaking vehicle inside and oncoming vehicle outside the curve and (b) both vehicles outside the curve. The corresponding curve geometries were also considered. The equations obtained were computer-solved for curves with slope difference ranging from 2 to 12 percent, passing sight distances corresponding to the design speed range of 50 to 110 km/hr, and length limits corresponding to the stopping and passing sight distances respectively. Results were rendered in convenient graph form, permitting determination of the type of division line and the length of the no-overtaking zone to be marked on a 2-way 2-lane highway in the vicinity of the curve. The length of the no-overtaking zone increases with the overall length of the curve, up to the maximum (unrestricted overtaking). The conclusion is that, in order to reduce the no-overtaking zone in below-maximum cases, it should preferably be as short as possible within the requirement limits of overtaking visibility and driving convenience.

•CONVENTIONAL DESIGN of a crest vertical curve (hereinafter referred to simply as curve) in a 2-way 2-lane highway involves 2 extreme cases: minimal, with overtaking prohibited altogether, and maximal, with overtaking allowed throughout its length, based respectively on the sight distance required for stopping and passing (overtaking) at a given speed. Even in a minimal curve a point exists beyond which visibility is insufficient for overtaking. The design is frequently based on a criterion radius that generally satisfies this minimal requirement, and overtaking is allowed along part of the curve in such cases. A solid division line throughout the curve and beyond it would be unrealistic and make every driver a potential offender, as is bound to happen wherever overtaking is prohibited along obviously safe stretches. Removal of the overtaking restriction is indicated on the highway by a broken line parallel to the solid line, and advance notice of the reduced sight distance is indicated by a special closely spaced broken line (warning line) equal in length to the passing sight distance and originating at the point beyond which there is no overtaking visibility. A vehicle that starts to overtake short of this point will be able to complete the move; after this point, the warning line is equivalent to the solid line.

The object of this paper is the theoretical determination of the different zones from the viewpoint of overtaking safety. This was done graphically or by means of practical field tests in situ.

Paper sponsored by Committee on Operational Effects of Geometrics.

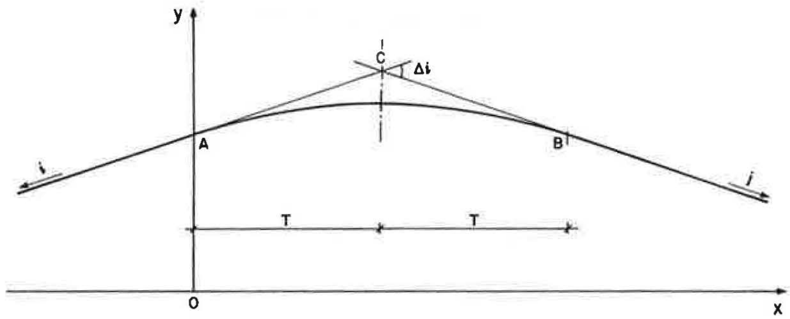

Figure 1. Parabolic crest vertical curve.

DETERMINATION OF VISIBILITY ZONES

The design is generally based on a parabolic curve (Fig. 1) with the following equation:

$$y = -\frac{\Delta i}{4T} x^2 + ix + H_A \tag{1a}$$

$$\Delta i = i - j \tag{1b}$$

where

y = elevation of a point on the curve;
x = distance of the point from the origin of the coordinate system;
Δi = algebraic difference of the tangential slopes i and j;
$2T$ = length of the curve in plan; and
H_A = elevation of point A, the initial point of the curve.

In the range of longitudinal slopes used in road design, it can be shown that the radius of curvature is constant at close approximation:

$$1/R = \frac{y''}{\left(1 + y'^2\right)^{3/2}} \tag{2}$$

Because the slope y' is relatively small, y'^2 in the denominator is negligible with respect to unity, and we have

$$R \cong 1/y'' = \frac{2T}{\Delta i} \tag{3}$$

An additional assumption is that the true length and projection of a sloping line are equal. In these circumstances, the results would not be affected by slight rotation of the system. Accordingly, the absolute values of slopes i and j are immaterial in the following analysis; the only significant parameters are h, level of driver's eyes above the ground, and the difference Δi. The sight distances in both directions are symmetric with respect to the intersection point C of the 2 tangents.

In determining the driver's sight distance with respect to an oncoming vehicle, the 4 cases given in Table 1 should be distinguished. According to the German Code (1), h equals the height of the oncoming car, so that cases 1a and 2b are identical.

Case 1a

Case 1a, in which the overtaking vehicle is inside and the oncoming vehicle is outside the curve, is shown in Figure 2. The parabola equation is used to obtain the elevation

of point E′, driver's eye level in the overtaking car, which is

$$H_{E'} = H_A + ix - \frac{\Delta i}{4T} x^2 + h \qquad (4)$$

The slope of the line of sight, y′, is

$$y' = -\frac{\Delta i}{2T}(x + m) + i \qquad (5a)$$

and

$$m = \sqrt{\frac{4T}{\Delta i} h} \qquad (5b)$$

m equals at least the stopping sight distance in the minimal case. The elevation of point D′, level of roof of oncoming vehicle, obtained from the slope of the sight line from Eq. 5a is given by

$$H_{D'} = H_{E'} + S_p y' = H_A + ix - \frac{\Delta i}{4T} x^2 + h + S_p \left[i - \frac{\Delta i}{2T}(x + m) \right] \qquad (6)$$

where S_p is the passing sight distance. The same elevation obtained from the parabola equation is

$$H_{D'} = H_A + Ti + (S_p + x - T) j + h \qquad (7)$$

TABLE 1

CASES USED IN DETERMINING SIGHT DISTANCE

Case	Vehicle	Position in Curve	Case	Vehicle	Position in Curve
1a	Overtaking Oncoming	Inside Outside	2a	Overtaking Oncoming	Outside Outside
1b[a]	Overtaking Oncoming	Inside Inside	2b	Overtaking Oncoming	Outside Inside

[a]For a curve not based on passing sight distance, case 1b is irrelevant.

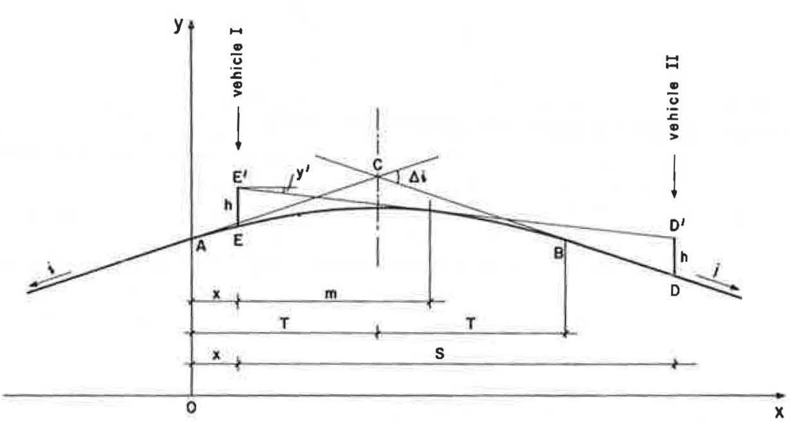

Figure 2. Sight line—one vehicle inside and one vehicle outside curve.

Equating both expressions, we have

$$\frac{\Delta i}{4T}x^2 + \left(\frac{\Delta i}{2T}S_p - \Delta i\right)x + \left[\left(T - S_p\right)\Delta i + \sqrt{\frac{\Delta i h}{T}}\,S_p\right] = 0 \tag{8}$$

For a given curve with known Δi and 2T and for S_p corresponding to a given design speed, Eq. 8 yields x, the coordinate of the point beyond which there is sufficient visibility for overtaking. Of the 2 roots of the quadratic equation, only the one satisfying the following 3 conditions is relevant:

$$x \geq 0 \tag{9a}$$

$$(x + m) \leq 2T \tag{9b}$$

$$(x + S_p) \geq 2T \tag{9c}$$

Case 2a

Case 2a, in which both vehicles are outside the curve, is shown in Figure 3. This situation exists only for $S_p > 2T$. As the diagram shows,

$$S_p = v + m + n + w = v + T + w \tag{10}$$

where

v = the distance from the vehicle situated outside the curve to the intersection point F of the line of sight of the vehicle with the grade line of the beginning of the curve (Fig. 3), and

w = the corresponding distance when the vehicle is on the other side of the curve.

The slope of the sight line is

$$y' = -\frac{\Delta i}{2T}x + i \tag{11}$$

The angles between tangent and sight line are $(i - y')$ and $(-j + y')$ respectively, and we have

$$v = \frac{h}{i - y'} = \frac{h}{x/R} \tag{12}$$

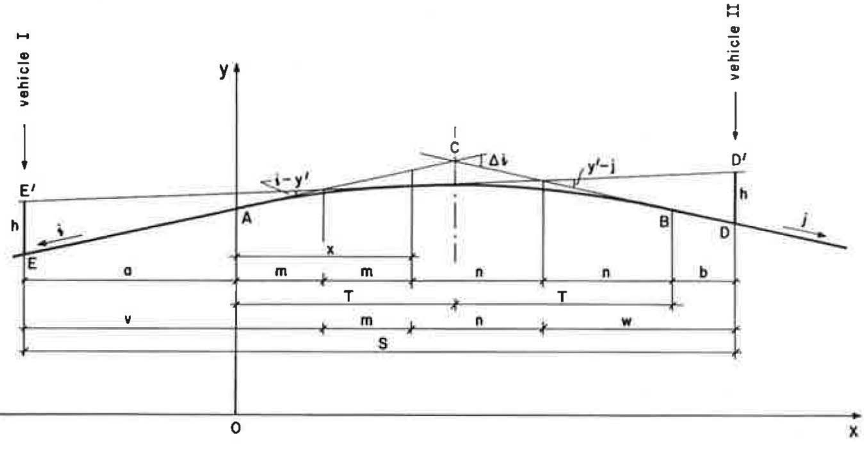

Figure 3. Sight line—both vehicles outside curve.

$$w = \frac{h}{-j + y'} = \frac{h}{-x/R + \Delta i} \tag{13}$$

Where R is the curve radius. Substituting these in Eq. 10, we have

$$(2S_p - R\Delta i)x^2 - R\Delta i(2S_p - R\Delta i)x + 2R^2h\Delta i = 0 \tag{14}$$

x_1 and x_2, the roots of Eq. 14, are symmetrical with respect to the vertical through C, namely,

$$x_1 = 2T - x_2 \tag{15}$$

The positions of the vehicles in the coordinate system are

$$a = v - m = \frac{Rh}{x} - \frac{x}{2} \tag{16}$$

for the overtaking vehicle, measured from the initial point of the curve, and

$$b = S_p - a - 2T \tag{17}$$

for the oncoming vehicle, measured from the end point of the curve.

Identification of Relevant Case

Case 1a, one vehicle inside and one vehicle outside the curve, is represented by Eq. 8. For this case, one of the solutions must obey conditions in Eqs. 9a, 9b, and 9c. Case 2a, both vehicles outside the curve, is represented by Eq. 14. For this case, both a and b, obtained from Eqs. 16 and 17, must be positive. Exclusion of one case indicates validity of the other.

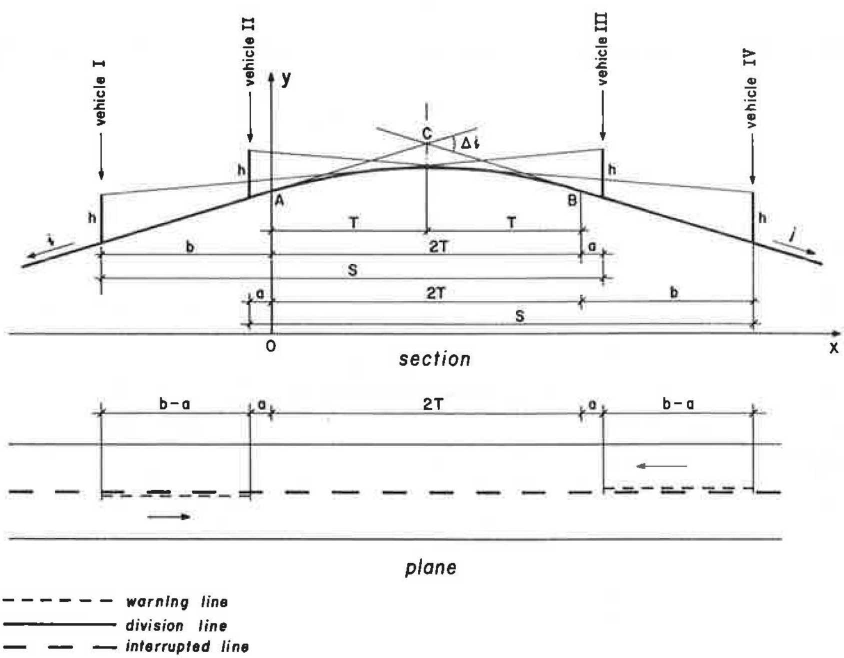

Figure 4. Determination of no-overtaking zone—both vehicles outside curve.

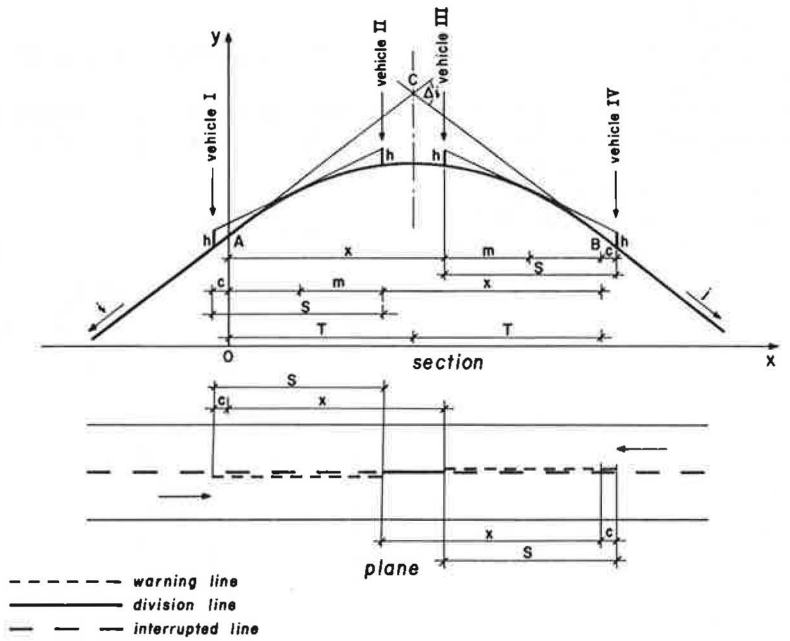

Figure 5. Determination of no-overtaking zone—one vehicle inside and one vehicle outside curve.

Marking of Division Line

Figure 4 shows the lines for case 2a, both vehicles outside the curve. For reasons of symmetry, a and b determine the length of the no-overtaking zone on both sides of the curve. This case is confined to a small angle Δi.

Figure 5 shows the lines for case 1a, one vehicle outside and one vehicle inside the curve. The distance x is obtained from Eq. 8, and the initial point of the division line is given by

$$c = S_p + x - 2T \tag{18}$$

where c is the position of the oncoming vehicle, measured from the end point of the curve.

TABLE 2

STOPPING AND PASSING SIGHT DISTANCE ACCORDING TO DESIGN SPEED

Design Speed (km/hr)	Passing Sight Distance (m)	Stopping Sight Distance (m)
50	340	60
65	460	85
80	550	110
95	640	145
105	700	170
110	760	185
130	820	230

COMPUTATION DATA

Equations 8 and 14 were solved on an Elliott 503 computer for the following i values: 0.02, 0.04, 0.06, 0.08, 0.10, and 0.12. Sight distance or S-values age given in Table 2 (2).

The curve length corresponding to each Δi and S was determined for h = 1.20 m, the elevation of an object on the road being taken as zero. 2T values ranged from $2T_s$ for stopping to $2T_p$ for passing and are shown at 100-m intervals in Figures 6 through 11. For given Δi, 2T, and S_p, the corresponding x is found subject to the relevant conditions. Results of Eq. 14 were not plotted because of the low frequency of the cases involved.

Numerical Example 1

Given $\Delta i = 0.08$, $2T = 1{,}600$ m, $R = 20{,}000$ m, design speed $v = 95$ km/hr (corresponding to $S_S = 145$ m, $S_P = 640$ m).

The length of the curve based on stopping sight distance is $2T_S = 700$ m; and the length based on permitting overtaking throughout the curve is $2T_P = 3{,}413$ m. As $S_P < 2T$, the relevant case is that of Eq. 8.

The data shown in Figure 9 yield $x = 1{,}320$ m, $c = 1{,}320 + 640 - 1{,}600 = 360$ m, and the resulting marking scheme is shown

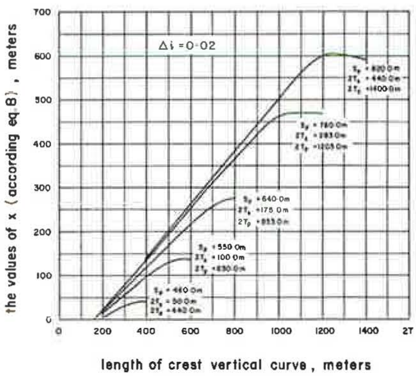

Figure 6. Determination of x values for S_P distances, various lengths of vertical curve 2T, and for $\Delta i = 0.02$.

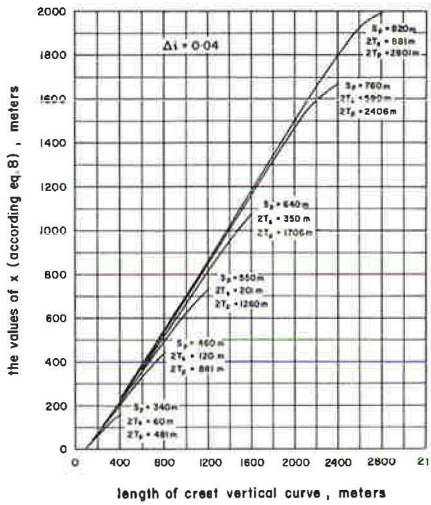

Figure 7. Determination of x values for various S_P distances, various lengths of vertical curve 2T, and for $\Delta i = 0.04$.

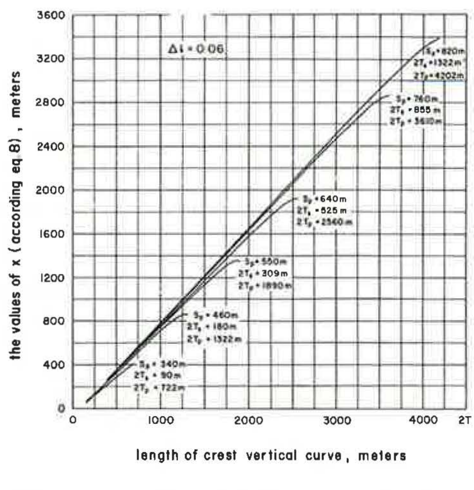

Figure 8. Determination of x values for various S_P distances, various lengths of vertical curve 2T, and for $\Delta i = 0.06$.

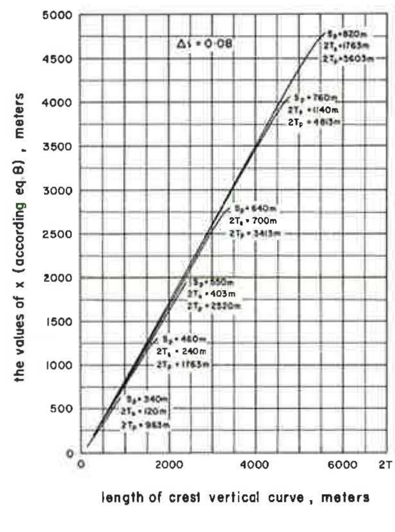

Figure 9. Determination of x values for various S_P distances, various lengths of vertical curve 2T, and for $\Delta i = 0.08$.

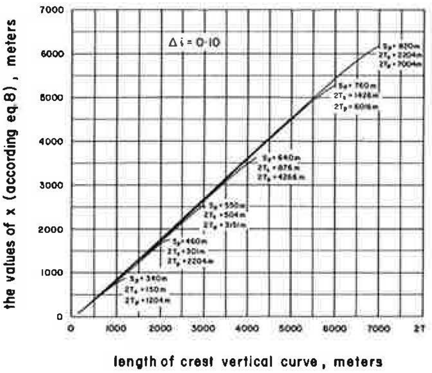

Figure 10. Determination of x values for various S_p distances, various lengths of vertical curve 2T, and for $\Delta i = 0.10$.

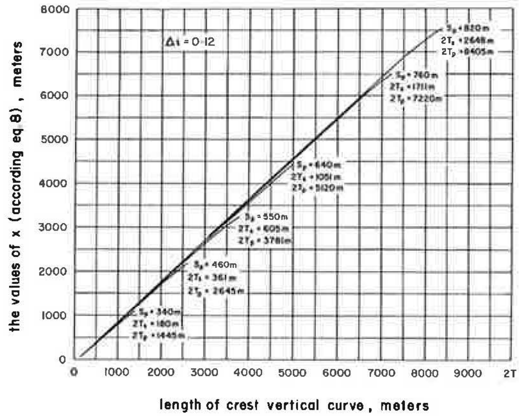

Figure 11. Determination of x values for various S_p distances, various lengths of vertical curve 2T, and for $\Delta i = 0.12$.

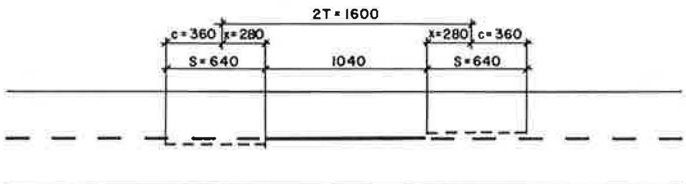

Figure 12. Example of proposed marking.

in Figure 12. For the same data and R = 25,000 m, we have 2T = 2,000 m and, accordingly, x = 1,700 m, c = 1,700 + 640 - 2,000 = 340 m. The corresponding length of the division line is 1,400 m, compared with 1,040 m for R = 20,000 m. The radius permitting unrestricted overtaking under these conditions is 42,667 m.

<u>Numerical Example 2</u>

Given $\Delta i = 0.04$, 2T = 800 m, R = 20,000 m, design speed v = 80 km/hr (corresponding to $S_s = 110$ m, $S_p = 550$ m).

The length of the curve based on stopping sight distance is $2T_s = 202$ m; and the length based on permitting unrestricted overtaking throughout the curve is $2T_p = 1{,}260$ m. As $S_p < 2T$, the relevant case is again that of Eq. 8.

The data shown in Figure 7 yield x = 1,320 m, c = 356 + 550 - 600 = 306 m. For the same data and a smaller radius R = 10,000 m, we have 2T = 400 m. Now $S_p > 2T$, and Eq. 14 is relevant; its roots are $x_1 = 38$ m and $x_2 = 362$ m. For x_1, Eqs. 16 and 17 yield a = 297 m and b = -187 m, a negative value, whereas it is required to be positive. Hence, the case of both vehicles outside the curve does not apply here, and the division line is marked as for the case with one vehicle outside and the other inside the curve.

SUMMARY AND CONCLUSIONS

This analysis permits determination of the zone of overtaking visibility for a crest vertical curve in a 2-way 2-lane highway. The proposed marking (Figs. 4 and 5) comprises a warning line, equal in length to the passing sight distance, indicating nearness of a zone of reduced overtaking visibility. In this zone the driver is allowed to complete an overtaking move begun earlier but not allowed to attempt a new one once he has passed the initial point of the warning line in the right lane. Analysis of the results

shows that the length of the no-overtaking zone increases with that of the curve, up to the maximum where overtaking is unrestricted.

The sudden vanishing of the division line in the diagrams is explained as follows: As 2T tends to $2T_p$, x tends to $2T - S_p$. In other words, the solution tends to the case of the oncoming vehicle near the end point of the curve and the other within the curve at distance x. At $2T = 2T_s$, $x = 2T - S_p$; in other words, one vehicle is at the end point of the curve and the other at a distance S_p from it. This corresponds to the case of both vehicles inside the curve with overtaking visibility throughout its length. Both x and $2T - x$ increase as 2T increases; $c = S_p - (2T - x)$ decreases accordingly; but as the rate of increase of x is steeper than that of c, the length of the division line increases with 2T.

The conclusion is that, in order to reduce the no-overtaking zone where the design cannot be based on the passing sight distance, the curve should be as short as possible but still comply with the requirements of stopping sight distance and driving convenience. The curve length complying with the latter aspect is obtainable, for example, from

$$\frac{v^2}{R} = 1 \text{ ft/sec}^2 \qquad (19)$$

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2. A Policy on Geometric Design of Rural Highways. American Association of State Highway Officials, 1965.

Relationship Between Lighting and Accident Experience Between Interchanges

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This paper examines the relationship between accident experience and lighting between interchanges of the Interstate Highway System in urban areas. Characteristics considered include existence of lighting, number of lanes, intensity of lighting, and average daily traffic volume. Accidents per hundred million vehicle-miles of travel are used as the primary indicator of accident experience. Results indicate that 24-hour accident rates and night (6 p.m. to 4 a.m.) accident rates are higher on lighted highways than on unlighted highways regardless of whether mainline units are classed by number of lanes or by ADT levels. Averages of various geometric, traffic, and accident characteristics for lighted and for unlighted units were compared and discrepancies noted. There is no discernible relationship between lighting intensity and accident rate on 2-lane mainline units or 3-lane mainline units.

•THE CONTRIBUTION OF LIGHTING on the Interstate Highway System between interchanges has been a subject of some recent controversy. This study was proposed to better assess the contribution of lighting in this area. The Interstate System Accident Research, Study II (ISAR-II), data base was used. This study is confined to between-interchange study units (also called mainline units) on Interstate highways in urban areas. All bridges, tunnels, and other overpasses and underpasses are excluded. All of the Interstate highways examined had from 2 to 4 traffic lanes in one direction. Lighting intensity in the data varies from 0 to 1.05 foot-candles (ft-c). The existence of lighting is studied with respect to number of lanes (2 to 4) and average daily traffic (ADT) per lane (1,000 to 10,000). The relationship between lighting intensity and accident rate is briefly examined.

FINDINGS

1. Approximately 93 percent of the lighted mainline units studied are illuminated by mercury vapor lamps.
2. The percentage of lighted mainline units increases with number of lanes; 11 percent of 2-lane units, 50 percent of 3-lane units, and 100 percent of 4-lane units are lighted.
3. During darkness (9 p.m. to 4 a.m.) when artificial lighting should be most effective, the accident rate is much higher on lighted units than on unlighted units regardless of number of lanes.
4. Comparison of lighted and unlighted units by ADT per lane, where sufficient data are available, demonstrates that lighted highways have higher accident rates.
5. There is no discernible relationship between lighting intensity and accident rate on 2-lane or 3-lane mainline units.
6. The lighted mainline units appear to be more constraining on the vehicles that move along them than unlighted units are. This is seen in the low percentage of noncollision accidents and higher percentage of rear-end collisions on lighted units. The

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obstructions along the road that cause a constraining highway are, of course, lighting poles.

7. For both lighted and unlighted units, injury rates and property damage rates are consistently and highly correlated with accident rates.

ANALYSIS

Data Characteristics

The level of lighting intensity in the ISAR-II data base is a discrete variable as follows:

Intensity Level (ft-c)	Code	Assigned Value
None	0	0
0 to 0.3	1	0.15
0.3 to 0.49	2	0.40
0.5 to 0.79	3	0.70
0.8 to 1.29	4	1.05
1.3 to 1.99	5	1.65
2.0 and over	6	2.00
Partial lighting	7	—

All partial lighting study units were discarded because partial lighting is not a definitive term and thus would be difficult to evaluate. Lighting effects were only examined on between-interchange units excluding overpasses, underpasses, or major river crossings (i.e., any bridge with a span of more than 20 ft). The between-interchange units studied varied in number of lanes in one direction from 2 to 4 lanes. In this paper, highways having 2, 3, and 4 lanes in one direction are referred to as 2-, 3-, and 4-lane mainline units.

The number of data points available are as follows:

	Urban Units		Rural Units	
No. Lanes	No.	Percent With Lighting	No.	Percent With Lighting
2	5,642	8.9	24,169	0.3
3	2,671	58.0	765	?
4	60	100.0		

Because 99 percent of the 2-lane rural data had no lighting, this study was confined to urban data points only. Figures 1 and 2 show the type of lighting used and the average lighting pole spacing in the mainline units studied.

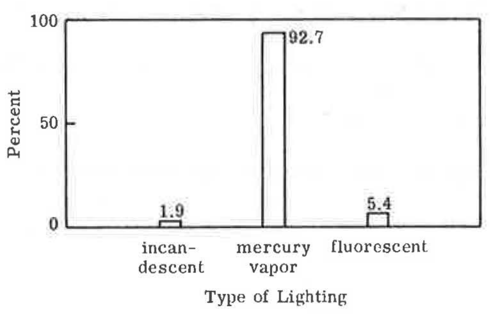

Figure 1. Type of lighting.

Response Variable

A rate based on vehicle-miles was used to avoid bias caused by ADT or unit length. Accident rate is the number of accidents per hundred million vehicle-miles. Although accident rate served as the response variable, injury, fatality, and property damage rates were also computed. Property damage rates and injury rates simulated accident rates consistently and accurately (Fig. 4).

Accident Rate by Lighting Intensity on 2- and 3-Lane Mainline Units

Figure 3 shows accident rate by lighting intensity on mainline units of 2 and 3 lanes. Three-lane mainline units have high accident

rates in each of the lighting intensity categories relative to those 3-lane units without lighting. For 2-lane mainline units, the 2 lighting intensity categories, 0.01 to 0.29 and 0.30 to 0.49, should be ignored because few data points were available. Otherwise, the 2 highest lighting categories have accident rates slightly lower than those 2-lane mainline units without lighting. From these data, no relationship can be discerned between accident rate and lighting intensity. That is, accident rate did not increase or decrease consistently with increasing lighting intensity.

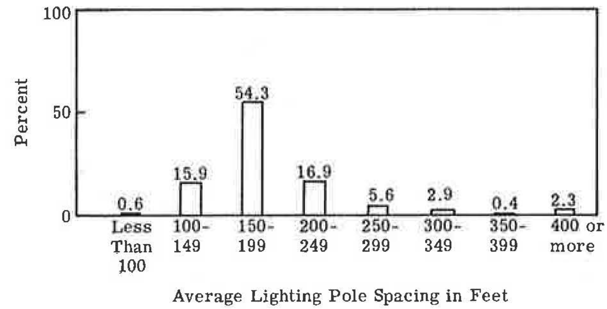

Figure 2. Average spacing of lighting poles.

Accident Rate by Existence of Lighting and Number of Lanes

All data were divided into 3 groups (2-lane, 3-lane, and 4-lane mainline units) to enable comparisons to be made. In each group, those units with lighting were separated from those without lighting, and accident rates were computed. As shown in Figure 4, only on 2-lane mainline units was the accident rate higher on unlighted units than on lighted units. Because the accident rates shown in Figure 4 were calculated for the entire 24-hour day, perhaps a closer examination of periods of the day could show other results. The periods of the day for which accident rates were computed are as follows:

Day	9 a.m. to 4 p.m.
Rush	7 to 9 a.m. and 4 to 6 p.m.
Dawn	4 to 7 a.m.
Dusk	6 to 9 p.m.
Night	9 p.m. to 4 a.m.

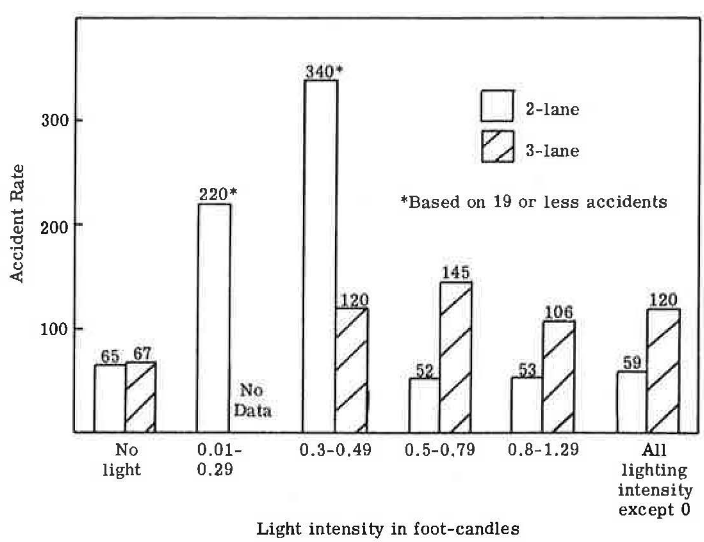

Figure 3. Accident rate by light intensity on 2- and 3-lane mainline units.

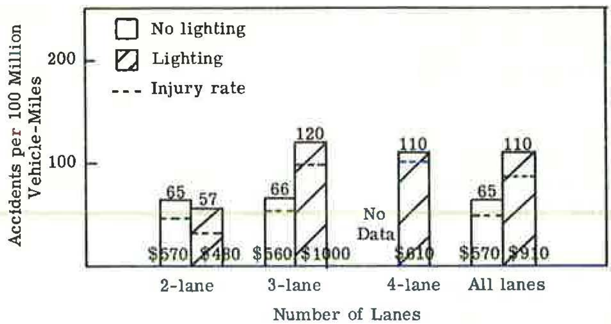

Figure 4. Accident rate by number of lanes.

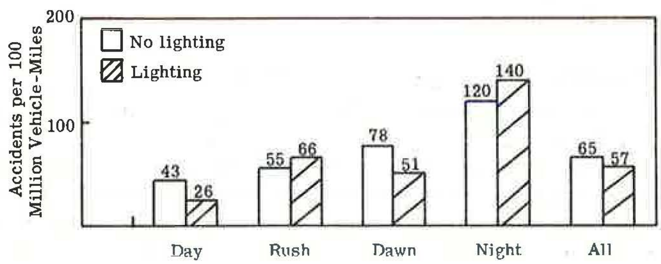

Figure 5. Accident rate by period of day on 2-lane mainline units.

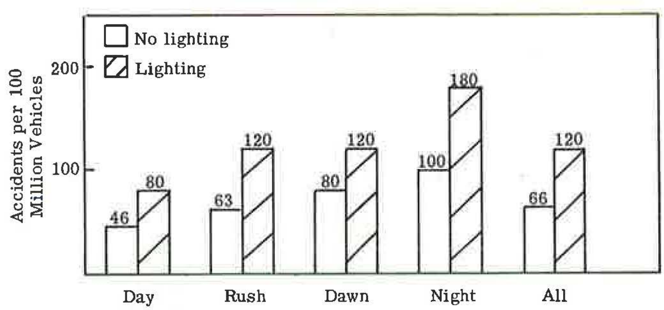

Figure 6. Accident rate by period of day on 3-lane mainline units.

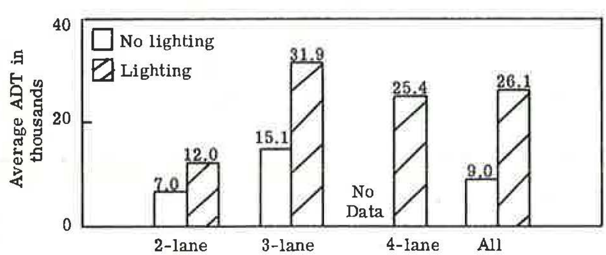

Figure 7. ADT by number of lanes.

These particular groupings were chosen because the data base had traffic volume and number of accidents separately for each of them and, therefore, accident rates could be developed. This breakdown by periods of the day was not done for 4-lane mainline units because of limited data. Figures 5 and 6 show the accident rates by time periods for 2- and 3-lane mainline units respectively. Of the 4 time periods, only day and night indicate the light conditions; day is always light and night is always dark. One would expect to find the greatest effect of lighting during the night (i.e., in darkness from 9 p.m. to 4 a.m.). In both 2-lane and 3-lane mainline units, however, the accident rate is higher on the lighted units during the night period. Of all the time periods, only day, dusk, and dawn on 2-lane units had a higher accident rate on unlighted units than on lighted units. On 3-lane mainline units, the accident rate on lighted units is nearly double the accident rate on unlighted units for each time period.

Figure 7 shows that the lighted units have a much higher ADT than unlighted units regardless of the number of lanes. This was expected because of the results of an informal survey of the lighting policies used by 5 states that contributed heavily to the ISAR-II data. Four of the 5 states have policies that require urban highways to have lights when the ADT levels warrant. In most cases, state planning manuals or AASHO (1) was cited as providing standards for lighting policies. Other reasons given for lighting an urban highway were (a) substantial commercial or industrial building development and (b) 3 or more successive interchanges with an average spacing of $1\frac{1}{2}$ miles or less. None of the 5 states claimed to use accident rate as an indicator in lighting urban highways, though by using ADT they indirectly used accident rate because the ADT level is the most important predictor of accident rate (2).

Accident Rate by Existence of Lighting and ADT Levels per Lane

A comparison was made of mainline units with and without lighting by ADT levels per lane. For each mainline unit, the ADT per lane is computed and then placed in one of the following 10 categories:

0 to 1,000
1,001 to 2,000
2,001 to 3,000
3,001 to 4,000
4,001 to 5,000
5,001 to 6,000
6,001 to 7,000
7,001 to 8,000
8,001 to 9,000
9,001 and over

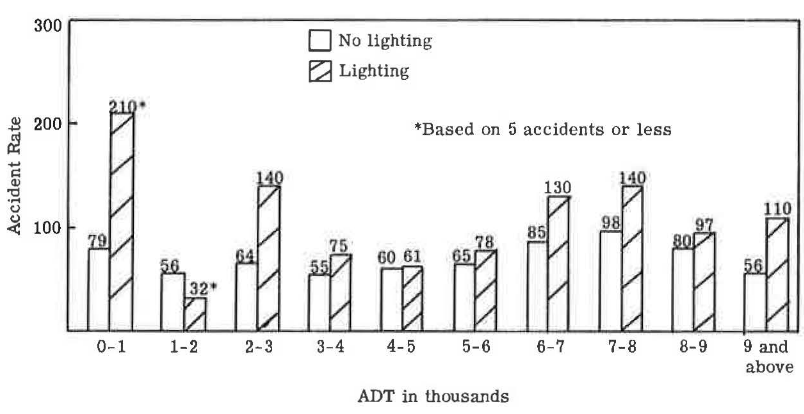

Figure 8. Accident rate by ADT per lane.

In each group, distinction is made between a lighted unit and an unlighted unit. Figure 8 shows the comparisons of accident rates on lighted and unlighted units by ADT per lane. As ADT per lane increases, accident rates for lighted mainline units and for unlighted mainline units are lowest in the 3,000 to 5,000 vehicle-per-lane (VPL) category and peak in the 7,000 to 8,000 VPL category. That is, accident rate on lighted mainline units and accident rate on unlighted mainline units increase with increasing ADT per lane only in the 4,000 to 8,000 VPL categories.

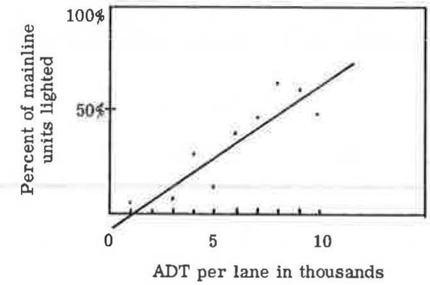

Figure 9. Mainline units lighted and ADT per lane.

The data shown in Figure 9 support the results of the informal survey mentioned earlier that urban mainline highways are lighted principally by ADT level. The percentage of mainline units lighted increases with ADT. The line shown was fit by simple linear regression. The correlation coefficient between percentage of mainline units lighted and ADT per lane is 0.91, which implies that they are highly correlated.

Two of the accident rates are higher on lighted units than on unlighted units for all ADT levels per lane except the first 2 levels. Each of these ADT levels per lane was

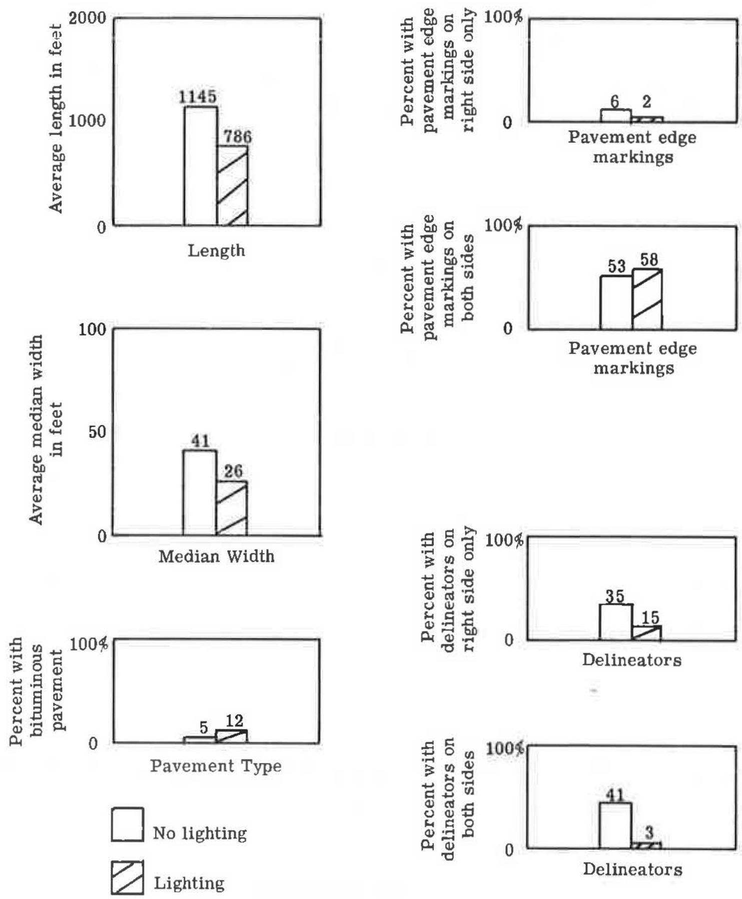

Figure 10. Geometric variables.

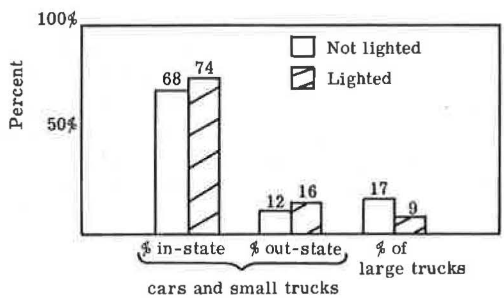

Figure 11. Traffic variables.

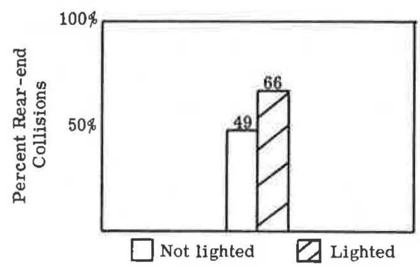

Figure 12. Rear-end collisions.

examined by period of the day. Again, during the night period, accident rates were much higher on lighted units than on unlighted units.

Differences Between Lighted and Unlighted Mainline Units

The lighted mainline units studied have accident rates higher than those on the unlighted mainline units in all but one ADT category. An attempt was made to explain this situation by averaging separately available geometric, traffic, and accident variables for lighted and unlighted units.

The geometric variables examined are median width, length of section, pavement type, pavement edge markings, and delineators. Figure 10 shows each geometric variable by lighted and unlighted units. The average unlighted mainline unit has a 56 percent wider median and is 47 percent longer than the average lighted unit. A greater percentage of unlighted units have delineators on the right side and delineators on both sides than do lighted units. Pavement type and pavement edge markings vary little between lighted and unlighted units.

The traffic variables examined are percentage of in-state cars and small trucks, percentage of out-of-state cars and small trucks, and percentage of large trucks (Fig. 11). Unlighted units have a much larger percentage of trucks (and smaller percentage of in-state and out-of-state cars and small trucks) than lighted units. The traffic variable ADT was examined earlier. In units with comparable number of lanes, the lighted units had much greater ADT than unlighted units, especially in 3- and 4-lane units.

The accident variable examined was manner of collision. Lighted units had a greater percentage of rear-end collisions (Fig. 12) than unlighted units and a smaller percentage (31 percent) of accidents in which the vehicle left the road, with or without striking an object, than unlighted units (44 percent). Thirty-four percent of the vehicles that left the unlighted roads did not strike an object, and only 9 percent of vehicles that left the lighted roads did not strike an object.

These data indicate that geometric conditions such as curbs and guardrails on lighted highways prevent a greater percentage of vehicles from leaving the roadway than geometric conditions on unlighted highways. In addition, if a vehicle does leave the lighted roadway, 90 percent of the time it strikes an object (e.g., lighting pole). These figures are presented only to illustrate differences in various variables between lighted and unlighted mainline units, and are only tentatively offered to explain accident rate differences between units.

CONCLUSIONS

In this study, only urban mainline units of the Interstate Highway System have been examined. Comparison of 24-hour accident rates and night (6 p.m. to 4 a.m.) accident rates between lighted (all lighting intensities) and unlighted mainline units by number of lanes and by ADT-per-lane groupings demonstrated that the lighted units have higher accident rates. This is understandable in the number-of-lanes analysis because the ADT was much higher on the lighted units than on the unlighted areas. But, in the ADT-

per-lane analysis, even in the same ADT-per-lane grouping, the lighted units had a higher accident rate. This statement should not be interpreted to mean that the lighting of a mainline unit causes a higher accident rate because many variables were not considered. Therefore, various geometric, traffic, and accident characteristics of lighted and unlighted units were contrasted. In general, these variables indicated some marked discrepancies between lighted and unlighted highway. These discrepancies, however, are not offered as the reasons for the difference in accident rates between lighted and unlighted units because only averages of the variables were considered (i. e., standard deviations and exposures were not calculated thus preventing statistical significance tests to be made).

Although the lighting policies used by the 5 states surveyed indicated that accident rate was not a criterion for lighting, it may be that the high-accident mainline units were lighted first.

No relationship was found between lighting intensity and accident experience, but the data were very limited.

ACKNOWLEDGMENTS

This report presents the results of an analysis conducted on the Interstate System Accident Research, Study II, data base by Westat Research, Inc., under contract with the Federal Highway Administration. The following states contributed data for this study: Arizona, California, Colorado, Connecticut, Florida, Illinois, Indiana, Kansas, Michigan, Minnesota, Mississippi, Montana, New York, North Carolina, North Dakota, Ohio, Oklahoma, Pennsylvania, Rhode Island, South Dakota, Tennessee, Vermont, Virginia, and Wisconsin. Members of the Traffic Systems Division, U.S. Bureau of Public Roads, were involved in the development and supervision of the research, and the Planning and Research engineers of the Bureau's Regional and Division Offices encouraged the states to participate and acted as liaison between the states and the Traffic Systems Division.

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An Evaluation of Design Criteria for Operating Trucks Safely on Grades

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This paper is concerned with the evaluation of design criteria relating truck operating characteristics on grades to the implementation of truck climbing lanes. The evaluation is specifically concerned with truck operating characteristics on grades, truck weight-horsepower ratios as they pertain to truck operating characteristics, and truck speed as it is related to truck operating characteristics and design criteria for climbing lanes.

• IN THE DESIGN OF HIGHWAY GRADES consideration is given to the critical length of grade. The critical length of grade is that combination of percentage and length of grade that will cause a designated design vehicle to operate at some predetermined minimum speed. A lower speed is considered unacceptable for safety and operational efficiency. Two alternatives are considered when a designed grade is longer than critical: (a) Adjust the grade line until it is no longer critical, or (b) add an auxiliary truck climbing lane in which slow-moving vehicles can operate adjacent to the main travel lane.

This study was conducted in response to an increasing concern by highway design engineers regarding the validity of geometric design criteria related to safe operation of slow-moving vehicles on highway grades. The report presents a review of current AASHO (1) design criteria and an evaluation of these criteria based on the existing practices. The evaluation was specifically concerned with truck operating characteristics on grades, truck weight-horsepower ratios related to operating characteristics, and truck speed related to operating characteristics and geometric design criteria.

Of all vehicles operating on highways, the large transport trucks have the lowest engine power relative to their weight. Hence, these vehicles are generally the slowest on upgrades and require the longest distances to accelerate. Realistic design of highway grades and acceleration lanes should be based on the performance of these particular vehicles, inasmuch as all other vehicles are capable of better performance.

REVIEW OF CURRENT DESIGN CRITERIA

Design criteria relating truck operating characteristics on grades to the implementation of critical lengths of grade and truck climbing lanes are examined under the following topics: truck operating characteristics on grades, truck weight-horsepower ratios related to climbing characteristics, and design criteria for critical lengths of grade and truck climbing lanes.

Truck Operating Characteristics on Grades

An extensive study (2) of truck performance was conducted from 1938 to 1941 to determine the separate and combined effects of roadway grade, tractive effort, and gross vehicle weight. Data from this study were analyzed (3) to determine the effect of length of grade on the speed of trucks for a wide range in load, grade, and vehicle size.

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Speed-distance curves were developed for 3 weight classifications: light, medium, and heavy. These curves formed the basis for the 1954 AASHO (4) design criteria for critical lengths of grade.

In 1949, Willey (5) documented the performance of trucks on grades. He developed speed profiles of truck performance on different mountainous grades in Arizona. The observed trucks were classified according to the following gross vehicle weight to brake horsepower (bhp) ratios:

Group	lb/bhp
A	199 and under
B	200 to 299
C	300 to 399
D	400 and over

Willey developed a gradability curve of heavily loaded trucks (combination of group C and group D), which showed the expected average behavior of vehicles loaded to capacity, or nearly so, on various grades (Fig. 1).

Huff and Scrivner (6) used Willey's gradability curves in developing their simplified climbing lane theory. This theory considered the forces acting on a truck ascending a grade to develop the force equation

$$\frac{W}{g}\frac{dv}{dt} = P - W \sin \theta \qquad (1)$$

where

- W = gross vehicle weight, lb;
- g = acceleration of gravity, 32.2 ft/sec^2;
- dv/dt = change in velocity with respect to time, ft/sec^2;
- P = net driving force on the vehicle, lb; and
- θ = the grade angle, deg.

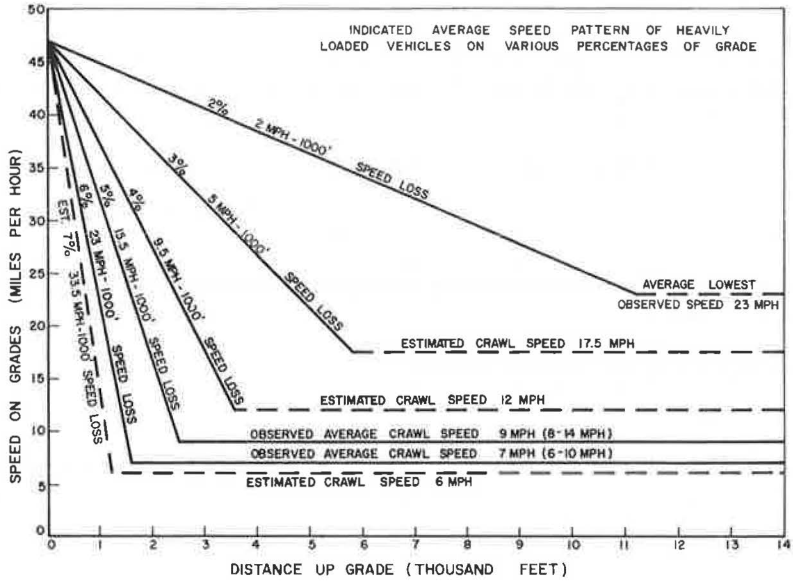


Figure 1. Gradability curves of heavily loaded trucks on different grades.

This equation holds when the driving force needed to impart angular acceleration to the rotating engine parts is neglected. Equation 1 may be rewritten as

$$\frac{P}{W} = \frac{1}{g}\frac{dv}{dt} + \sin\theta \tag{2}$$

The net driving force acting on the vehicle, P, is the total traction exerted by the driving wheels against the road surface, minus wind and road surface resistance.

Engine operation at partial throttle was not considered because the driver's choice rather than highway geometry would then determine the vehicle performance. Therefore, if the truck operates at the highest possible speed and within the manufacturer's recommendations, the total driving force as a function of the velocity only can be approximated if the following assumptions are made:

1. That there is no inertial resistance to angular acceleration;
2. That no wind exists; thereby, air resistance is considered as a function of the velocity; and
3. That there is no change in pavement type or roughness; thereby, surface resistance is considered as a function of the velocity.

It was concluded, therefore, that, although the net driving force must satisfy Eq. 2, it may also be expressed as some function of velocity only.

If a truck operates at maximum sustained speed on any grade, the value of P/W may be calculated from Eq. 2, which reduces to P/W = sin θ. This value of P/W will always exist at the respective speed, at least approximately, regardless of the value of the acceleration.

Figure 2 shows the relation of P/W to maximum sustained speeds, v, on various grades. The maximum sustained speeds were taken from the gradability curves shown

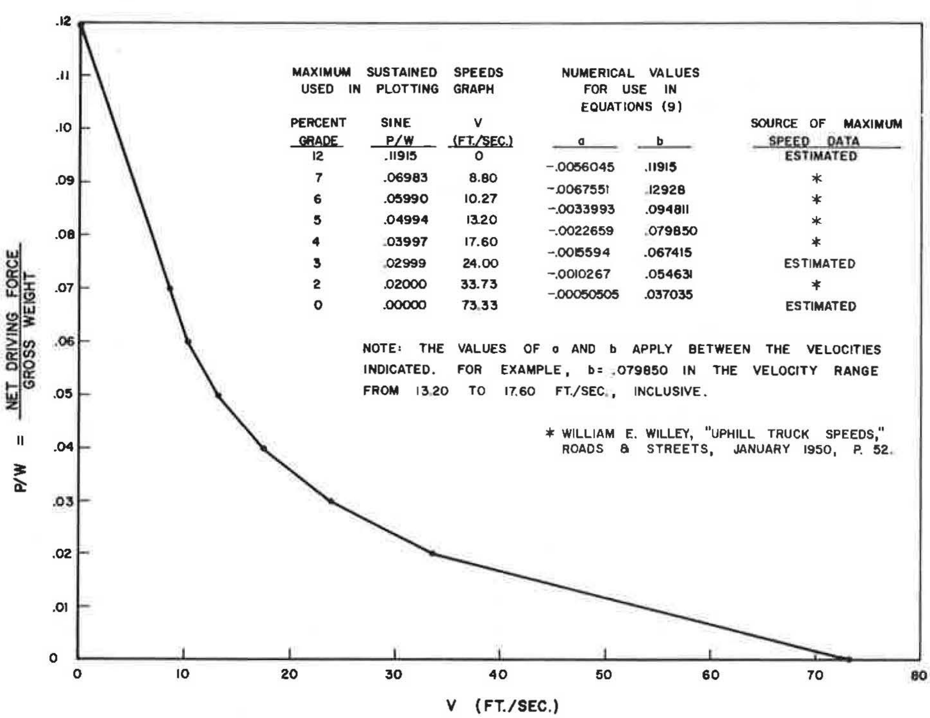


Figure 2. P/W versus maximum sustained speeds on various grades.

in Figure 1. The points shown in Figure 2 were connected by straight-line segments to form a continuous graph. Each line segment was represented by the general equation

$$P/W = av + b \tag{3}$$

where v is the velocity at any point along a line segment, v_n to v_{n+1}, and a and b are constant along the same line segment. By substituting the P/W value of Eq. 2 into Eq. 3, we derive a new general motion equation:

$$\frac{dv}{dt} - gav + g(\sin\theta - b) = 0 \tag{4}$$

where v, a, and b are restricted, as noted earlier.

The position of the truck along the grade may be represented at any instant by its coordinate, x, measured along the direction of the truck. If dv/dt is the change in velocity, v, with respect to time, t, along that line segment, Eq. 4 may be developed into an equation suitable for the construction of speed-distance and time-distance curves. Appendix A discusses the derivation.

$$x = \frac{v - v_o}{2g} + (\sin\theta - b)t \tag{5}$$

where

$$t = \frac{1}{ag}\ln\left(\frac{av - \sin\theta + b}{av_o - \sin\theta + b}\right) \tag{5a}$$

To construct speed-distance by using Eq. 5, where the velocity change involves more than one line segment, requires that the distance or time be calculated over each interval and added in order to obtain total distance or total time. Actually, by utilizing the same assumptions made by Huff and Scrivner in developing Eqs. 5 and 5a, one can derive a much simpler, singular speed-distance. Appendix A discusses the derivation.

$$x = \frac{1}{g}\,\frac{V_o^2 - V^2}{a(v_o - v) - 2(\sin\theta - b)} \tag{6}$$

In December 1953, Huff and Scrivner (6) conducted a road test of a heavy truck to determine whether these theoretical equations applied to the actual performance on grades. The operating features and data for the truck test are given in Table 1. Eleven grades ranging from 700 to 1,500 ft in length and from 0.16 to 7.62 percent in grade were used in the tests.

Figure 3 shows the data obtained in the tests of the heavy truck. Each computed value of P/W was plotted against its corresponding velocity. The points represent any instant where the acceleration was not zero, and the circles represent any instant at which the truck was operating at maximum sustained speeds. Certain areas, where the points were scattered so as not to represent any consistency, were ignored, and an average line was drawn through the remaining points. This line represented P/W as a function of velocity only.

The data given in Table 1 were also used to compute the maximum sustained speeds according to the SAE procedure (7). These speeds, plotted against the corresponding sin θ, are also shown in Figure 3.

The average values of P/W versus velocity (Fig. 3) were used to develop speed-distance curves for each of the 11 test grades and then compared against the gradability curves developed from the field test. If the curves for each grade coincided, the computed curve

TABLE 1

OPERATING FEATURES OF TEST TRUCK

Feature	Description or Dimension
Vehicle	International R-195 tractor with Hobbs tandem-axle, flat-bed trailer
Dimensions	
Height, ft	7.75
Width, ft	7.75
Gross vehicle weight, lb	57,180
Rated gross vehicle weight, lb	50,000
Gear ratios	
Transmission	6.98, 3.57, 1.89, 1.00, 0.825
Auxiliary transmissions	None
Axle	6.50, 8.86
Total gear reductions	61.84, 45.37, 31.63, 23.21, 16.75, 12.28, 8.86, 6.50, 7.31, and 5.36
Tire size	10 × 20
Net engine horsepower at sea level	146 at 2,600 rpm
Brake horsepower	162 at 2,800 rpm
Altitude, ft	950
Road type and condition	Bituminous, good
Net weight-horsepower ratio, lb/hp	391
Weight to rated horsepower ratio, lb/hp	353

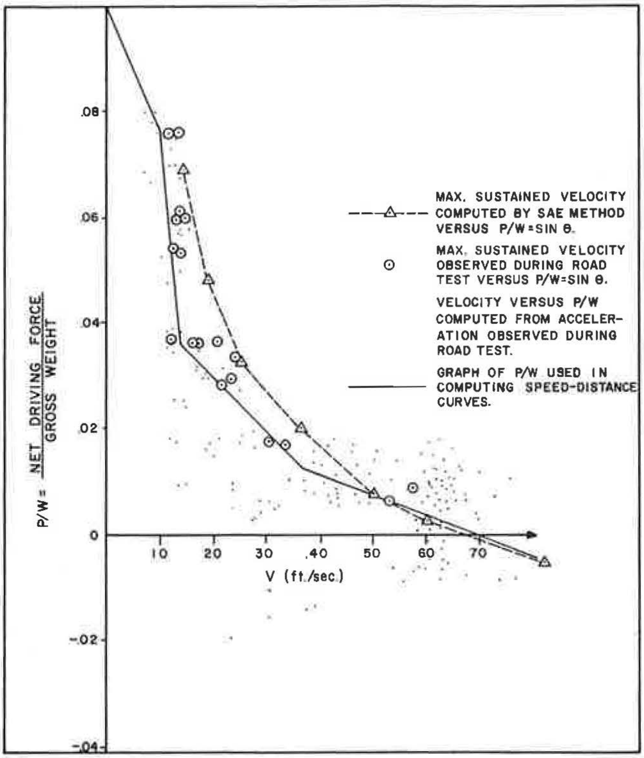


Figure 3. P/W versus velocity as computed and observed.

was considered to be representative of the measured test data; if they did not coincide, the opposite was considered.

A comparison of the computed curves and the measured gradability curves showed a fair amount of consistency. There were, however, 2 major discrepancies:

1. Some irregularity that appeared in the curves was caused by the motion of the truck, especially on some of the upgrade deceleration curves where maximum sustained speeds were reached.
2. The actual maximum sustained speeds were 1 to 3 mph greater than the maximum sustained speeds shown on the computed curves.

It was concluded that, although these discrepancies existed, the gradability curves shown in Figures 4 and 5 (developed through the use of Eq. 5 and Fig. 3) represented the performance of the test truck on grades. Equation 5, therefore, was considered satisfactorily accurate for use in predicting truck operations on grades for design purposes. The gradability curves shown in Figures 4 and 5 are those employed in the 1965 AASHO Policy (1).

Firey and Peterson (8) presented an equation that is almost identical to that of Huff and Scrivner:

$$\frac{W}{g}\frac{dv}{dt} = F_T - F_R - W \sin \theta \tag{7}$$

where F_T is the truck engine thrust force and F_R is the truck rolling resistance force.

The engine thrust force, F_T, is zero when the clutch is disengaged and, based on the assumption that the engine torque at wide-open throttle is constant over the operating speed range of the engine, F_T was calculated from the following equation:

$$F_T = \frac{E}{v_{max}} (550) \tag{8}$$

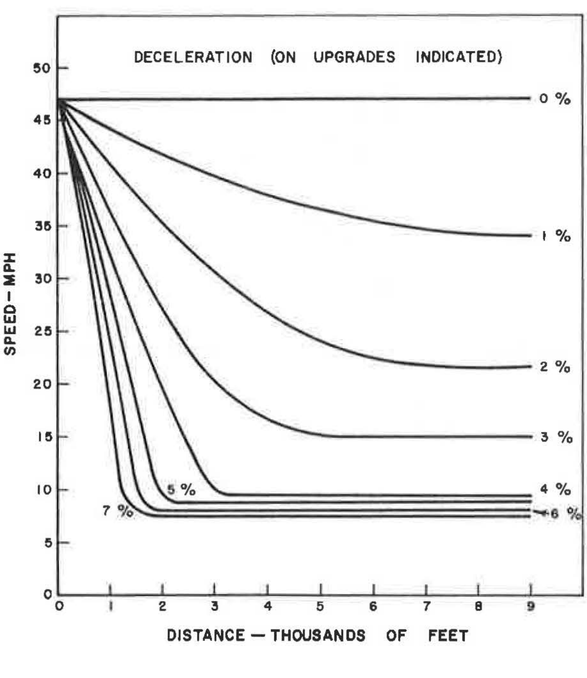


Figure 4. Speed-distance curves for typical heavy truck operating on various grades–deceleration on upgrades indicated.

where

E = engine rpm at wide-open throttle and

v_{max} = maximum truck speed attainable in a particular gear setting, ft/sec.

The truck rolling resistance force, F_R, was calculated from the following equation:

$$F_R = \frac{W}{148.5} + 195 \tag{9}$$

This is an empirical equation subject to the constraints of the coasting tests of several heavy trucks as described in another study (9). For significant upgrades, the exactness of F_R in Eq. 9 is not very important because F_T is the dominant resisting force to vehicle motion.

The net force, F_o, acting on a truck was defined by the following equations:

$$F_o = \frac{W}{g}\frac{dv}{dt} = \frac{E(550)}{v_{max}} - \frac{W}{148.5} \tag{10}$$

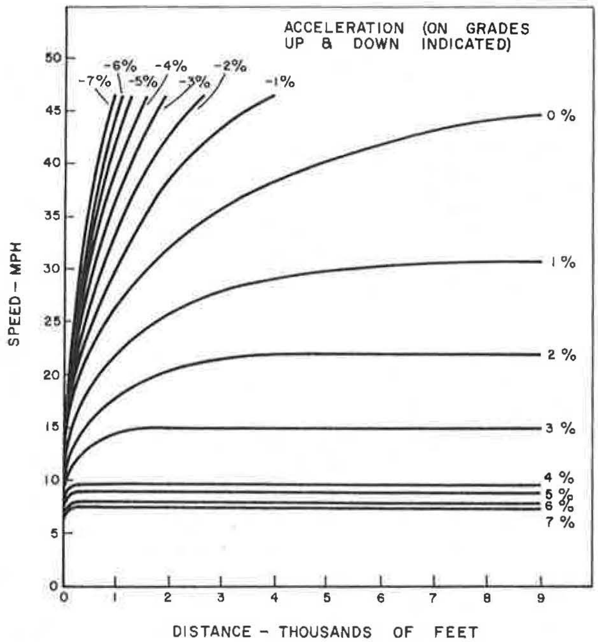


Figure 5. Speed-distance curves for typical heavy truck operating on various grades—acceleration on upgrades and downgrades indicated.

at wide throttle. With clutch disengaged, $F_T = 0$. Therefore,

$$F_O = \frac{-W}{148.5} - 195 - W \sin \theta \qquad (11)$$

For computing speed-distance relationships on uniform grades, the following basic physics equations were used:

$$x = v_O t + 1/2\ at^2 \qquad (12)$$

$$v = v_O + at \qquad (13)$$

Because the acceleration, a, in these equations was considered equivalent to dv/dt and because $dv/dt = F_O g/W$, the following equations were derived for computing speed-distance relationships:

$$x = v_O t + \frac{F_O g t^2}{2W} \qquad (14)$$

$$v = v_O + \frac{F_O g t}{W} \qquad (15)$$

The velocity versus distance curves on uniform grades was calculated by the following steps:

1. Values were assumed for W, W/H_P, θ, and initial v_O.
2. These values were substituted into the vehicle motion equations (Eqs. 4 and 5).

3. On deceleration curves the first gear shift was assumed to be 0.8 v_0, and on acceleration curves, $v_0/0.8$.

4. An average time of 2 sec was determined (9) to shift the gears, and it was assumed that the vehicle followed the vehicle motion equations for clutch disengagement during the gear shifting interval.

5. Steps 2 and 3 were repeated, and the vehicle motion equations for the clutch disengagement over the gear shifting interval were used.

6. For the second wide-open throttle periods, steps 2 and 3 were repeated, and the terminal speed from step 5 was used as v_0 in Eqs. 14 and 15.

7. The preceding steps were reiterated with values of v_0 until that value reached the established limitations: 10 mph on deceleration curves, or 50 mph on acceleration curves.

Firey and Petersen developed gradability curves from the foregoing procedure for truck weight-horsepower ratios of 200, 300, and 400. The gradability curve for a weight-horsepower ratio of 400 is shown in Figure 6.

Truck operations can be related to design for highway grades by selecting a design vehicle that represents some lower boundary of operation. Willey (5) was the first to classify truck operating characteristics according to weight-horsepower ratios. Because the weight-horsepower ratios of trucks can be measured in field studies, this measure appears to be best suited as a parameter for determining a design vehicle.

In 1957, Saal (10) studied the relationship between the gross weight of a motor truck and its horsepower. This study indicated that the percentages of trucks in 1950 having a weight-horsepower ratio greater than 400 were as follows: 3-axle trucks, 10 percent; 2-axle truck-trailers with 1-axle semitrailers, 13 percent; 2-axle truck-trailers with 2-axle semitrailers, 41 percent; all other combinations, 57 percent. He also stated that from 1955 to 1958 a 10 percent improvement in the performance ratio of all groups had occurred.

In 1963, Wright and Tignor (11) reported on the 1949, 1955, and 1963 brake studies of the Bureau of Public Roads. Figure 7 shows cumulative frequency distributions of weight-horsepower ratios from the 1963 study for trucks classified by number of axles. Of all the loaded trucks in this study, only 8 percent did not meet the 400:1 ratio accepted by AASHO (1) as a tolerable design performance ratio. Of all the trucks (loaded and unloaded) weighed in the 1963 study, only 5 percent exceeded the 400:1 ratio.

There has been a definite trend toward decreasing weight-horsepower ratios of trucks operating on the highways. Figure 8 shows this trend for the 1949, 1955, and 1963 brake studies (11). Another study (12) indicates that there has also been a trend toward more heavy trucks on the highways. The number of heavy trucks (over 26,000-lb gross vehicle weight) on the highways increased approximately 3.4 times from 1954 to 1967 and is predicted to increase 3 times from 1967 to 1980 (12).

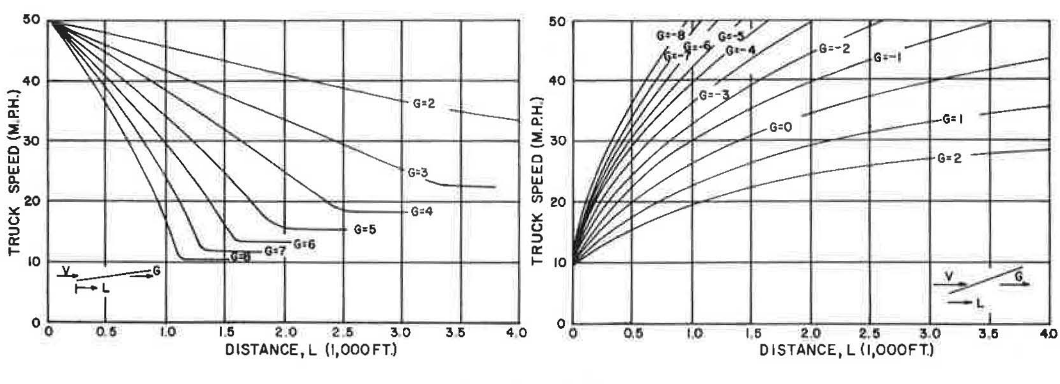


Figure 6. Deceleration and acceleration gradability curves for trucks for weight-horsepower ratio of 400.

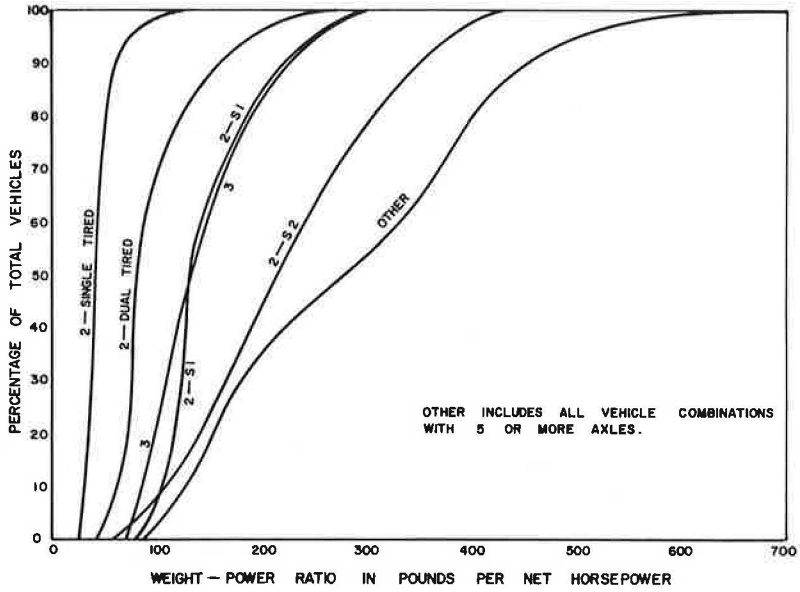


Figure 7. Cumulative frequency distributions of weight-horsepower ratios of commercial vehicles.

In 1968, more International Harvester (IH) trucks were registered across the United States in the heavy category (26,000 lb and over). International Harvester offers 5, 8-cylinder diesel engines to power its 65,000-lb trucks. The weight to net horsepower ratio of an IH truck powered by each of those engines would be 279:1, 298:1, 342:1, 392:1,

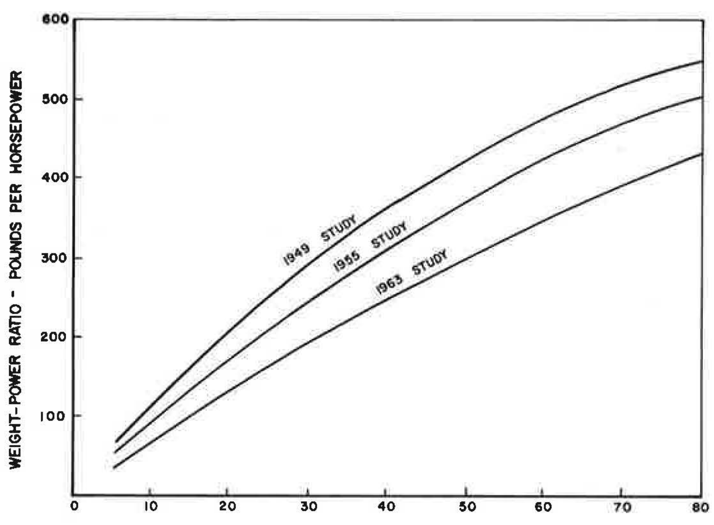


Figure 8. Weight-horsepower ratios based on average data for all types of vehicles from 1949 to 1963.

or 414:1, depending on which model was chosen. Only 1 of the 5 engines offered would be outside the accepted tolerable performance ratio of 400:1 (13).

The AASHO Policy (1) states that trucks with a weight-horsepower ratio of about 400:1 have acceptable operating characteristics from the standpoint of the highway user. It is stated that such a ratio will ensure a maximum sustained speed of 15 mph on a 3 percent grade. There is also evidence that the industry is finding the 400 ratio a desirable goal and is voluntarily accepting it as a performance control, with the result that the weight-horsepower ratios of trucks over the last several years have improved. This improvement is illustrated by the trend curves shown in Figure 8.

Design Criteria Related to Truck Operations

The 1965 AASHO Policy indicates that the average truck speed is approximately 6 mph less than the average passenger car speed on a level highway section. It increases on downgrades of 5 percent or less, and decreases on downgrades of 7 percent or steeper. On upgrades, the maximum sustained speed that a truck can maintain depends on the length and steepness of the grade and the weight-horsepower ratio of the truck. Factors affecting the average speed over the entire section are the truck's entering speed and wind resistance and the skill of the operator.

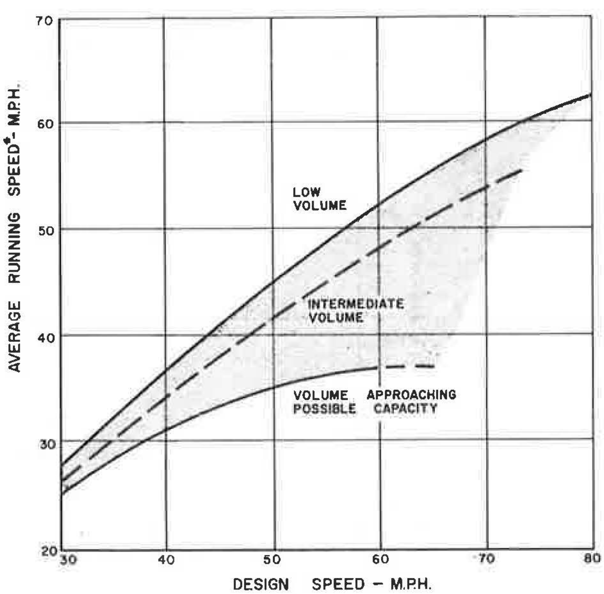
The "critical length of grade" is defined by AASHO (1) as the maximum length of a designated upgrade on which a loaded truck can operate without an unreasonable reduction in speed. If a truck is to operate reasonably on grades longer than the critical length, either the grade must be reduced or an additional climbing lane must be provided.

The AASHO Policy states that climbing lanes are necessary when the length of a specific grade causes truck speeds to reduce 15 mph or more, provided the volume of traffic and percentage of heavy trucks justify the added cost. Therefore, truck gradability or highway capacity or both can determine the critical length of grade. If truck gradability governs, the AASHO Policy considers that the following factors must be determined or assumed:

1. The size and power of the design truck as well as its gradability data. The 400:1 weight-horsepower ratio is accepted as the national design vehicle; therefore, the gradability curves shown in Figures 4 and 5 are employed by the AASHO Policy.

2. Truck speed at entrance to critical length of grade. The average running speed, as related to design speed, can be used to approximate the average speed of vehicles beginning an uphill climb, as shown in Figure 9 (2). For downhill or uphill approaches, the entering speed should be adjusted accordingly.

3. Minimum tolerable speed at which a truck should operate on the grade. Although no specific data are available on the minimum tolerable speeds of trucks, it seems logical that they would have a direct relationship to design speeds. Minimum speeds of 20 to 35 mph



RUNNING SPEED IS THE SPEED (OF AN INDIVIDUAL VEHICLE) OVER A SPECIFIED SECTION OF HIGHWAY, BEING DIVIDED BY RUNNING TIME

*AVERAGE RUNNING SPEED IS THE AVERAGE FOR ALL TRAFFIC OR COMPONENT OF TRAFFIC, BEING THE SUMMATION OF DISTANCES DIVIDED BY THE SUMMATION OF RUNNING TIMES. IT IS APPROXIMATELY EQUAL TO THE AVERAGE OF THE RUNNING SPEEDS OF ALL VEHICLES BEING CONSIDERED.

Figure 9. Relation of average running speed to volume conditions.

on highways with a design speed of 40 to 60 mph would be tolerable for a vehicle unable to pass on a 2-lane highway, provided the no-passing interval is short. As the volume on a 2-lane highway approaches capacity, the time interval will become more annoying. Multilane highways present more opportunity for and less difficulty in passing; therefore, lower tolerable truck speeds are applicable. In any case, highways should be designed so that trucks can maintain a tolerable speed.

Although all states are not in agreement as to what constitutes the critical length of grade, the most common determining factor is the 15-mph reduction in truck speed below the average truck running speed (1). Some states specify a minimum tolerable speed ranging from 20 to 35 mph instead of the 15-mph reduction. Figure 10 shows the critical length of grade for different speed reductions on specific grades (derived from Fig. 4). The 15-mph curve shown in Figure 10 is suggested by AASHO as a general design guide for establishing critical lengths of grades that are preceded by relatively level approaches. If there is an uphill approach to the grade, the critical length will be shorter; for downhill approaches, the converse will be true.

Climbing lanes may be justified from the standpoint of highway capacity as well as truck gradability. The effect of trucks on highway capacity is primarily a function of the difference in average running speeds between trucks and passenger cars. Passenger car equivalents for trucks at various combinations of running speeds are given in Table 2. By selecting the appropriate values from Table 2 and by using the gradability curves of Figures 4 and 5, one can calculate the design capacity on any grade for a given percentage of trucks.

The AASHO Policy (1) states that climbing lanes may be justified if the design hour volume (DHV) for a highway exceeds the design capacity of that highway by more than 20 percent. Table 3 gives the minimum design hour volumes, including trucks (not passenger car equivalents), for which climbing lanes should be considered.

The beginning of a climbing lane depends on the entering speed of the truck on a grade. The data shown in Figure 4 may be used to determine when a truck's speed has decreased enough to be sufficient cause for the implementation of a climbing lane. The AASHO Policy recommends that the beginning of the climbing lane should be preceded by a tapered section at least 150 ft long.

It is desirable to end a climbing lane when the truck's speed has accelerated to a speed at least equal to that at which it entered the climbing lane. The AASHO Policy states that this may be impractical on many grades because of the long distance required to accelerate to such a speed; therefore, a practical point for ending the lane is

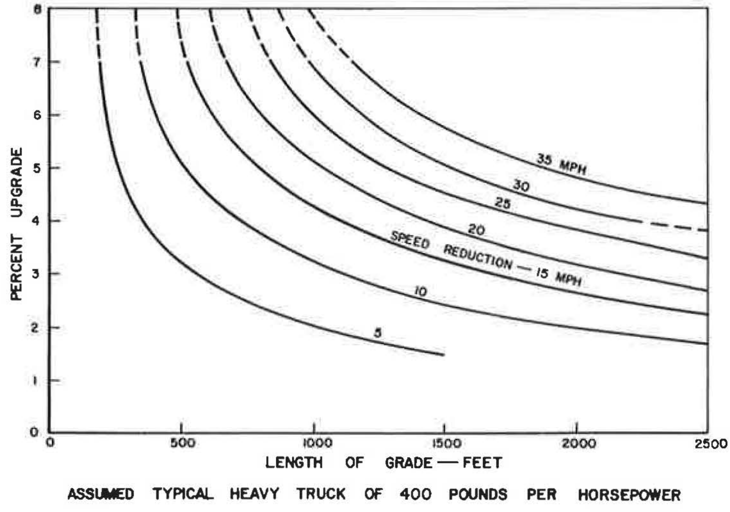


Figure 10. Critical lengths of grade for design.

where a truck can safely reenter the normal flow of traffic. This would be at a point where the sight distance is sufficient to permit passing with safety. The AASHO Policy recommends that a taper of at least 20 ft should be provided to allow the truck to reenter the flow of traffic.

A climbing lane should be at least 10 ft wide, preferably 12 ft. It should be easily distinguishable as an extra lane, and signs should precede the lane to notify trucks that there is a climbing lane ahead (1).

TABLE 2

PASSENGER CAR EQUIVALENTS FOR TRUCKS AT VARIOUS AVERAGE TRUCK SPEEDS AS RELATED TO PASSENGER CARS FOR INDIVIDUAL GRADES ON 2-LANE ROADS

Truck Speed (mph)	Number of Passenger Cars to Which One Truck Is Equivalent		
	45 to 50 mph[a]	40 to 45 mph[a]	35 to 40 mph[a]
35	3.0	2.7	2.5
30	5.0	4.9	3.0
25	8.6	7.6	5.0
20	13.9	11.7	8.8
15	22.9	18.7	15.0
10	40.5	32.5	25.2
5	94.5	75.0	50.0

[a]Average passenger car speed.

EVALUATION OF THE DESIGN CRITERIA

This evaluation of the design criteria for climbing lanes and critical lengths of grade covers the following areas: truck operating characteristics on grades, effect of weight-horsepower ratios on truck operating conditions, truck entering speeds, and speed reduction criteria related to safe operations.

Truck Operating Characteristics on Grades

Truck gradability procedures have been developed to predict the performance of trucks on grades in order to establish a design procedure that will enable all vehicles to operate safely on modern highways. Willey (5) documented the gradability characteristics of trucks and classified the observed trucks according to their weight-horsepower ratios. Gradability curves were developed for the heavily loaded trucks on different grades, a heavily loaded truck being one with a weight-horsepower ratio greater than

TABLE 3

MINIMUM 2-WAY DESIGN HOUR TRAFFIC VOLUMES FOR CONSIDERATION OF CLIMBING LANES ON GRADES ON TYPICAL 2-LANE ROADS

Percent Gradient	Length of Grade (miles)	DHV for Various Percentages of Dual-Tired Trucks			
		3 Percent	5 Percent	10 Percent	15 Percent
4	1/3			600[a]	525[a]
	1/2		700[a]	550	450
	3/4		670	500	390
	1	750[a]	640	470	370
	1 1/2	730	610	440	340
	2	710	590	420	340
5	1/3		640[a]	550[a]	480[a]
	1/2	690[a]	620	460	370
	3/4	650	540	380	300
	1	630	510	360	270
	1 1/2	600	490	340	260
	2	600	480	330	250
6	1/3	625[a]	580[a]	480	390
	1/2	570	470	330	250
	3/4	540	430	290	220
	1	530	420	280	210
	1 1/2	520	410	270	200
	2	510	410	270	200
7	1/3	470	410	310	240
	1/2	400	320	210	160
	3/4	380	300	200	150
	1	360	280	180	140
	1 1/2	350	270	170	130
	2	340	260	160	120

[a]Four lanes warranted for DHV over this amount.

Note: Detailed analysis of each grade is recommended instead of using tabular values.

300:1. Although Willey's observations may have been accurate at the time they were made, his report was not documented well enough to allow a verification of the number of heavily loaded trucks observed or the specific weight-horsepower ratio each heavily loaded truck had. No direct comparison of Willey's gradability curves can be made, therefore, with those developed by any of the other prediction procedures.

Huff and Scrivner (6) developed a prediction procedure and compared this theoretical procedure with actual field tests of the performance of a heavily loaded truck with a weight-horsepower ratio of 391. From the field tests, it was concluded that the theoretical procedure compared fairly well with the actual truck performance on grades. Huff and Scrivner's procedure appears to describe the performance of trucks on grades, although their average curve of P/W versus v, derived from the 1953 road test data, ignored some of the field data. The truck gradability curves derived from this procedure have been adopted as part of the AASHO Policy.

Firey and Peterson (8) developed truck gradability curves for trucks with weight-horsepower ratios of 200, 300, and 400. Figure 6 shows the speed-distance curves for the 400:1 ratio.

From a design viewpoint, the controlling factor for climbing lane design criteria is the maximum sustained speed that a truck can maintain on a grade. The higher the sustained speed is, the shorter the climbing lane needed, and the converse is also true. Table 4 gives a comparison of the maximum sustained speeds derived from the various truck gradability prediction procedures presented in this report. Also included are the maximum sustained speeds calculated by the SAE Procedure (7) for Huff and Scrivner's test truck. A considerable disparity is evident among the various prediction methods. The Huff and Scrivner values are the lowest, and the Firey and Peterson values are considerably higher than the others. The Huff and Scrivner values, however, are the only values that were substantiated by using a design vehicle, one that had a representative weight-horsepower ratio. Therefore, the Huff and Scrivner gradability curves adopted by the AASHO Policy are comparatively valid for design.

The Effect of Weight-Horsepower Ratios on Truck Operating Conditions

The weight-horsepower ratio of a truck determines how that truck will operate on grades. The higher the ratio, the more difficulty a truck will have ascending a grade, and the maximum sustained speed attainable will be lower.

There is a definite trend toward a maximum tolerable ratio of 400:1. In 1963, only 8 percent of all loaded trucks had a ratio greater than 400:1. The AASHO Policy states that the 400:1 ratio has been accepted from the viewpoint of the highway user and that the trucking industry has accepted the 400:1 ratio as a performance control. Only 1 of the 5, 8-cylinder engines offered by International Harvester in its heavy trucks would have a weight-horsepower ratio over 400:1. From all indications, it would seem reasonable to accept the 400:1 ratio as a design criteria.

TABLE 4

GRADE VERSUS MAXIMUM SUSTAINED SPEED AS DETERMINED BY DIFFERENT GRADABILITY PROCEDURES

Percent Grade	Maximum Sustained Speed (mph)			
	Willey	Huff and Scrivner	Firey and Peterson	SAE
1	N.A.	33.5	45.3	33.5
2	23.0	22.0	31.1	24.2
3	17.5	15.0	23.0	18.5
4	12.0	9.5	18.5	15.0
5	9.0	9.0	15.3	12.5
6	7.0	8.0	13.0	11.0
7	6.0	7.5	11.8	9.5

Truck Entering Speeds

Truck operating speeds along a highway, obviously, are determined by the profile of that particular highway. Huff and Scrivner selected an entering speed on grades of 47 mph because it was the average speed of trucks on approximate level grades in Texas. Although this no longer represents the average speed, the Texas Highway Department's 1968 statewide speed survey (13) indicates that a speed of 47 mph now represents the 15th percentile truck speed on Texas highways. Because the 15th percentile truck represents a reasonable lower boundary condition, the 47-mph entering speed is

appropriate for design when entry to a grade from a level approach is considered.

Speed Reduction Criterion

Truck speeds may be related to the average running speed of all traffic along a highway. The conclusion in a study reported by Solomon (14) was that, regardless of the average speed on the highway, the greater a vehicle's deviation from this average speed, the greater was its chance of being involved in an accident. The accident involvement rates related to the deviation from the average speed are shown in Figure 11.

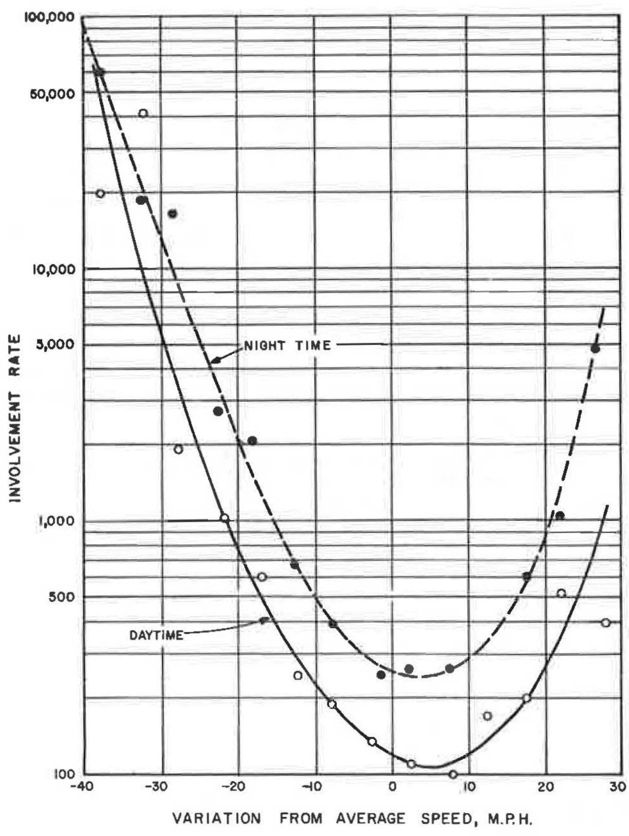


Figure 11. Accident involvement rate by variation from average speed during day and night.

The speed distribution of vehicles traveling Texas highways may be obtained from the Texas Highway Department (13). By utilizing this speed distribution and relating it to the accident involvement rates shown in Figure 11, one can obtain the accident involvement rate for 4-or-more-axle trucks operating on level grades. If the reduction in the average speed of all vehicles on a grade is assumed to be 30 percent of the truck speed reduction on that same grade, the accident involvement rates for truck speed reductions of 5, 10, 15, and 20 mph may also be developed (Appendix B).

The results of the analysis are given in Table 5 and shown in Figure 12. Most states base their climbing lane design on the criterion of a 15-mph reduction of truck speed. The accident rate for a 15-mph reduction is almost 9 times the involvement rate for a 0-mph reduction and approximately 2.4 times the rate for a 10-mph reduction (Table 5). The accident involvement rate increases, in absolute terms, 1,280 from the 10-mph to the 15-mph reduction. This is an increase of more than 5 times the increase from the 0-mph to the 5-mph reduction. This would indicate that a definite consideration should be given to the 10-mph reduction as a climbing lane design criterion, in place of the present 15-mph reduction.

TABLE 5

ACCIDENT INVOLVEMENT RATE OF TRUCKS ON GRADES COMPARED TO THE VARIATION FROM THE AVERAGE SPEED OF ALL VEHICLES ON A HIGHWAY

Speed Reduction (mph)	Accident Involvement Rate	Involvement Rate Ratio Related to 0-Speed Reduction
0	247	1.00
5	481	1.95
10	913	3.70
15	2,193	8.90
20	3,825	15.90

For the steeper grades, thought should be given to further reduction of the speed criterion. A 5-mph decrease in the speed reduction criterion does not substantially increase the required climbing lane length for the steeper grades (Fig. 10). This small increase in climbing lane length would be more than offset by the concomitant reduction of the accident involvement rate. These

same considerations apply on the downstream end of the climbing lane, where it is necessary to allow acceleration of the truck to a speed at which it can safely reenter the normal traffic stream.

In terrain that dictates consecutive climbing lanes at short intervals, consideration should be given to joining the separate climbing lanes to form one continuous lane. This would eliminate the hazardous situation of reentering the truck into the normal flow of traffic and then, in a short distance, removing the truck again.

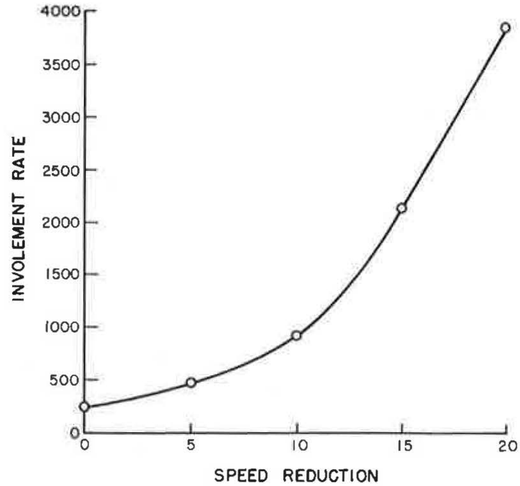


Figure 12. Accident involvement rate of trucks by speed reduction of design vehicle.

SUMMARY AND CONCLUSIONS

Based on this evaluation, which covered several truck gradability studies and prediction procedures, there is no substantiated justification for upgrading the truck gradability curves developed by Huff and Scrivner (2) as employed by the 1965 AASHO Policy (1). These curves were theoretically derived and validated by road tests of a heavily loaded truck with an approximate weight-horsepower ratio of 400:1. The trucking industry appears to have accepted this ratio as a performance control, although this does not account for the overloading occasionally practiced. From all indications of the trends in weight-horsepower ratios of trucks in operation, the 400:1 ratio appears to have continuing application as a design criterion.

The truck gradability curves developed by Huff and Scrivner utilize a 47-mph speed for trucks entering a grade from a level section. This represented the maximum sustained speed of the test truck on a level grade. This speed was the average of all trucks on Texas highways in 1953 and was considered as representative of a critical operating condition. Actually, a more representative critical speed would be the speed that is exceeded by, say, 85 percent of the trucks on the highway. The Texas Highway Department's 1968 speed survey indicated that approximately 85 percent of the trucks exceeded 47 mph. Therefore, the 47-mph truck entering speed is applicable for current design considerations.

The AASHO Policy currently employs a 15-mph speed reduction criterion for determining critical lengths of grades. No objective basis could be found for this criterion. Some existing data were applied to establish an objective basis for a speed-reduction criterion.

Taragin (3) developed a curve that related accident involvement rate to deviation from the average speed of the traffic stream. This relationship showed that the involvement rate increases logarithmically as this deviation increases. This relationship and the Texas Highway Department's 1968 speed survey data were used to compute accident involvement rates for various speed reductions of 4-or-more-axle trucks (Fig. 12). The accident involvement rate related to a 15-mph speed reduction of the design vehicle is almost 9 times that of a 0-mph reduction. The involvement rate increases very rapidly for increases in speed reduction beyond 10 mph. This relationship indicates that a 10-mph speed reduction criterion should be substituted for the present 15-mph criterion.

Highway engineers have been concerned that present design criteria are often responsible for truck climbing lanes that are too short for efficient operation. Operational problems are created for the following reasons:

1. With the present 15-mph speed reduction criterion, the common practice has been to end a climbing lane when the design truck regains a speed equivalent to that speed for which the climbing lane was begun. This practice, for many profile conditions, allows

the ending of the climbing lane shortly over the crest of the hill. This practice can create a lack of adequate operational sight distance to the end of the climbing lane, especially for slow-moving automobile drivers who choose to use the auxiliary lane.

2. Truck drivers find it difficult to maintain desired operation of their vehicles on short climbing lanes and, therefore, are often reluctant to use climbing lanes in areas where they know by experience these auxiliary lanes tend to be short.

Although no investigation was made of the optimum length of truck climbing lanes, the substitution of a 10-mph speed reduction criterion in place of the current 15-mph criterion would alleviate these operational problems.

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Appendix A

DERIVATION OF FORMULAS

Huff and Scrivner's Speed-Distance and Time-Distance Formulas

Through the summation of forces acting on a truck ascending any grade, a basic force equation may be developed:

$$\frac{W}{g}\frac{dv}{dt} = P - W \sin \theta \tag{16}$$

When divided by W, Eq. 16 becomes

$$\frac{P}{W} = \frac{1}{g}\frac{dv}{dt} + \sin \theta \tag{17}$$

If it is stipulated that

$$\frac{P}{W} = av + b \tag{18}$$

then, by substitution, an equation is formed that does not contain P/W:

$$\frac{dv}{dt} - gav + g(\sin\theta - b) = 0 \tag{19}$$

If dv/dt is the change in velocity with respect to time, $\frac{v_o - v}{t}$, and v is the average velocity, $\bar{v}$, then Eq. 19 becomes

$$\frac{v_o - v}{t} - ga\bar{v} + g(\sin\theta - b) = 0 \tag{20}$$

By multiplying by the time, t, and solving for $\bar{v}t$, we may write Eq. 20 as

$$\bar{v}t = \frac{v_o - v}{ga} + \frac{gt(\sin\theta - b)}{ga} \tag{21}$$

Any distance, x, may be measured by the average velocity multiplied by time; therefore, Eq. 21 becomes

$$x = \frac{1}{a} = \frac{v_o - v}{g} + (\sin\theta - b)t \tag{22}$$

which is Huff and Scrivner's first equation.

Their second equation may be derived by first solving for dt in Eq. 19.

$$dt = \frac{dv}{g(av - \sin\theta + b)} \tag{23}$$

If Eq. 23 is integrated from t_o to t,

$$\int_{t_o}^{t} dt = \frac{1}{g}\int_{v_o}^{v} \frac{dv}{av - \sin\theta + b} \tag{24}$$

and, because $(\sin\theta + b)$ is constant over any interval v_o to v, then Eq. 24 becomes

$$t = \frac{1}{ag}\int_{v_o}^{v} \frac{adv}{av + (-\sin\theta + b)} \tag{25}$$

Then,

$$t = \frac{1}{ag}\ln\left[av + (-\sin\theta + b)\right] - \frac{1}{ag}\ln\left[av_o + (-\sin\theta + b)\right] \tag{26}$$

or

$$t = \frac{1}{ag} \ln \frac{av - \sin\theta + b}{av_o - \sin\theta + b} \tag{27}$$

Simplified Speed-Distance Formula Using Huff and Scrivner's Assumptions

A simplified speed-distance formula may be derived by using the same assumptions made by Huff and Scrivner. If dv/dt is the change in velocity with respect to time and v is the average velocity, Eq. 19 becomes

$$\frac{v_o - v}{t} - ga\overline{v} + g(\sin\theta - b) = 0 \tag{28}$$

When divided by the average velocity, $\overline{v}$, Eq. 28 becomes

$$\frac{v_o - v}{\overline{v}t} - ga + \frac{g(\sin\theta - b)}{\overline{v}} = 0 \tag{29}$$

Any distance, x, may be represented by an average speed times time, $\overline{v}t$; therefore, Eq. 29 becomes

$$\frac{v_o - v}{x} - ga + \frac{g(\sin\theta - b)}{\overline{v}} = 0 \tag{30}$$

Solving for x and substituting $\frac{v_o + v}{2}$ for $\overline{v}$, we may write Eq. 30 as

$$x = \frac{1}{g} \frac{v_o^2 - v^2}{a(v_o + v) - 2(\sin\theta - b)} \tag{31}$$

Appendix B

ANALYSIS OF ACCIDENT INVOLVEMENT RATES OF 4-AXLE TRUCKS ON GRADES

An analysis was made of accident involvement rates to ascertain whether the 15-mph speed reduction design criterion is adequate for determining the critical length of grade. In a report by Solomon (14), accident involvement rates were related to average running speeds of vehicles on a highway. The conclusion was that, regardless of the average speed on a highway, the greater a vehicle deviated from the average running speed of all vehicles, the greater was its chance of being involved in an accident. The involvement rates as they relate to the deviation from the average running speed of all traffic along the highway are shown in Figure 11 of this report.

Each year the Texas Highway Department reports the speed distribution of all vehicles traveling on the highways in Texas. This survey is made by recording the actual speed of vehicles at 31 strategically located speed-survey stations across the state. In 1968, the speeds of 48,253 vehicles were checked, 35,776 of which were passenger cars and 3,284 were 4-or-more-axle trucks.

The following assumptions were made to facilitate the analysis of accident involvement rates:

1. That the statewide average speed determined by the Texas Highway Department was the typical average speed of all vehicles operating on level grades along a highway.

2. That the statewide speed distribution for 4-or-more-axle trucks determined by the Texas Highway Department was the typical speed distribution for this type of truck operating on level grades along a highway.

3. That the involvement rate for each category would be multiplied by the daytime involvement rates versus deviation from the average speed (Fig. 11). The daytime rates were employed because they were lowest and were considered to be conservative for this analysis.

4. That all 4-or-more-axle trucks decelerated in the manner shown in Table 6.

5. That the average speed reduction of all vehicles on a grade was 30 percent of the average truck speed reduction on that same grade.

TABLE 6

ASSUMED SPEED REDUCTION OF 4-AXLE TRUCKS ACCORDING TO SPEED CATEGORIES FOR VARIOUS SPEED REDUCTIONS OF THE DESIGN TRUCK

Truck Speed Categories (mph)	Speed Reduction of Design Truck[a] (mph)				
	0	5	10	15	20
30 to 35	0	8	13	18	23
35 to 40	0	7	12	17	22
40 to 45	0	6	11	16	21
45 to 50	0[a]	5[a]	10[a]	15[a]	20[a]
50 to 55	0	4	8	12	16
55 to 60	0	3	6	9	12
60 to 65	0	2	4	6	8
65 to 70	0	1	2	3	4
70 to 75	0	0	0	0	0
Avg. speed of all traffic	59.4	57.9[b]	56.4[b]	54.9[b]	53.4[b]

[a]Design truck operates within the 45 to 50 mph category.
[b]Assumed average speed of all traffic on grades is calculated by subtracting 30 percent of the design truck speed reduction from the average speed, 59.4 mph, of all vehicles on level grades.

The following procedure was used to determine the accident involvement rates on grades:

1. The average speed of all vehicles on level grades and the speed distribution categories were obtained from the data reported by the Texas Highway Department.

2. The midpoint of each speed category was subtracted from the average speed of all vehicles to determine the difference.

3. The deviation in speed from the average for each category was used to determine the involvement rate for that category from the daytime involvement rates versus speed variation (Fig. 11).

TABLE 7

ACCIDENT INVOLVEMENT RATE OF 4-AXLE TRUCKS WITH AN ASSUMED SPEED REDUCTION ON GRADES OF 15 MPH BELOW THE SPEED ON LEVEL GRADES

Average Speed (1)	Truck Speed Categories (2)	Midpoint (3)	Difference From Average (4)	Percent of Total 4-Axle Trucks (5)	Involvement Rate (6)	Percent of Trucks × Involvement Rate (7)
54.9	12 to 17	14.5	-40.4	0.9	100,000	90,000
	18 to 23	20.5	-34.4	3.9	17,000	66,300
	24 to 29	26.5	-28.4	6.1	3,700	22,570
	30 to 35	32.5	-22.4	18.3	1,180	21,594
	38 to 43	40.5	-14.4	19.8	350	6,930
	46 to 51	48.5	- 6.4	37.4	175	6,545
	54 to 59	56.5	+ 1.6	10.0	118	1,180
	62 to 67	64.5	+ 9.6	3.4	123	4,182
	70 to 75	72.5	+17.6	0.2	200	40
Total				100.0		219,341

Involvement rate = 219,341/100 = 2,193

Notes: Col. 1. Average speed of all vehicles on level grades less 30 percent of assumed reduction in truck speed on grades: 59.4 - (0.3)(15) = 54.9.
Col. 2. Truck speed categories as determined by subtracting the assumed truck speed reduction in Table 6 from the speed categories established by the Texas Highway Department.
Col. 3. The midpoint of each truck speed category.
Col. 4. Difference of the average truck speed from the average speed (col. 1 minus col. 3).
Col. 5. Percentage of total 4-axle trucks in each speed category as determined by the Texas Highway Department.
Col. 6. Involvement rate taken from Figure 15.
Col. 7. Product of the percentage of total 4-axle trucks and the involvement rate for the speed differential for each speed category (col. 5 times col. 6).

4. This involvement rate for each category was multiplied by the percentage of 4-or-more-axle trucks within each speed category to obtain the weighted involvement rate.

5. All weighted rates were totaled and divided by 100.

6. The same procedure was followed, with one exception, to determine the involvement rates on grades that would cause a truck speed reduction of 5, 10, 15, and 20 mph. The average speed on the grade was established by subtracting 30 percent of the truck speed reduction from the average speed of all vehicles on level grades. All other steps, 2 through 5, were exactly the same. An example of the calculation procedure is shown in Table 7 for the 15-mph speed reduction.

Passing Performance With In-Car Displays

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Passing Aid System II (PAS II), a traffic monitoring system currently under development, is designed to advise drivers on 2-lane roads whether it is safe to pass in a sight-limited situation. The present study compares the effects of alternate PAS II in-car displays on driver performance in a highway driving simulator. The 3 displays tested were an alphameric readout, a flashing-color indicator, and an auditory signaling system; all 3 displays indicated the time before an oncoming car would be met. The alphameric display was related to consistent and conservative passing under normal circumstances. Drivers did not respond to emergency situations as well with the alphameric as with the flashing display. The auditory display, with a repetitive rate of sound, was confusing to drivers. Drivers preferred the alphameric display and rated the auditory low. Based on both throughput and safety maximization, the flashing display was recommended.

•TRAFFIC ON 2-LANE RURAL HIGHWAYS is often impeded by slow-moving vehicles. The driver who decides to pass must evaluate a complex problem involving acceleration and handling characteristics of his own vehicle, the passing car (PC), closing rate of his car with respect to the leading (LC) and approaching (AC) cars, distances to LC and AC, and road geometry, all within a few seconds. In the sight-limited, no-passing zone situation, information about the AC is not available and is replaced by the a priori probabilities estimated from previous oncoming traffic density. In a high-risk game against unpredictable events, the driver probably obeys the traffic law. When he does not, he occasionally loses the game.

BACKGROUND

Various techniques for estimating "goodness" of passing decisions have been applied to the oncoming-car, limited pass, where the AC is in sight while the passing decision is made. Drivers tend to overestimate low closing rates, underestimate high ones (1), and only slightly adjust to closing velocity (2). Jones and Heimstra (3) found that almost half of the judgments by drivers of the last point in time for safe passing were underestimates. Crawford (4) reported driver response speeded up as time to pass became short, then slowed down as time became too short for a safe pass, thus compounding risk in the latter situation.

Passing judgments were reported to be significantly improved when drivers were provided with knowledge of AC speed or closing velocity (1). Although the addition of information did not change passing time and safety margins, the variances of accepted passing time gaps and of safety margins were reduced significantly, indicating that the driver with knowledge was a more stable part of the driver-car-road system. A final criterion for the value of an information system is accident and mortality data, not available from the field except in grossest form and experimentally accessible only from simulation studies.

Paper sponsored by Committee on Operational Effects of Geometrics.

The U.S. Bureau of Public Roads has been exploring electronic techniques for detecting traffic flow and for giving drivers passing-aid information in sight-limited situations. A feasibility study was conducted on an early configuration of the Passing Aid System I (PAS I) at the Fairbank Highway Research Station, McLean, Virginia (5). Drivers were informed by an auditory signal whether an oncoming car was more than 1,300 ft away (steady tone) or less than 1,300 ft away (intermittent tone that increased in rate as distance decreased). Results indicated that PAS I increased the frequency of safe passing but did not change the frequency of unsafe passes.

The PAS concept is being developed into an operational system by the Raytheon Company under contract to the Bureau of Public Roads. A critical question arising from the PAS I findings is how should the information be given to the driver considering a pass. The PAS II system will be capable of determining time gaps between PC and AC. Three types of displays were considered practical and compatible with the PAS II system: (a) an auditory display similar to that used in PAS I; (b) a visual display of an alphameric message that gives time gap as a readout with redundant green-yellow-red color coding; and (c) visual display of a flashing, color-coded technique without the readout of the number of seconds in the time gap.

The question addressed in the present research was: If we presume that information concerining the AC may be used to reduce driver error (1, 5), which of the 3 types of displays proposed for use in PAS II, auditory (AUD), alphameric (A/N), or flashing (FL), would maximize road throughput and minimize risk in the passing maneuver? This study investigated the problem of display selection through a realistic simulation of a 2-lane highway situation, and an analysis of passes made under conditions expected to be found on the road.

METHOD

The Driving Environment

The PAS II experiment used the optical-belt simulator of the Injury Control Laboratory, Providence, Rhode Island. Four movable belts simulate the lanes and roadsides of a 2-lane rural road. The belts are optically interfaced with a full-sized, laterally moving car body (Fig. 1).

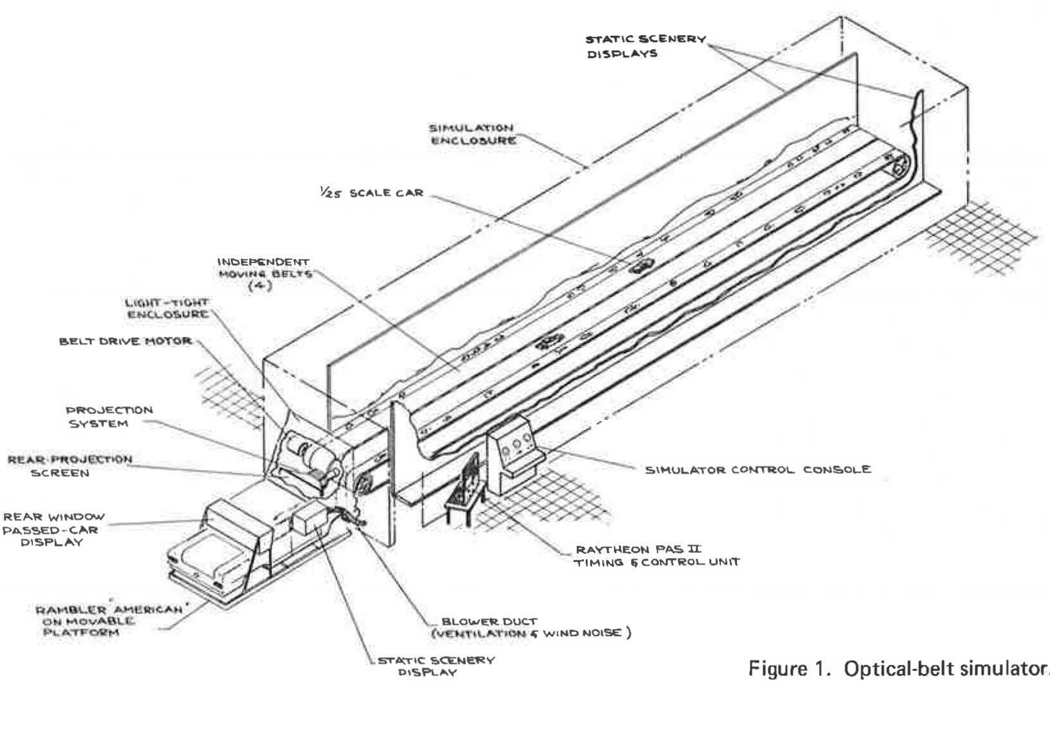

Figure 1. Optical-belt simulator.

Figure 2. Driver's view of roadway and PAS II display on dashboard.

The driver sits in a Rambler American 2-door sedan with automatic transmission. He views a display of the road scene (Fig. 2) appearing on a rear-projection screen directly in front of the Rambler windshield. Functioning instruments and controls and wind and motor noise are added for realism. Steering action causes the Rambler to move laterally in the direction steered; the input optics also move laterally across the near end of the conveyor belts. Further details concerning the simulator are given in other reports (6, 7).

PAS II Displays and Simulation

Three types of display messages, alphameric, flashing, and auditory, were used in the study to indicate the time before meeting an AC. Each display used either 5- or

TABLE 1

INFORMATION GIVEN IN EACH DISPLAY

<table>
<tr><th>Condition</th><th>Alphameric 6-Message</th><th>Alphameric 4-Message</th><th>Flashing 6-Message</th><th>Flashing 4-Message</th><th>Auditory 6-Message</th><th>Auditory 4-Message</th></tr>
<tr><td>System not operating</td><td>Black</td><td>Black</td><td>Black</td><td>Black</td><td>No sound</td><td>No sound</td></tr>
<tr><td>25 sec or more</td><td>25 SEC green background</td><td rowspan="2">20 SEC green background</td><td>Green light at 1.0 signal per sec</td><td rowspan="2">Green light at 1.0 signal per sec</td><td>1,100 Hz tone at 1.0 signal per sec</td><td rowspan="2">1,100 Hz tone at 1.0 signal per sec</td></tr>
<tr><td>20 to 25 sec</td><td>20 SEC green background</td><td>Green light at 1.3 signals per sec</td><td>1,100 Hz tone at 1.3 signals per sec</td></tr>
<tr><td>15 to 20 sec</td><td>15 SEC yellow background</td><td rowspan="2">10 SEC yellow background</td><td>Yellow light at 1.6 signals per sec</td><td rowspan="2">Yellow light at 2.3 signals per sec</td><td>1,100 Hz tone at 1.6 signals per sec</td><td rowspan="2">1,100 Hz tone at 2.3 signals per sec</td></tr>
<tr><td>10 to 15 sec</td><td>10 SEC yellow background</td><td>Yellow light at 2.3 signals per sec</td><td>1,100 Hz tone at 2.3 signals per sec</td></tr>
<tr><td>10 sec or less</td><td>DON'T PASS red background</td><td>DON'T PASS red background</td><td>Red light at 3.6 signals per sec</td><td>Red light at 3.6 signals per sec</td><td>1,100 Hz tone at 3.6 signals per sec</td><td>1,100 Hz tone at 3.6 signals per sec</td></tr>
<tr><td>Unanticipated approach car (abort)</td><td colspan="2">ABORT with intermittent red background and 1,450 Hz tone at 3.6 signals per sec</td><td colspan="2">Red light and 1,450 Hz tone at 3.6 signals per sec</td><td colspan="2">1,450 Hz tone at 3.6 signals per sec</td></tr>
</table>

10-second intervals, requiring 6 or 4 messages respectively to cover the time period of interest, 30 sec. The information presented in each display is given in Table 1.

All displays were generated on an in-car display unit mounted on top of the dashboard in the center of the Rambler PC. The visual displays were produced on an IEE Series 10 rear-projection readout device and shown on a $^{15}/_{16}$-in. square screen. Auditory signals were presented by a 3-in. loudspeaker mounted within the in-car display unit. Specific details of display design are given in another report (6).

An experimenter's control unit contained the electronics required for setting up time gaps, selecting display mode and message length, and controlling message presentation. Outputs from the display generator were used to trigger the start of the AC down the simulator roadway at the appropriate time and to feed a Beckman 8-channel pen recorder.

Experimental Design

The experimental parameters are given in Table 2. A 4-factor repeated-measures design was used, with all drivers tested individually under all combinations of displays, number of messages, time gaps (TG), and designated velocities (DV). The abort condition occurred as a random factor within the 30-sec time gap and was balanced across the other 3 factors.

Six combinations of display mode and numbers of messages were available. Each combination was given to a driver on a separate day. The drivers were randomly assigned to counterbalanced orders of display presentation to control practice effects.

On a given day, the driver was presented with 24 optional passing events that included each time gap at each designated velocity plus 4 abort conditions occurring at the 4 designated velocities with an initial 30-sec time gap. Passing during each designated velocity was aborted only once in each session. The 24 optional passing events were presented in random order to each driver. Experimental sessions, including warm-up, were an hour long. Drivers finished the experiment within 2 weeks of an initial practice session.

Experimental Procedure

After 10 min of warm-up, the driver began the test section of each session by bringing his vehicle, the PC, up to and maintaining a velocity designated by the experimenter (30, 40, 50, or 60 mph). After 30 sec of driving with no oncoming or impeding traffic, an LC was brought into view. The experimenter manipulated the velocity of LC so that PC, at the designated speed, gradually overtook it until a prepass headway corresponding to 1 car length per 10 mph was obtained. The experimenter disengaged the clutch for the right lane belt, holding the headway constant, and monitored the PC speed.

After 15 to 30 sec, the experimenter initiated the optional passing event by simultaneously engaging the clutch of the right lane belt and switching on the PAS II in-car display, already set for the preselected TG. The TG was assigned 5 values: 10, 15, 20, 25, or 30 sec. The display in use for the particular session-day immediately indicated the TG available and showed, in decreasing intervals, the TG remaining.

The driver had to decide, based on the information presented on the display, whether he could pass in the available time. The driver was preinstructed that, if he passed, he must remain in the passing lane until he saw the LC headlights switch on in his rearview mirror (Fig. 1). He could then return to the right lane. The LC headlights were timed to appear 2.5 sec after the PC simulated passing the LC, which allowed for a reasonable clearance before returning to lane.

If the pass was successful, the driver proceeded on the open road at a newly designated speed with no oncoming or impeding traffic for approximately 30 sec at which point another LC came into view.

TABLE 2

EXPERIMENTAL PARAMETERS

Parameter	Level
Number of messages	4-message, 6-message
Display mode	Alphameric, Flashing, Auditory
Time gap	30, 25, 20, 15, 10 sec
Designated velocity Prepass headway	30, 40, 50, 60 mph 60, 80, 100, 120 ft
Abort condition	On, Off p(Abort) = $1/6$

The experimenter proceeded to establish the prepass headway again by controlling the LC speed while monitoring the PC speed and keeping it within limits.

An unsuccessful pass occurred when the PC (actually the input optics to the driver's compartment) struck either an LC or AC in the act of passing or aborting a pass. All conveyor belts were stopped, the vehicles were repositioned, and the experiment was resumed as after a successful pass.

If the driver decided not to pass, he remained in lane until the AC passed, whereupon the experimenter adjusted the LC speed to the speed for the next trial, notified the driver of the proper speed, and readjusted the prepass headway to correspond to the newly designated speed.

If the driver started to pass, and then changed his decision, he could return behind the LC. In this mode, a head-on collision might have taken place, depending on the point of the PC reentry into the right lane. After reentry, the experiment proceeded as after a rejection of a passing option.

An abort signal was initiated by the experimenter once during each of the 4 designated velocities with a TG of 30 sec. At the 30-mph DV, the abort signal appeared 3 sec after the initial TG = 30 sec message; at 40 mph, 5 sec after; at 50 mph, 7 sec after; and at 60 mph, 9 sec after. (Extensive pilot testing indicated that at these times and speeds the average driver would be at the point in his pass where it would be equally difficult to get safely behind or in front of the LC.) Simultaneously with the abort signal, the AC was started down the conveyor belt at the same speed as the PC's initial and designated velocity. The driver could then either attempt to pass LC or attempt to maneuver behind LC. The abort signal was displayed until the AC passed or collided with the PC.

Subjects

Twenty-four drivers took part in the study, 13 males and 11 females. These subjects, whose ages ranged from 21 to 64, were licensed drivers with at least 20/30 corrected distance acuity and normal color vision as determined on the Bausch and Lomb Orthorater. Drivers had a minimum of 4 years' driving experience. Each participated in all phases of the experiment and used all display configurations. Further information on the driver sample is contained in another report (6).

RESULTS

Number of Successful Passes

The objective of a passing-aid system is to facilitate traffic flow while maximizing driver safety. A global index of level of traffic in the present study was the number of passes successfully completed with each display. An analysis of variance of passes made using the 3 display modes and 2 message lengths indicated significant effect of display mode ($p < 0.05$). The A/N displays were associated with significantly fewer passes than either the FL or AUD, averaging about one pass less per session. There was no significant difference between 4 and 6 messages, nor were there any significant interactions.

Passes completed at each designated velocity and time gap for each display were not treated statistically because of the great variation in number of items per cell. The frequencies of passes made at DV's and TG's were used to derive median acceptable time gap for a successful pass, or time gap threshold (Fig. 3). Passing times averaged about 5 sec longer in this study than in the literature (8, 9) because in the present study passing time commenced with the onset of the display and not with the leaving of the right lane, and also included overtaking time from a safe prepass headway. Time gap threshold was determined as the time gap where 50 percent of fully attempted passes would have occurred, based on frequency of successful passes and occurrence of accidents during normal passing. Time gap thresholds for FL displays were lowest of the 3 displays at 30 mph, a relatively safe condition where the driver had the greatest part of the accelerative capability of his car available. FL-4 thresholds were lowest at all DV's except 60 mph where the threshold was as high or higher than for any other display. The AUD-6 time gap threshold was lowest of the 6 displays at the most hazardous

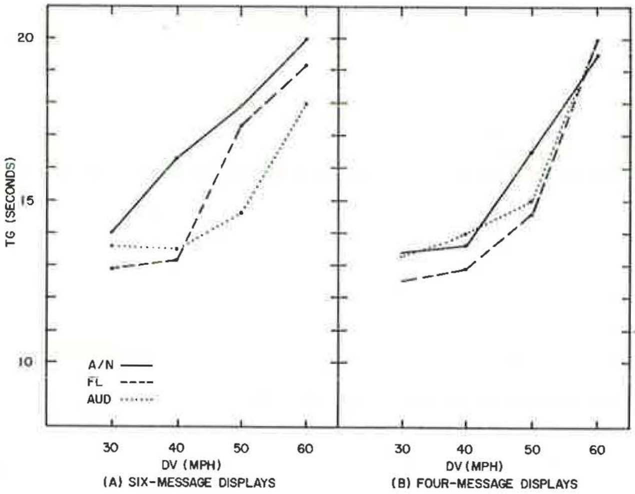

Figure 3. Time gap thresholds at designated velocities for displays.

velocity, 60 mph. The A/N-6 display was related to the highest threshold at every velocity but 60 mph.

Safety Margin

A second criterion of passing performance was the amount of time between the moment the PC returned to the right lane and the moment the PC and AC passed on the road, or the safety margin. Figure 4 shows the safety margins for all 3, 6-message

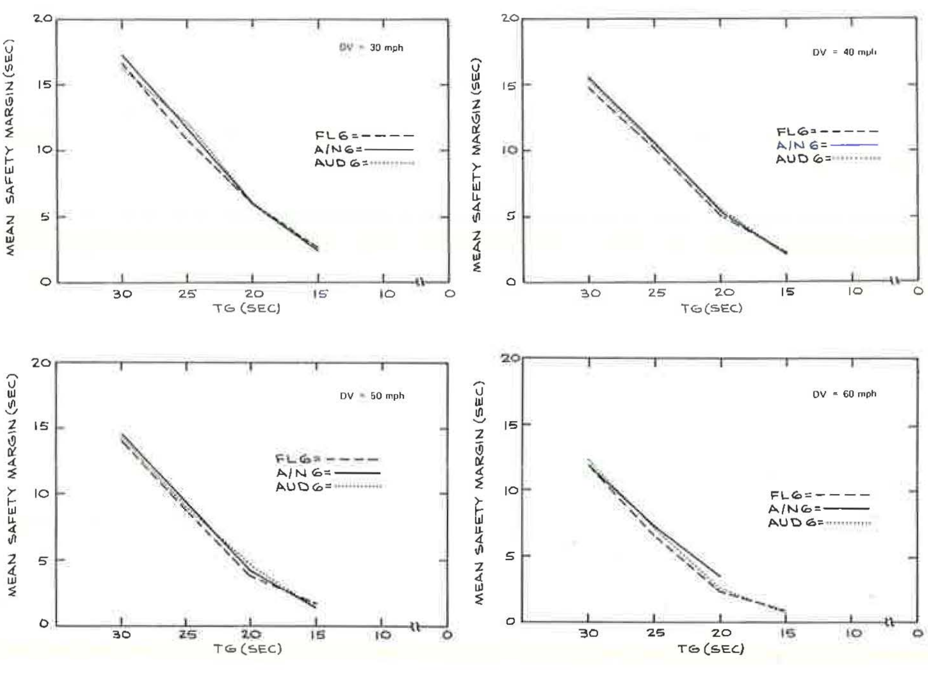

Figure 4. Mean safety margin as a function of time gap and designated velocity for displays.

displays, the functions for 4-message displays being essentially identical. Differences among displays in terms of safety margins were negligible. The only detectable trend in the data was that FL displays were associated with lowest average safety margins at time gaps above 15 sec. At the 15-sec time gap, there was no apparent difference with regard to displays.

Safety margins decreased linearly with available time gap, dropping by about 5 sec with each decreasing TG except at the lowest TG's, where margins decreased proportionately less. The linear decrease was attributable to a general anxiousness to complete the pass, regardless of displayed message, perhaps because of the possibility of abort occurring. At the shortest TG's, however, deviations from linearity were apparent in every case. This would appear to mean that subjects wasted a little safety margin during safe (TG > 15 sec) passes. The nature of the driver sample changed radically, however, when TG was short in that only highly capable and risk-taking (or confused) drivers attempted to pass. (Many other passes were begun but aborted as soon as the AC appeared.) This reduced sample was probably achieving maximum safety margin at every TG. At the shorter TG's the cautious drivers tended not to pass. The increase in the mean safety margin at shortest time gaps appeared to be due, primarily, to selective sampling.

Safety margin also decreased slightly but regularly with increasing designated velocity because of the gradual loss of accelerative capability and the longer prepass headways found at the higher speeds. At 60/15, margins approached zero, and in no case were mean safety margins above 1 sec (Fig. 4), again indicating the hazardous nature of that particular DV/TG condition.

Safety margins obtained by passing during an abort condition followed the same trend with respect to DV as in the nonabort cases. Margins averaged about 2 sec at DV = 30 mph, about 1.25 sec at DV = 40, about 0.75 sec at DV = 50, and less than 0.5 sec at DV = 60 mph. Passing on an abort in a high-speed trial was again highlighted as an extremely hazardous maneuver.

Passing Time

Driver performance was only one factor in safety margin; this margin was determined also by the starting time and speed of the AC. A more direct measure of driver performance was passing time, defined as the interval from display onset until the PC returned to the right lane at the completion of a pass. The passing time, as defined, included decision and response times, time to overtake and pass, and time to return to lane after onset of the LC headlights in the rear of the PC. Prepass headways were fixed at 1 car length per 10 mph, a restriction not normally found on the road but one that was imposed for experimental control. The increase in passing time with increasing DV reflects, in part, this restriction.

Passing times were plotted for the 6 displays at the 4 DV's and 5 TG's. Differences among displays were extremely minor and no trends were apparent. Mean passing times for the DV's and TG's for a representative display, FL-6, are shown in Figure 5. Passing time increases with increasing DV and with increasing TG. The longest passing times occurred at TG = 30 sec. Slight decreases appeared at TG = 25 sec and again at 20 sec. A much larger decrease in passing time occurred at TG = 15 sec, ranging from 1.7 to 3.0 sec. The reduction in passing

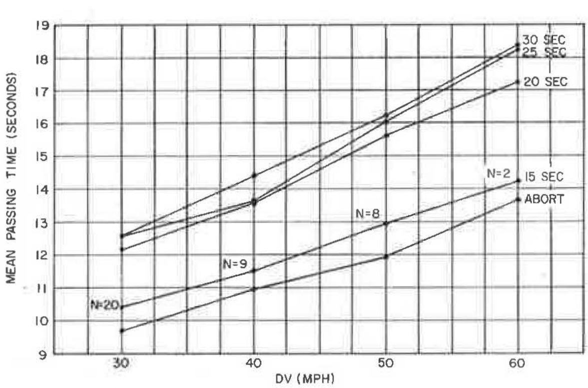

Figure 5. Mean passing time as a function of time gap and designated velocity for FL-6 display.

time varied directly as a function of DV and inversely as the number of drivers passing at 15 sec. However, only a small part, if any, of the reduction in passing time was attributable to variation in the size of the driver sample and elimination of low performers. At 30/15, 20 of the 24 drivers passed. At the same DV and TG = 20 sec, 22 passed. The reduction of the sample by 2 drivers' scores would not have accounted for a reduction in mean passing time of 1.7 sec.

Apparently, drivers were expending maximum effort not at every pass but only when TG became short. Passing times on abort were lowest of all and appeared to represent close to the minimum possible time obtainable with the particular PC used.

A criterion for stability of performance, variance reduction, was applied to the passing time data. Graphical inspection revealed no trend with respect to the small variance differences among DV's. Variances of the distribution of passing times were averaged over DV's and are shown in Figure 6. Performance using the A/N-6 display was associated with variance slightly less than that of the rest of the displays. The A/N-4 display condition had the highest variance at the shorter gaps. No clear distinctions were seen among display types or message lengths.

Passing time variance decreased regularly as TG became short, indicating again that drivers were tightening up their responses as time became critical. The reduction of mean passing time plus the reduction of variance were the drivers' only methods of maintaining their safety margin when they decided to pass.

Two items are deserving of note with regard to passing time variance. First, this statistic did not differentiate among displays, although numerous other measures reported here indicate substantial differences among the media. The lack of sensitivity of variance seemed to imply that passing time may be a rather automatic product of car and driver with the primary modifier being time to pass. Second, a misinterpretation of Farber and Silver (1) might lead researchers into a blanket application of variance reduction as a criterion for safety in measures other than passing distance or time judgments. Application of variance reduction to passing time as an indicant of safety was obviously unwarranted, as variance became smaller as time-to-pass decreased and risk increased.

Accidents

In the present investigation, 13 accidents were observed; accidents were defined as vehicular contact of PC-AC or PC-LC. Probability of an accident in 3,456 total trials given was 0.0038; probability of an accident in 2,007 complete attempts at passes (1,994 passes plus the 13 accidents) was 0.0065. Eleven of the 13 accidents involved AC contact and were classed as head-on or head-on and sideswipe collisions. No PC left the roadway, striking scenery, during the study.

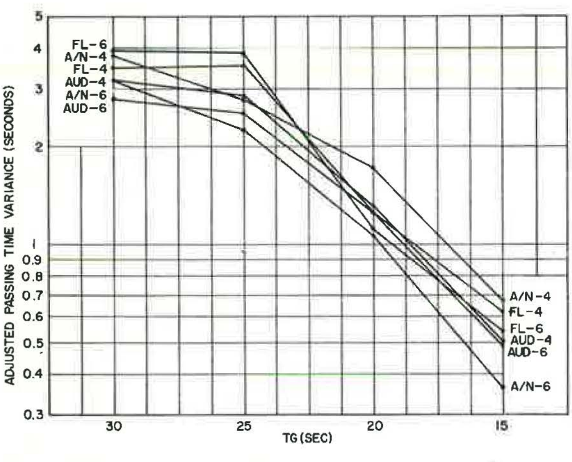

Figure 6. Passing time variance as a function of time gap and display.

The probability of accident during the simulation was considerably greater than in a real road situation because neither the LC nor AC was able to take evasive action, compensating for a poor decision by the PC driver or for an abort emergency. Accidents as stated may thus be interpreted as hazards to which LC and AC drivers would have to react, the required amount of their reaction varying from slight to considerable.

The AUD-4 display was associated with 6 accidents; 2 accidents

happened under the AUD-6 display. Four accidents occurred with A/N-6 and one with A/N-4, all under abort at 60 mph. Accidents did not occur when flashing displays were in use. The 60/abort situation was the most hazardous condition, accounting for 7 collisions, followed by the 60/15 with 3, the 50/abort with 2, and the 30/15 with 1.

The incidence of collision on abort was not a fixed percentage of passing attempts per display. In the 576 abort conditions, 160 successful passes and 9 accidents occurred. Passes were made primarily in the middle range of DV and, except for AUD-4, were made with uniform frequency across displays. The complex relationship of displays to accidents is exemplified in the DV = 60 condition. With A/N-6, drivers made 6 attempts to pass and had accidents 4 times; with FL-6, they succeeded in passing on all 6 attempts.

Only with AUD displays did accidents occur elsewhere other than under abort. A rear-end collision occurred at 30/15 with AUD-6 when the driver veered back into lane at the onset of the fastest signal rate. This was the only accident below DV = 50 mph.

The 3 accidents occurring at 60/15 with the AUD-4 display appeared to have been the result of message confusion. If drivers were interpreting the 2.3 signals per sec tone as a "safe" 1.0 signal per sec tone, and then perceived the 3.6 signals per sec tone as a marginal area, effectively having shifted their go/no-go criterion one step from true message, their first indication of error would have been the appearance of the AC on the belt at a closing rate in excess of 130 mph, were a pass being made. A driver would have about 3½ sec to see the AC, reject his erroneous decision, take evasive action, and clear the left lane. The time-velocity constraints did not seem to permit sufficient time for action with this display.

Decision-Reaction Time

Decision-reaction time (DRT) was defined as the interval from onset of the display to depression of the accelerator. The fastest mean DRT's were associated with AUD-6 (0.61 sec) and AUD-4 (0.64 sec). Intermediate mean DRT's were the A/N-4 (0.66 sec) and the A/N-6 display (0.69 sec). The slowest responses were to the FL displays, 0.70 sec for FL-4 and 0.73 sec for FL-6. The auditory-visual DRT difference paralleled that found in intermodal comparisons of simple reaction time. Differences among displays, although possibly statistically significant, were too small for practical significance.

The other DRT data available were the accelerator or brake responses to the abort signal. These were examined primarily to elucidate the relatively high accident rate during aborts for A/N and AUD displays. Again, variability due to sample size was in evidence. Sometimes during an abort condition the driver stayed on maximum accelerator, which would not permit estimate of his DRT. Two features of abort DRT were worthy of note. The abort DRT was just the opposite of normal DRT in magnitude over the 3 types of displays; abort DRT's in the flashing modes were fastest, followed by alphameric and then auditory. Differences in abort DRT's among displays did not seem to be of sufficient magnitude of themselves to explain the differential accident rate across displays. Second, abort DRT increased as a function of increasing speed, an unexpected result and one that would make aborts at high speeds even more dangerous.

Post-Experiment Subjective Ratings

At the end of the last experimental session, drivers ranked the 6 displays in order of their preferences. The ratings were tested for differential display preferences with the Friedman analysis of variance by ranks, resulting in $\chi_r^2 = 62.86$, which was distributed as chi-square with df = 5, with a probability of chance occurrence <0.001. Drivers had definite preferences for displays. To find out what these preferences were and how stable they were, we compared ratings for each display with every other using a sign test. A significance level of 0.001 was used to ensure that repeated tests would be unlikely to lead to chance significance. These individual comparisons indicated that alphameric and flashing displays were preferred over auditory displays. Alphameric displays were preferred over flashing in 1 comparison. The 6-message systems were not any more highly rated than the 4-message systems. The post-experiment ratings

agreed with questionnaire data taken at the end of each session. Drivers on the average downrated only auditory displays with respect to (a) how much these displays would add to safety on the road and (b) how much, in dollar terms, these displays would be worth.

DISCUSSION OF RESULTS

The major difference between this investigation and others is that this one was a simulated situation whereas other studies have been conducted on the road. In textbook discussions contrasting field and laboratory research, the matter always rests on the comparison of realism versus precision. Several measures gain substantially in precision or become available for the first time only in the traffic simulator. Gains in precision are seen in reaction times based on either of the 2 major pass-oriented responses to a specified stimulus: in prepass and post-pass headways, safety margins, and lateral position on the road at all times. Accuracy is limited, ultimately, only by the amount of error in the simulator and by the level of sophistication of the recording device. Readily interpretable hard copy of the passing performance is a valuable asset. Detailed accident data, the ultimate criteria for highway research, become available, permitting analysis of performance components for etiological significance. A comprehensive analysis of accident data, e.g., reaction times, accelerator, brake, and steering performance, is given in another report (6).

The advantage of field studies has been, undeniably, the realism they have provided, although, except for the purely observational studies, certain artificial procedures have had to be used to safeguard the lives of the subjects involved. The present study, although not intended as a validation of the simulator approach, offers evidence that the situation was realistic. Heartbeat rate and EKG data, not analyzed formally, indicated that most drivers did react autonomically to the driving problem, particularly to "close calls," accidents, and abort situations. Drivers' verbal reports, both outside and inside the simulator, showed their emotions were aroused during these same situations. Expletives, screams, and self-directed conversation were a frequent accompaniment of the insertion of abort, indicating that some stress had been induced. Finally, as is often reported in real-life situations, interviews with drivers after accidents were notably poor in obtaining details. Drivers were often unsure which car they had collided with, and the pen recording had to be the mainstay of accident interpretation. Simulation appeared to be an extremely promising technique for future traffic studies.

In contrast with previous findings indicating driver capabilities were poor (3), drivers showed themselves to be remarkable in their ability to adapt to the diversity of passing problems and information systems, as indicated by the consistency of safety margins, the small but consistent modifications introduced into passing times and time gap thresholds. The one puzzling feature of the data was the increase in abort DRT with increasing velocity, a finding analogous to Crawford's report (4) of increased DRT as passing times dropped below available time gap. The phenomenon could be related to duration of initial decision or the effects of vehicle speed on the decision process. In any case, this factor may be a significant component of the hazards of emergencies at high speeds and deserves further investigation.

The most efficient displays, in terms of number of passes made, were the flashing message units. Graphical analysis indicated that FL displays influenced drivers to make easier passes and fewer hazardous ones; i.e., a driver appeared more able to integrate the performance of his own vehicle and available time gap best when time gap was presented as a color-coded flash rate. Drivers appeared generally cautious when using A/N information and somewhat less cautious with the AUD displays used, particularly at higher speeds. In post-experiment interviews, several drivers commented that the AUD displays were not very clear. When asked why they had passed so much with the AUD displays, the consensus was that they were willing to start a pass in "ignorance" and wait to see what happened. At low speeds and short time gaps they tended to terminate passing when the AC appeared on the highway, even though it was traveling relatively slowly. At high speeds and short time gaps, using the same strategy, drivers often found themselves in trouble. In the AUD-display conditions, drivers seemed far more road-oriented than display-oriented and were behaving as if they were in a low-level or zero-information situation.

A/N displays transmitted the most information per unit time, and this feature appeared adequate for the usual situation. In emergencies this relationship did not hold. Perhaps the time of onset of abort was critical for A/N and AUD conditions but not for the FL display. Inspection of the PC velocities and locations at onset of abort gave no evidence for this view, but it should be throroughly tested in any future investigation of the abort situation. If such was not the case, why was the driver in greater danger during abort with the A/N display than with the other displays?

Amount of information available presumably influences confidence in decision and also set toward response. The stronger the set toward a particular response, the more difficult it is to adapt to changing situations. The A/N display might give the driver too much information, making him less able to selectively adapt to a sudden emergency.

In contrast, the FL displays provided color, a clear indicant of risk status, plus the intermittent signal itself that may have been a stimulus for maintaining high arousal state. The driver was neither dependent on displayed numerical values nor required to process them. Because of the more abstract level of coding in the FL display, the driver appeared to proceed with a healthy degree of doubt. In the event of abort he did not have a strong set to eliminate before taking evasive maneuvers; hence, he was more adaptable.

Was time in seconds really a good and useful display? Drivers tended to prefer this mode, but it is doubtful whether alphameric time readout made any uniquely beneficial contribution to the driver's decision process. The passing driver is processing a considerable amount of information concerning his own car, other cars, and road geometry. It would appear that his spare capacity for relating time messages to the outcomes of each of his sets of options is very limited. A more appropriate display would use a more abstract level of coding, such as color or word messages or both, to communicate the system's prediction to the driver as rapidly and clearly as possible and to more directly relate to the outcome of the driver's decision to pass.

CONCLUSIONS

1. In the investigation of simulated sight-distance-limited passing aided by a time-available information system, data indicated that, of the 3 types of displays tested, advisory information presented by a flashing display was related to the greatest increase in traffic flow and to maximum degree of driver safety.
2. An alphameric display, although serving adequately for normal passing, was associated with a relatively high number of accidents during system-initiated abort, where oncoming traffic suddenly intruded into a previously announced safe-passing zone. Apparently, more than an optimal amount of information was being conveyed, making the driver less adaptive to a radically changed situation.
3. The auditory display used did not communicate time gaps clearly and was inadequate for marginal passing.
4. A comparison of 4- versus 6-message systems indicated no difference in number of passes, safety margins, or accident rate.
5. The study was designed not to test the validity of the PAS II concept but only to provide a comparison of 3 display techniques that were compatible with PAS II equipment. The specific simulation approach to passing behavior employed here resulted in apparently consistent and realistic data and should be used in the future to determine the increase in driver efficiency with a PAS II-type system over a no-display condition. A study to quantify more thoroughly the role of information on driver passing behavior is recommended.

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