TRANSPORTATION RESEARCH RECORD 1026

Evaluation Methods and Design and Operational Effects of Geometrics

TRB
TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. 1985

Transportation Research Record 1026
Price $10.00
Editor: Naomi Kassabian
Compositor: Harlow A. Bickford
Layout: Marion L. Ross

mode
1 highway transportation

subject areas
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54 operations and traffic control
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Printed in the United States of America

Library of Congress Cataloging in Publication Data
National Research Council. Transportation Research Board.

Evaluation methods and design and operational effects of geometrics.

(Transportation research record ; 1026)
1. Roads–Design–Addresses, essays, lectures.
I. National Research Council (U.S.). Transportation Research Board. II. Series.
TE7.H5 no. 1026 380.5 s 85-28523
[TE175] [625.7'25]
ISBN 0-309-03916-9 ISSN 0361-1981

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Expected Traffic Conflict Rates and Their Use in Predicting Accidents

W. D. GLAUZ, K. M. BAUER, and D. J. MIGLETZ

ABSTRACT

The purpose of this research was to establish relationships between traffic conflicts and accidents and to identify expected and abnormal conflict rates given various circumstances. The data on which the conclusions and recommendations are based were collected during the summer of 1982 at 46 signalized and unsignalized intersections in the greater Kansas City area. The conclusions are limited to daytime (7:00 a.m. to 6:00 p.m.) and weekday (Monday-Thursday) traffic and to dry pavement conditions. Accident/conflict ratios have been statistically determined for several types of collisions for each of four types of intersections (signalized high volume, signalized medium volume, unsignalized medium volume, and unsignalized low volume). These ratios can be applied to comparable intersections to obtain an expected accident rate of a specific type after the appropriate conflict data are collected. Also, statistical procedures were developed to determine conflict rate values that could be considered abnormally high. Overall, traffic conflicts of certain types are good surrogates for accidents in that they produce estimates of average accident rates nearly as accurate, and just as precise, as those produced from historical accident data. Therefore, if there are insufficient accident data to produce an estimate, a conflicts study should be very helpful.

The traffic conflicts technique (TCT) has been studied and applied since its early development in 1967 by Perkins and Harris (1). Although it was originally developed to investigate whether General Motors vehicles were driven differently than others, the method was soon used by several agencies to evaluate accident potential and operational deficiencies of intersections. It was believed that a direct relationship existed between accidents and conflicts. However, efforts to verify such a relationship were generally unsuccessful, for a variety of reasons to be discussed subsequently. A review in 1980 by Glauz and Migletz (2) identified 33 previous studies that dealt, at least in part, with conflict-accident relationships.

The use of the TCT did not continue to increase in the United States in the late 1970s; in fact, it declined. However, research did become international in scope, led originally by Canada and England. Now the efforts are widespread and include those of many European and other countries.

Partly in recognition of the widespread interest in TCT and because of the diversity of opinions on its usefulness as well as the definitions and operational procedures, an international workshop was convened in Oslo, Norway, in 1977 (3). That workshop has been followed by others in France, Sweden, West Germany, and Belgium. Although investigators throughout the world have not agreed on the specific operational definitions of traffic conflicts, a universal, generalized definition was generated at the Oslo workshop (3):

> A traffic conflict is an observable situation in which two or more road users approach each other in space and time to such an extent that there is a risk of collision if their movements remain unchanged.

Because the situations are observable and happen at a high frequency (relative to that of accidents, say), conflicts are an enticing traffic measure. The operational differences between investigators are primarily in relation to the severity of the situation--how great the potential risk of a collision was.

Despite such differences, most traffic engineers and analysts believe that traffic conflicts are of value in describing or identifying operational problems at intersections. However, there exist no standards or norms against which to base judgments. How many conflicts per hour or per day suggest a problem? One of the purposes of this paper is to suggest normal and abnormal levels of conflict rates in the United States for certain classes of intersections and types of conflicts.

Perhaps the most important potential application of the TCT, however, is in identifying safety deficiencies. Conventionally, safety is measured in terms of accidents and accident rates--the ultimate measures. Unfortunately, accidents are so rare, statistically, that one must often wait for years, and for many accidents to happen, before enough data are available to enable rational decisions. If a surrogate measure such as traffic conflicts could be used, decisions might be made much more quickly. As noted earlier, however, the heretofore lack of satisfactory agreement between conflicts and accidents has cast this role of the TCT in doubt. The second purpose of this paper is to illustrate that, in fact, a reasonable agreement does exist.

DATA COLLECTION

Definitions

To be useful, the TCT procedures must be formalized and standardized so that investigators can duplicate each other's work. This step was taken in the United States with NCHRP Project 17-3 (2). In that research, the TCT methodology was refined and a standardized set of operational definitions and procedures that were cost-effective was developed.

A traffic conflict was defined in that study, in general agreement with the 1977 international definition (3), as follows:

> A traffic conflict is a traffic event involving two or more road users, in which one user performs some atypical or unusual action, such as an change in direction or speed, that places another user in jeopardy of a collision unless an evasive maneuver is undertaken.

Given this overall conceptual definition, precise operational descriptions for a number of types of conflicts were developed. Of those, 12 were considered in this study:

1. Left turn same direction,
2. Slow vehicle,
3. Lane change,
4. Right turn same direction,
5. Opposing left turn,
6. Left turn from left,
7. Cross traffic from left,
8. Right turn from left,
9. Left turn from right,
10. Cross traffic from right,
11. Right turn from right,
12. Opposing right turn on red.

Detailed definitions of these and other conflicts can be found elsewhere (2). Let it suffice here to provide a few examples. All the conflicts become observable, by definition, when the offending or conflicted vehicle undertakes an evasive maneuver, typically by braking or swerving. Conflict 4, for example, occurs when a vehicle slows to make a right turn, which causes the following vehicle to evade a rear-end collision. Type 5 is instigated by a vehicle turning left in front of an oncoming vehicle. Type 7 involves a vehicle to the left, on a cross street, proceeding across in front of another vehicle, which causes the latter to take evasive action.

Experimental Plan

The methodology published in NCHRP Report 219 (2) was used by Migletz et al. (4) to produce the data required for this paper. The data were collected in the Kansas City metropolitan area (population about 1.5 million). The results are believed to be appropriate for much of the United States, but because of regional differences in driving habits, they may not be directly applicable elsewhere. The results are undoubtedly not usable, numerically, in many countries outside the United States. However, the research approach should be universally applicable.

Traffic conflict and accident data were collected at 46 urban intersections located in four cities in the greater Kansas City metropolitan area. These intersections were stratified, first, according to whether they were signalized and then within signalization class according to intersection traffic volume level (not accident history). The volume levels assigned were

- High: more than 25,000 vehicles per day,
- Medium: 10,000 to 25,000 vehicles per day,
- Low: 2,500 to 10,000 vehicles per day.

The assignment of the 46 intersections to the cells was as follows:

	High	Medium	Low
Signalized	14	12	0
Unsignalized	0	10	10

Traffic conflicts were observed and recorded at each of these intersections for 4 days (replicates) during the period from 7:00 a.m. to 6:00 p.m. over the summer months of 1982. The 11-hr period was sampled in 16 sets of 25-min periods, and the sample counts were then adjusted to be representative of the entire period. Three years of accident data (1979-1981) for these same intersections were also obtained and reduced, as well as special 1-day volume and turning-movement counts.

Conflict Data

Aside from a few rare instances of missing data (in most cases of difficulty, additional data were collected), a total of 576 observer-days of conflict data, representing nearly 90,000 traffic conflicts, was obtained. Table 1 shows the raw conflict counts (along with the accidents) obtained in the study. Of these, 64,210 conflicts were used in the analyses. There were comparatively few wet-pavement accidents and conflicts. Because it was suspected that traffic behavior might be different under dry and wet conditions, the latter were not analyzed in depth. Also, a number of secondary conflicts were observed. These are conflicts created or caused by a vehicle in the process of taking evasive action because of a prior conflicting event. They were dropped from further analysis because corresponding accidents were found to be very rare. Table 2 displays the adjusted conflict rates (conflicts per 11-hr day) by conflict type and intersection class.

Accident Data

Hard-copy accident reports of all accidents occurring at the 46 intersections over the 3-year period (1979-1981) were reviewed. A total of 1,292 accidents made up this data base, given in Table 3. The following types were not used in the ultimate analyses, however:

- Secondary road accidents,
- Wet-road accidents,
- Other accidents such as single vehicle and pedestrian,
- Nighttime accidents (those not occurring between 7:00 a.m. and 6:00 p.m.),
- Weekend accidents (not occurring on Monday through Thursday), and
- Multiple-vehicle accidents not matching one of the 12 conflict types.

TABLE 1 Conflicted-Related Accidents and Conflicts by Road Condition

	Signalized Intersection (N = 26)		Unsignalized Intersection (N = 20)	
Road and Condition	No. of Accidents	No. of Conflicts	No. of Accidents	No. of Conflicts
Dry				
Primary	244	49,337	75	14,873
Secondary	2	14,111	1	3,933
Wet				
Primary	42	3,865	25	972
Secondary	0	1,274	2	255
Total	288	68,587	103	20,033

TABLE 2 Conflict-Related Accidents and Conflicts by Type

Conflict		Signalized Intersection				Unsignalized Intersection			
		High Volume (N = 12)		Medium Volume (N = 14)		Medium Volume (N = 10)		Low Volume (N = 10)	
No.	Type	Accidents/ 3 yr	Conflicts/ Day	Accidents/ 3 yr	Conflicts/ Day	Accidents/ 3 yr	Conflicts/ Day	Accidents/ 3 yr	Conflicts/ Day
1	Left turn same direction	5	1,003.73	3	1,886.14	6	1,327.45	2	706.45
2	Slow vehicle	4	8,028.61	3	5,291.13	1	1,518.31	1	1,018.61
3	Lane change	1	218.53	5	106.70	3	27.97	0	1.05
4	Right turn same direction	2	2,623.50	2	1,742.66	1	616.95	0	579.12
5	Opposing left turn	73	264.01	44	406.80	7	89.82	1	36.40
6	Left turn from left	0	7.57	0	6.48	0	39.13	1	33.66
7	Cross traffic from left	26	1.68	30	4.05	14	32.50	19	66.98
8	Right turn from left	0	0.75	2	4.67	0	1.65	0	5.67
9	Left turn from right	1	5.00	1	7.21	0	43.33	0	49.93
10	Cross traffic from right	19	3.47	14	3.21	6	33.27	12	52.28
11	Right turn from right	7	31.23	1	51.89	1	89.72	0	55.46
12	Opposing right turn on red	1	2.72	0	1.32	–	–	–	–

Note: The values tabulated are totals over the number of intersections in each class (e.g., there were 5/(12)(3) or 0.139 accident/yr of the Left-turn same-direction type of accident at an average high-volume, signalized intersection.

TABLE 3 Accidents by Road Condition

Signalization and Volume Class	Multiple-Vehicle Accidents by Road Condition						Other Accidents[a]	Total
	Dry		Wet		Other or Unknown			
	Primary	Secondary	Primary	Secondary	Primary	Secondary		
Signalized								
High (N = 12)	392	11	103	2	48	1	19	576
Medium (N = 14)	314	2	60	1	28	1	31	437
Unsignalized								
Medium (N = 10)	105	1	30	3	8	0	2	149
Low (N = 10)	82	1	29	0	14	1	3	130
Total	893	15	222	6	98	3	55	1,292

[a]For example, single-vehicle or pedestrian accidents.

A common example of the last type is a rear-end collision at a red traffic signal involving a stopped or stopping vehicle. (The conflict definition does not consider stopping for a red traffic signal to be an "atypical or unusual" action.) The 319 accidents retained are included in Table 3.

EXPECTED AND ABNORMAL CONFLICT RATES

One objective of this paper is to suggest, on the basis of data collected, conflict rates that might be expected or typical for intersections like those studied, as well as abnormal rates. "Abnormal" implies rates significantly greater than average, in a statistical sense. The user who finds such abnormal rates at an intersection should be suspicious, either of the data or of the traffic behavior at that intersection.

One defines abnormal or extreme values statistically by examining the probability distribution of a number of observations. This is typically done by calculating the mean and standard deviation, or variance, and using them to represent the properties of the distribution. However, whereas it is common to establish limits in terms of the mean plus or minus some number of standard deviations, this method is not correct for traffic conflicts or many other traffic measures because it indirectly assumes that the data follow a normal distribution. Traffic conflict data do not behave in that way. The counts can never be negative, for example, and their distribution tends to be skewed, with a longer tail at the higher conflict-count values.

This property of nonnormality for traffic data is well known. Researchers have long used the Poisson distribution for certain data, such as queue lengths, headways, and accidents. The Poisson distribution has a variance equal to its mean. Cursory examination shows this to be far from the truth for conflict data--the variance is often 10 to 100 times as large as the mean. Therefore, a more general distribution should be used.

Early in the research (4), it was suggested (E. Hauer, University of Toronto, unpublished data) that the gamma probability distribution be used. It is very general and can be made to fit a variety of data sets. However, it is more difficult to work with than are the normal or Poisson distributions. The probability density function [$f(c)$] for the gamma distribution is

$$f(c) = te^{-ct}(ct)^{s-1}/\Gamma(s) \tag{1}$$

where Γ is the gamma function, and t and s, both positive, are called the parameters of the distribution. The random variable c is taken to be the daily number of conflicts of a given type associated with one intersection in this paper.

The parameters t and s are defined in terms of the expected value or mean [$E(c)$] and variance [$Var(c)$] of the distribution through the following equations:

$$t = E(c)/Var(c) \tag{2}$$

$$s = t\,E(c) \tag{3}$$

An examination of some typical plots of Equation 1 for selected types of conflicts follows. Figure 1 shows the distribution of all-same-direction conflicts (the sum of types 1 through 4) for signalized medium-volume intersections; it looks much like a normal distribution. The mean value in this case is about 645, and the standard deviation is 159 [= $(25{,}338)^{1/2}$], so individual sample counts can be expected to be much greater than zero but fairly tightly clustered about the mean. Note, however, that the curve is not quite symmetrical. The average value for this type of conflict (645) is slightly to the right of the peak at 605. The value of c at the peak of the curve is called the mode of the distribution. The mode and the mean are the same for a normal distribution; the more they differ, the more the distribution is skewed.

Also shown in Figure 1 are the 90th and 95th percentiles. In this case, 90 percent of all intersections of this class are expected to have less

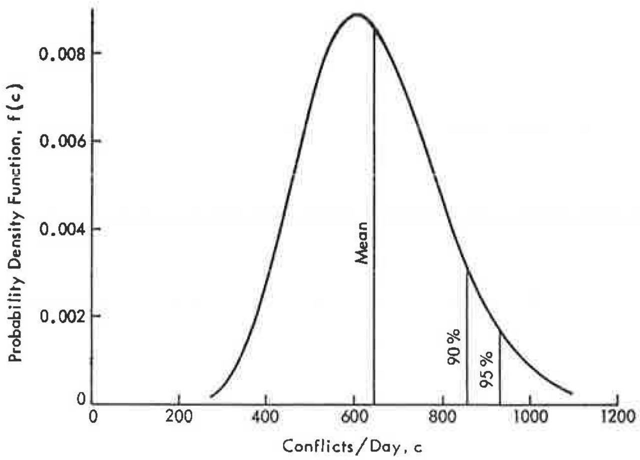

FIGURE 1 Distribution of all-same-direction conflicts for signalized medium-volume intersections.

than 860 conflicts per day of this type, and 95 percent are expected to have less than 930 conflicts per day. In other words, only 10 percent (or 5 percent) of all intersections should be worse than these values indicate. In the remainder of this discussion, limits of 10 and 5 percent will be used as alternative definitions of abnormal conflict rates.

A quite different shape results when the gamma distribution is applied, for example, to opposing-left-turn conflicts for signalized high-volume intersections, as shown in Figure 2. It is highly skewed, with the mean value 5 times as large as the mode (4.8). For this type of conflict, most of the intersections may be expected to have fairly low daily conflict rates--in fact, half will have less than 16 (the median). However, many will have quite large values, so the idea of abnormality takes on a different aspect. Whereas in the previous case the 95th percentile (930) was only 1.44 times as large as the average, in this case an intersection would be required to have nearly three times as many conflicts as the average to be considered abnormal.

A final example shows an even more extreme case (Figure 3). The variance for this type of conflict is so large that the standard deviation, 108 = $(11{,}613.7)^{1/2}$, is greater than the mean of about 84 conflicts per day. In such a case, the gamma distribution has no mode or peak. The value of f(c) becomes increasingly large as c approaches zero. The median is about 42 conflicts per day, so half the intersections should experience less than that rate. The average, however, is about twice as large as the median (~84), and the 95th percentile is nearly 4.5 times the average (360 conflicts per day).

It remains to explain how these limits and other numerical values are determined. The mode is easily calculated as

$$\text{Mode} = (s - 1)/t \tag{4}$$

which is only meaningful if s is greater than 1. The 90th percentile is the value of c (say, c_{90}) for which

$$\int_{c_{90}}^{\infty} f(c)dc = 0.10 \tag{5}$$

That is, c_{90} is chosen so that the area under the curve to the right of that point is only 10 percent of the total.

Equation 5 could be solved by numerical integration with the expression for f(c) given in Equation 1. Alternatively, the integral can be transformed to the probability integral $[Q(\chi^2/v)]$ of the χ^2-distribution, which has been tabulated by several authors (5,pp.978-983).

To use these tables, simply replace v by 2s and χ^2 by 2tc. For example, for the data used in Figure 2, s and t are 1.281 and 0.05824, respectively. Interpolating in the table for v = 2.562, it is found that Q = 0.10 (approximately) for $\chi^2_{90} = 5.55$. Then $c_{90} = \chi^2_{90}/2t = 47.6$. Values of c_{95}, and so on, are obtained in a similar fashion.

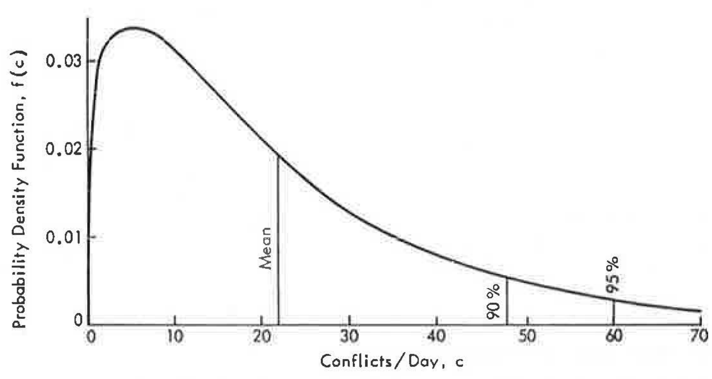

FIGURE 2 Distribution of opposing-left-turn conflicts for signalized high-volume intersections.

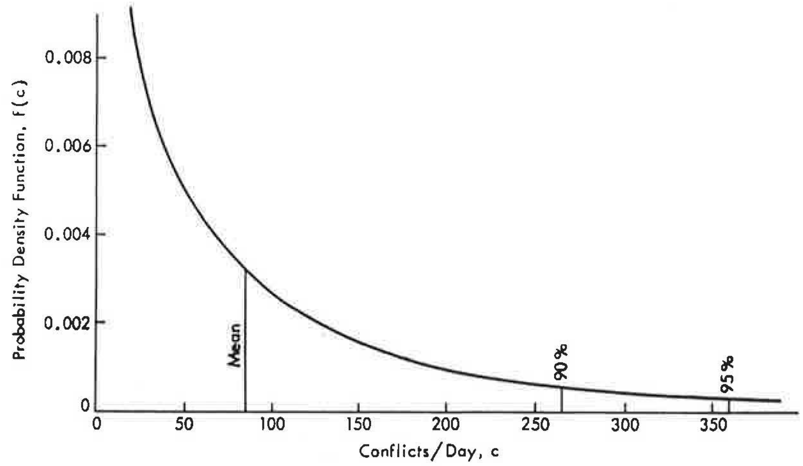

FIGURE 3 Distribution of left-turn-same-direction conflicts for signalized high-volume intersections.

Tables 4 through 7 summarize these calculations. The expected conflict rates (conflicts per 11-hr day) are given in the column headed Mean. The precision of an expected conflict rate is expressed as the standard error of the mean, or $(\text{Variance}/N)^{1/2}$, where N is the number of intersections in the sample. For example, from Table 4 the standard error of the mean for left-turn same-direction conflicts is $(11{,}613.7/14)^{1/2} = 28.8$. When conflict types are so rare that it might be considered abnormal to observe any, no quantitative percentile values are given.

The results given here are based on data obtained from the sample of intersections in this study. It is expected that other users, at least in the United States, should obtain roughly comparable values,

TABLE 4 Daily Conflict Rates for Signalized High-Volume Intersection

Conflict					Percentile[b]	
No.	Type	Mean	Variance	Mode[a]	90th	95th
1	Left turn same direction	83.644	11,613.7	-	265.0	360.0
2	Slow vehicle	669.051	23,994.7	633.0	870.0	940.0
3	Lane change	18.211	160.6	9.4	35.0	43.0
4	Right turn same direction	218.625	7,587.5	184.0	470.0	510.0
5	Opposing left turn	22.001	377.7	4.8	48.0	60.0
6	Left turn from left	0.631	0.824	-	1.7	2.5
7	Cross traffic from left	0.140	0.135	-	-	-
8	Right turn from left	0.062	0.022	-	-	-
9	Left turn from right	0.417	0.261	-	1.1	1.4
10	Cross traffic from right	0.290	0.215	-	-	-
11	Right turn from right	2.603	2.268	0.9	4.6	5.4
12	Opposing right turn on red	0.227	0.124	-	-	-
1-4	All same direction	989.531	67,198.4	921.0	1,340.0	1,460.0
7+10	Through cross traffic	0.430	0.335	-	1.1	1.5

[a]Maximum value of the gamma distribution of conflicts (c) for $f(c) = te^{-ct}(c)^{s-1}/\Gamma(s)$, if a maximum exists.
[b]For the rarest types of conflicts, no values are given; any observed conflicts should be viewed with suspicion. Otherwise, values given suggest limits, at two levels, for normally expected conflict rates.

TABLE 5 Daily Conflict Rates for Signalized Medium-Volume Intersection

Conflict					Percentile[b]	
No.	Type	Mean	Variance	Mode[a]	90th	95th
1	Left turn same direction	134.724	10,298.3	58.0	270.0	340.0
2	Slow vehicle	377.938	4,928.9	365.0	470.0	500.0
3	Lane change	7.621	52.8	0.7	17.0	22.0
4	Right turn same direction	124.476	2,445.1	105.0	190.0	220.0
5	Opposing left turn	29.057	211.2	22.0	49.0	56.0
6	Left turn from left	0.463	0.466	-	1.3	1.9
7	Cross traffic from left	0.289	0.240	-	-	-
8	Right turn from left	0.333	0.188	-	0.8	1.1
9	Left turn from right	0.515	0.125	0.3	1.0	1.2
10	Cross traffic from right	0.229	0.118	-	0.7	1.0
11	Right turn from right	3.707	2.839	2.9	6.0	7.0
12	Opposing right turn on red	0.094	0.058	-	-	-
1-4	All same direction	644.760	25,338.4	605.0	860.0	930.0
7+10	Through cross traffic	0.519	0.215	0.1	1.1	1.4

[a]Maximum value of the gamma distribution of conflicts (c) for $f(c) = te^{-ct}(ct)^{s-1}/\Gamma(s)$, if a maximum exists.
[b]For the rarest types of conflicts, no values are given; any observed conflicts should be viewed with suspicion. Otherwise, values given suggest limits, at two levels, for normally expected conflict rates.

TABLE 6 Daily Conflict Rates for Unsignalized Medium-Volume Intersection

Conflict					Percentile[b]	
No.	Type	Mean	Variance	Mode[a]	90th	95th
1	Left turn same direction	132.745	11,643.4	45.0	275.0	350.0
2	Slow vehicle	151.831	5,921.8	113.0	255.0	290.0
3	Lane change	2.797	22.6	–	–	–
4	Right turn same direction	61.695	1,156.5	43.0	105.0	125.0
5	Opposing left turn	8.982	39.8	4.6	17.0	21.0
6	Left turn from left	3.913	6.452	2.3	7.0	9.0
7	Cross traffic from left	3.250	4.644	1.8	6.0	7.5
8	Right turn from left	0.165	0.077	–	–	–
9	Left turn from right	4.333	21.2	–	10.0	14.0
10	Cross traffic from right	3.327	4.297	2.0	6.0	7.5
11	Right turn from right	8.972	99.4	–	21.0	29.0
12	Opposing right turn on red	–	–	–	–	–
1-4	All same direction	319.068	28,650.5	229.0	540.0	640.0
7+10	Through cross traffic	6.577	15.7	4.2	12.0	14.0

[a]Maximum value of the gamma distribution of conflicts (c) for $f(c) = te^{-ct}(ct)^{s-1}/\Gamma(s)$, if a maximum exists.
[b]For the rarest types of conflicts, no values are given; any observed conflicts should be viewed with suspicion. Otherwise, values given suggest limits, at two levels, for normally expected conflict rates.

TABLE 7 Daily Conflict Rates for Unsignalized Low-Volume Intersection

Conflict					Percentile[b]	
No.	Type	Mean	Variance	Mode[a]	90th	95th
1	Left turn same direction	70.645	1,005.0	56.0	110.0	130.0
2	Slow vehicle	101.861	9,648.2	7.1	225.0	295.0
3	Lane change	0.105	0.050	–	–	–
4	Right turn same direction	57.912	2,197.3	20.0	120.0	150.0
5	Opposing left turn	3.640	8.300	1.4	7.5	9.0
6	Left turn from left	3.366	7.790	1.1	7.0	9.0
7	Cross traffic from left	6.698	42.0	0.4	1.5	19.0
8	Right turn from left	0.567	0.828	–	–	–
9	Left turn from right	4.993	72.7	–	16.0	23.0
10	Cross traffic from right	5.228	11.6	3.0	10.0	12.0
11	Right turn from right	5.546	12.1	3.4	10.0	12.0
12	Opposing right turn on red	–	–	–	–	–
1-4	All same direction	230.523	17,929.2	153.0	410.0	490.0
7+10	Through cross traffic	11.926	75.2	5.6	24.0	29.0

[a]Maximum value of the gamma distribution of conflicts (c) for $f(c) = te^{-ct}(ct)^{s-1}/\Gamma(s)$, if a maximum exists.
[b]For the rarest types of conflicts, no values are given; any observed conflicts should be viewed with suspicion. Otherwise, values given suggest limits, at two levels, for normally expected conflict rates.

although this statement is made without proof. If other parts of the country produce different conflict rates, the user can establish his own expected and abnormal conflict rates by using the procedures explained here.

ACCIDENT PREDICTION

Philosophy

If one wants to know how many accidents have occurred at a specified location, one should review the accident records. Bypassing such records and using a surrogate such as traffic conflicts cannot possibly produce the correct answer.

Unfortunately, this rather obvious concept has usually been overlooked or is unappreciated by researchers and practicing traffic engineers in their search for some measure that might supplant accident data and be used to support difficult decisions. The general approach used to validate a surrogate measure has been to compare observed accidents with the observed surrogate measure, and then to be quickly discouraged and disappointed by the lack of agreement. For example, correlation coefficients of 0.4, 0.6, or even 0.8 are quickly rejected as not being large enough to adequately estimate or predict accidents. Any attempt to match conflicts (or any other surrogate) with accidents in this manner is doomed to failure.

To look at this differently, why would one even want to consider using surrogates? It would not be to identify high-accident locations--the accident data do this. It might be to identify locations with a high accident potential--locations that may be suspected to have safety problems although the accident data do not yet support this. Perhaps it is to determine whether a redesign or countermeasure can be expected to be effective in improving safety without a wait of months or years to establish an accident data base. It might be to determine whether the recent occurrence of a few accidents at an intersection previously presumed safe means that, in fact, the intersection is becoming or has become less safe, for whatever reason. All these potential applications require an estimation or prediction of what may happen in the future--not a duplication of what has happened in the past. In reality, engineers commonly use accident data not just to determine what has already happened; they surmise that the history predicts the future unless changes are made.

Accidents are, in a way, random events. They cannot be predicted except in a statistical sense. Given a substantial accident history for an intersection, one can estimate the expected number of accidents for that intersection in a succeeding year. The actual number of accidents in the succeeding year will undoubtedly be numerically different from this expectation, either higher or lower, but the number will normally be within statistically expected bounds.

It is a major purpose of this research to determine how well traffic conflicts can be used to estimate expected accident rates as distinguished from the number or rate actually observed in any given period. A difficulty then arises--what is the expected accident rate? Two viewpoints will be taken. One is to compare the expected accident rate as predicted by traffic conflict data with the expected accident rate as predicted by historical accident data. The latter is, in effect, the traditional approach; the degree to which the two predictions agree will provide an indication of the validity of traffic conflicts as an accident surrogate. Second, by pooling actual accident data from a number of intersections, years, and so on, another estimate of accident expectations can be derived; both of the foregoing predictions can be compared with this expectation.

In addition to a comparison of estimates of the expected accident rates based on conflicts and on accidents, the quality of the estimates will be assessed as measured by the variance of the estimate. The smaller the variance, the better is the estimate. Whether a certain variance of the estimate is deemed acceptable depends on the variance obtained by other methods of estimation and on the relative costs of estimation by different methods.

Finally, it may be noted that many attempts to prove that conflicts or other measures are satisfactory surrogates failed because the accidents were not suitably disaggregated. Reference to Table 2 shows that most of the conflicts at signalized intersections, for example, involve vehicles traveling in the same direction (types 1 through 4), whereas most of the accidents involve vehicles crossing or meeting head on. If one compared total conflicts and total accidents, one would in effect be comparing conflict movements of one type with accident movements of another type. They are basically unrelated, so no valid statistical relationship should be expected.

Therefore, in this paper only like types of events were analyzed. This has the obvious advantage that if the surrogate (conflicts) is found to be statistically acceptable, it is also logical and defensible. The disadvantage is that it does not deal with total accidents, the ultimate measure that most people feel most comfortable with.

Use of Accident/Conflict Ratios

It is proposed that accident expectation be predicted for an intersection by using conflict data from that intersection in conjunction with accident and conflict data from other intersections of the same class (signalization and volume level). The appropriate equations are

$$\hat{A}_0 = C_0\hat{R} \tag{6}$$

$$\mathrm{Var}(\hat{A}_0) = \mathrm{Var}(C)\mathrm{Var}(\hat{R}) + C_0^2\mathrm{Var}(\hat{R}) + \hat{R}^2\mathrm{Var}(C) \tag{7}$$

where

$\hat{A}_0$ = expected number of accidents,
C_0 = expected conflict rate obtained from the field study at the intersection, and
$\hat{R}$ = estimate of the accident/conflict ratio for that class of intersections (6).

A summary of conflicts and conflict-related accidents by type and class of intersection was given in Table 2. The fractional conflict values arise primarily from the interpolation process used to cover the time periods when conflict observations were not made (7).

Accident/conflict ratios were determined on the basis of reported accident data for 3 years and observed conflict data for 4 days adjusted to 3 years. The accidents and conflicts from a group of similar intersections (for example, signalized high volume) were used to calculate accident/conflict ratios for types of collisions within that group of intersections. Each accident/conflict ratio for a signalization-volume class is the mean value of the accident/conflict ratios of the intersections in that class. The variance of the ratio was taken to be the sample variance of the individual intersection ratios.

Accident Types Subject to Prediction

In the development of accident/conflict ratios, not all types of collisions were analyzed because of a lack of accident or conflict data or both, and some types were pooled to facilitate analysis. The reasons for the choice of the types of collisions analyzed are briefly presented next.

The number of accidents and corresponding conflicts varied considerably from type to type. For most types there was less than one accident per intersection in 3 years. For signalized intersections, because there were so few accidents of types 1, 2, 3, and 4--too few to enable meaningful rate calculations--they were pooled to form a category entitled All Same Direction. In each case, the conflicts are the result of vehicles traveling in the same direction. Even for this pooled category, however, there were no accidents at 12 of the 26 signalized intersections. The opposing-left-turn accidents and conflicts (type 5) showed the best distribution of all the types at signalized intersections. Even here, 7 of the 26 signalized intersections experienced no opposing-left-turn accidents in the 3 years studied.

Note that for signalized intersections, a red-light violation must occur if there is to be a cross-traffic conflict or accident of any kind (types 6 through 11). Such conflicts were observed only rarely. For example, there was a total of only 14 cross-traffic-from-right conflicts observed in 4 days for the 26 signalized intersections (7). This is an average of about 0.13 conflict/day per intersection. To state it differently, one would have to observe all four approaches of an intersection for an average of 7 days to see one conflict of this type. Clearly, such a rare event would not be economically practical as an accident surrogate.

Thus, it is obvious that some sort of pooling is necessary to make cross-traffic conflicts practical. Examination of Table 2 revealed that the most frequent cross-traffic conflict at signalized intersections is type 11, right turn from right, which commonly occurs with illegal right-turn-on-red maneuvers. However, only 6 of the 26 intersections experienced any accidents of this type in 3 years, and none had more than two. The second most common cross-traffic conflicts are those involving left turns, either from the left or from the right (types 6 and 9). Yet there were only two accidents altogether for these two types over the set of 26 intersections. The two conflict types involving through movements of cross traffic (types 7 and 10) were exceedingly rare yet represent the most common type of cross-traffic accident.

In summary, although it might appear desirable to pool the cross-traffic conflicts and accidents, it does not appear legitimate to do so. If pooling did occur, it would be almost equivalent to comparing through-cross-traffic accidents with right-turn-from-right conflicts. Therefore, no further work on cross-traffic accident/conflict ratios at signalized intersections appears warranted.

Finally, the opposing-right-turn-on-red category (type 12) yielded very few conflicts and just one accident. This type involves right-turning vehicles conflicting with opposing-left-turn vehicles with a protected phase (2). Therefore this type, too, was dropped from further analyses.

Examination of the data from the unsignalized intersections also led to decisions about the subsequent analyses of accident/conflict ratios. The left-turn-same-direction data (type 1) for the medium-volume intersections were deemed adequate (marginally) for analysis. They were not combined with the data from the other three same-direction types (2-4), or the type 1 data from low-volume intersections, however. Accidents for conflict types 2 and 4 were very rare. There were very few conflicts of type 3, and although there were three accidents, all occurred at one intersection when one vehicle sideswiped a left-turning vehicle when the first vehicle attempted to pass the second on the shoulder.

As expected, the unsignalized intersections experienced more cross-traffic conflicts than the signalized intersections. Inasmuch as all but one of the cross-traffic accidents involved through movements (conflict types 7 and 10), they were retained; the other cross-traffic data were dropped from further analyses. The opposing-left-turn (type 5) data were retained for the medium-volume intersections but not for the low-volume sites.

To recapitulate, the following accident and conflict types were used in the analysis of accident/conflict ratios:

1. Signalized high- and medium-volume intersections
   a. All same direction (pooled)
      (1) Left turn same direction
      (2) Slow vehicle
      (3) Lane change
      (4) Right turn same direction
   b. Opposing left turn
2. Unsignalized medium-volume intersections
   a. Left turn same direction
   b. Opposing left turn
   c. Through cross traffic (pooled)
      (1) Cross traffic from left
      (2) Cross traffic from right
3. Unsignalized low-volume intersections: through cross traffic (pooled)
   a. Cross traffic from left
   b. Cross traffic from right

Accident Prediction Data

The accident/conflict ratios in Table 8 reveal the large differences from type to type. The all-same-direction type has the smallest accident/conflict ratios, with an average of about 2×10^{-6} accident/conflict. The opposing-left-turn and through-cross-traffic types have ratios of the order of 500×10^{-6} accident/conflict. Thus, it is evident that some types of conflicts are far more likely to yield an accident than other types. Indeed, the differences are of 2 to 3 orders of magnitude.

One might be tempted to impute meanings to the differences in accident/conflict ratios for a given type between intersection classes. For example, the mean all-same-direction ratio for signalized medium-volume intersections is twice that for signalized high-volume intersections (2.663×10^{-6} versus 1.428×10^{-6}). However, the corresponding standard deviations are fairly large compared with the means, which indicates that the data have a lot of scatter. Therefore, the apparent difference might not be statistically significant.

To test for differences in means, one commonly uses the t-test, which is not applicable in this instance because the data are not from a normal distribution, a requirement for using the t-test. Instead, the distributions of the two sets of accident/conflict ratios were compared by using the Kolmogorov-Smirnov test (8), which did not show a significant difference in the distributions. However, this test is known to be conservative when the data sets contain many ties. In this case, 12 of the 26 signalized intersections had no accidents in the all-same-direction category. Repeating the test on the remaining 14 intersections indicated that the two distributions were significantly different at the 5 percent significance level. That is, for signalized intersections having accidents of this type, those with medium volume had higher accident/conflict ratios than those with high volume.

There is a legitimate argument against deleting intersections without accidents--these data are just as valid as those for intersections with accidents. In this case, the accident data base is just too sparse to enable strong conclusions to be drawn. Despite the fact that the difference in the two complete data sets is not statistically significant, combination of the two sets is not believed to be appropriate. Given more data, one might be able to show that a difference exists.

TABLE 8 Accident/Conflict Ratio Statistics

Type of Conflict and Intersection Class	No. of Intersections	Mean Accident/Conflict Ratio[a]	Standard Deviation[a]	Variance		Coefficient of Variation (%)		
				$Var(\hat{R})$[b]	Var(C)[c]	Ratio	Accidents	Conflicts
Left turn same direction, unsignalized medium volume	10	15.024×10^{-6}	31.810×10^{-6}	101.204×10^{-12}	11,643	211.8	151.8	81.3
All same direction								
Signalized high volume	12	1.428×10^{-6}	1.500×10^{-6}	0.189×10^{-12}	67,198	105.4	112.8	26.2
Signalized medium volume	14	2.663×10^{-6}	3.703×10^{-6}	0.979×10^{-12}	25,338	139.1	129.9	24.7
Opposing left turn								
Signalized high volume	12	671.087×10^{-6}	$1{,}002.990 \times 10^{-6}$	83.832×10^{-9}	337.7	149.5	130.3	88.3
Signalized medium volume	14	184.906×10^{-6}	187.500×10^{-6}	2.511×10^{-9}	211.2	101.4	105.1	50.0
Unsignalized medium volume	10	212.456×10^{-6}	293.010×10^{-6}	8.586×10^{-9}	39.8	137.9	135.5	70.2
Through cross traffic								
Unsignalized medium volume	10	735.425×10^{-6}	$1{,}088.780 \times 10^{-6}$	118.544×10^{-9}	15.7	148.0	115.5	60.3
Unsignalized low volume	9	489.229×10^{-6}	302.292×10^{-6}	10.153×10^{-9}	75.2	61.8	78.2	72.7

[a](Accidents/3 yr) ÷ (conflicts/3 yr).
[b][(Accidents/3 yr) ÷ (conflicts/3 yr)]^2.
[c](Conflicts/day)^2.

The same arguments can be made regarding the opposing-left-turn ratios at signalized high- and medium-volume intersections and those for through cross traffic at unsignalized medium- and low-volume intersections. Although the mean values differ considerably, statistical tests are unable to prove the differences to be significant. Nevertheless, it is probably wise not to pool the data.

The standard deviations of the accident/conflict ratios are fairly large. Another way of examining the variability in these ratios is through the coefficients of variation (CVs). The CV is the standard deviation divided by the mean of the accident/conflict ratio. It gives a measure of the relative variation, or imprecision, of the ratio. The CVs obtained are rather high, ranging from 61.8 to 211.8 percent (see Table 8).

A more careful review of the raw data suggests that these high values are largely the result of the variability in the accident data rather than in the conflict data, as is seen in Table 8. In general, higher relative variations in accidents parallel higher relative variations in accident/conflict ratios. The CVs of the conflicts are about half those of the corresponding ratios.

Validation

The procedure for validating the use of traffic conflicts as accident surrogates was as follows. Within each class of intersections, two locations were randomly selected. Accident/conflict ratios were then computed as described earlier but based only on the data from the remaining 38 locations. With the conflict rates and variances obtained from the study, the expected accident rates and their variances were then computed for the selected intersections and compared with those based on the average of the actual accident counts during the years 1979, 1980, and 1981. (No corrections were made for other covariates, such as volume changes, accident trends, etc.)

The computations of the expected accident rates and their variances will be demonstrated on the all-same-direction type of conflict at one of the signalized high-volume sites (Location 19). With the notations given earlier, the computation is as follows:

C_0 = 1,386 conflicts/day from the study;
$\hat{R} = 1.308 \times 10^{-6}$, the average accident/conflict ratio for signalized high-volume intersections (note that this is not the finally recommended value in Table 8, based on all intersections);
$\mathrm{Var}(\hat{R}) = 2.6462 \times 10^{-13}$; and
$\mathrm{Var}(C) = 65{,}697.8\ (\text{conflicts/day})^2$.

Thus, the expected accident rate per 11-hr day will be

$$\hat{A}_0 = C_0 \times \hat{R} = 1{,}386 \times 1.308 \times 10^{-6} = 1.813 \times 10^{-3} \text{ accident/day.}$$

$$\mathrm{Var}(\hat{A}_0) = \mathrm{Var}(C)\ \mathrm{Var}(\hat{R}) + C_0^2\ \mathrm{Var}(\hat{R}) + \hat{R}^2\ \mathrm{Var}(C) = 0.6381 \times 10^{-6}\ (\text{accident/day})^2.$$

In summary, the expected daily all-same-direction accident rate at Location 19 is 0.0018 accident/day, with a standard deviation of 0.0008 accident/day [square root of $\mathrm{Var}(\hat{A}_0)$]. In units of accidents per year on weekdays (Monday-Thursday), these results are adjusted by a multiplication factor of 4/7 x 365, giving 0.38 accident/year with a standard deviation of 0.17 accident/year. This prediction is for that specific type of accident on dry pavement and during daylight hours only. The CV of the expected number of accidents of this type at this intersection is 44.1 percent. These values, based on conflicts and conflict/accident ratios, are to be compared with the expected accident rate of 0.67 accident/year, standard deviation of 1.15, and CV of 173.2 percent based on previous accident rates. These results, along with those for the other validation locations and conflict types, are given in Table 9.

Overall, for this set of intersections and these types of conflicts, the total number of expected accidents based on conflicts is 18.20, very close to the expected number based on accidents (19.67). Both expectations are in good agreement with the observed

TABLE 9 Expected Accident Rates

			Expected Accidents/Yr					
			Based on Conflicts			Based on Accidents		
Intersection and Volume Class	Validation Intersection No.	Type of Conflict	Accidents/Yr	Standard Deviation	Coefficient of Variation (%)	Accidents/Yr	Standard Deviation	Coefficient of Variation (%)
Signalized								
High volume	19	All same direction	0.38	0.17	44.1	0.67	1.15	173.2
	20		0.26	0.13	48.8	0.33	0.58	173.3
	19	Opposing left turn	3.88	3.54	91.2	8.33	1.53	18.3
	20		6.51	4.52	69.4	3.33	2.08	62.5
Medium volume	12	All same direction	0.39	0.19	48.6	0.0	0.0	-
	26		0.35	0.18	50.9	0.33	0.58	173.3
	12	Opposing left turn	0.67	0.64	95.4	1.33	0.58	43.3
	26		1.14	0.70	61.3	0.33	0.58	173.3
Unsignalized								
Medium volume	34	Left turn same direction	0.24	0.56	233.4	0.0	0.0	-
	46		0.26	0.56	220.7	0.0	0.0	-
	34	Opposing left turn	0.12	0.24	205.6	0.33	0.58	173.3
	46		0.08	0.23	299.7	0.33	0.58	173.3
	34	Through cross traffic	1.42	1.13	79.5	1.67	1.15	69.3
	46		0.70	0.88	126.6	0.33	0.58	173.3
Low volume	27	Through cross traffic	0.93	0.97	104.4	1.0	1.0	100.0
	33		0.87	0.97	111.4	1.33	1.15	86.6
Total[a]			18.20			19.67		

[a]Actual total in 1982 = 20.

number of 20 in 1982. This suggests that conflicts are nearly as good as accidents in predicting expected accidents, at least for the total from this sample of 16 predictions.

One can then ask how the two sets of 16 predictions fit the set of 16 observations. Do the predictions from one set tend to be better (closer) than those from the other? The expectations based on accidents were closer in nine cases, the expectations based on conflicts were closer in six cases, and there was one tie. Statistically, there is no evidence that either set of predictions is more likely to be closer more often than the other. Pursuing this a little farther, one can ask whether the magnitudes of the errors in fitting the two predictions to the set of observations are the same. Wilcoxon's signed rank test (8) was used to test this hypothesis, showing no significance at the 95 percent level. At the 90 percent confidence level, the accident-based predictions are marginally closer than the conflict-based predictions. It is noted that the conflict-based expectations were closer than the accident-based expectations more often for unsignalized intersections, and the reverse was true for signalized intersections. However, the data set is too small to allow convincing generalizations.

Another way of comparing the two estimation procedures is to examine their variances (precision). This can be done by comparing the CVs (standard deviation/mean) obtained in both cases. Again, Wilcoxon's signed rank test was used. It showed no evidence that one method produces, on the average, more precise predictions than the other method. In some instances the conflict-based expected value is more precise, and in other instances the accident-based value is more precise.

Effect of Volume

It has been noted that conflicts and accidents are both correlated with intersection volumes (2), suggesting that conflict-accident relationships may exist because of this volume effect. In order to minimize the influence of volume, the intersections were stratified by volume level, as discussed earlier. It is nevertheless appropriate to question whether the stratification effectively removed the volume effect.

Volume counts were obtained during the research. The actual volumes in any cell of the design differed only by a factor ranging from 1.85 to 2.50 (7). Correlation analyses were performed, within cells, between the conflict types in Table 8 and the corresponding intersection volumes, with the results shown in Table 10. In most cases the correlations are far from significant, and in some instances they appear to be negative. The exception is for signalized medium-volume intersections where the correlations for the two conflict types considered are significant ($p = 0.07$ and 0.04, respectively). In these cases the regression accounts for 25 or 31 percent (R^2) of the variance in the conflict counts.

Thus, although some of the variation in conflict counts can be explained by differences in volumes for signalized medium-volume intersections, the amount explained is not large. And for the other intersections there is no detectable effect of volume.

Minimum Variance Predictions

Two sets of predictions (expectations) have been discussed--one based on conflicts and one based on accidents. There is no conclusive evidence that one is more accurate or precise than the other. However, the two sets of expectations can be combined to yield expected values with variances less than those for either set alone. If $\hat{A}_a$ is the expected accident rate based on accident data, then $\hat{A}_m$, the expected accident rate with minimum variance, can be computed as follows:

$$\hat{A}_m = [\hat{A}_0/\mathrm{Var}(\hat{A}_0) + \hat{A}_a/\mathrm{Var}(\hat{A}_a)]\mathrm{Var}(\hat{A}_m) \quad (8)$$

where

$$\mathrm{Var}(\hat{A}_m) = 1/[1/\mathrm{Var}(\hat{A}_0) + 1/\mathrm{Var}(\hat{A}_a)] \quad (9)$$

Thus, Equation 8 yields a more precise estimate of the expected accident rate than do either accidents or conflicts alone. The results are shown in Table 11.

CONCLUSIONS

This study culminated in the following conclusions:

1. A fundamental difficulty with a study of this kind is the rarity of accidents, the very reason that one searches for accident surrogates in the first place. The 1,292 total accidents in the 3-year, 46-intersection data base yielded an average of only about 28 accidents per intersection. After those accidents that involved single vehicles, nighttime, adverse pavement conditions, and so on, were deleted, only 319 accidents (about 7 per intersection in 3 years) remained that could be considered conflict-related. Further subdivision into 12 conflict types yielded a sparse data set indeed.

2. There are 12 basic conflict types that are possible, according to NCHRP Report 219 (2). Of these, some are fairly common, but others are so rare that they are impractical for operational applications. At signalized intersections same-direc-

TABLE 10 Correlations Between Intersection Volumes and Conflicts

Intersection and Volume Class	Type of Conflict	N	Correlation Coefficient (R)	Probability (p)
Signalized				
High volume	All same direction	12	0.34	0.28
	Opposing left turn	12	−0.26	0.41
Medium volume	All same direction	14	0.50	0.07
	Opposing left turn	14	0.55	0.04
Unsignalized				
Medium volume	Left turn same direction	10	−0.04	0.91
	Opposing left turn	10	−0.04	0.91
	Through cross traffic	10	0.10	0.77
Low volume	Through cross traffic	10	0.37	0.30

TABLE 11 Minimum Variance Accident Expectations

Intersection and Volume Class	Validation Intersection No.	Type of Conflict	Expected Accidents/Yr Conflict-Based	Accident-Based	With Minimum Variance	Variance Conflict-Based	Accident-Based	Minimum
Signalized								
High volume	19	All same direction	0.38	0.67	0.39	0.029	1.32	0.028
	20		0.26	0.33	0.26	0.017	0.34	0.016
	19	Opposing left turn	3.88	8.33	7.63	12.5	2.34	1.97
	20		6.51	3.33	3.88	20.4	4.33	3.57
Medium volume	12	All same direction	0.39	0.0	0.0	0.036	0.0	0.0
	26		0.35	0.33	0.35	0.032	0.34	0.029
	12	Opposing left turn	0.67	1.33	1.03	0.41	0.34	0.19
	26		1.14	0.33	0.66	0.49	0.34	0.20
Unsignalized								
Medium volume	34	Left turn same direction	0.24	0.0	0.0	0.31	0.0	0.0
	46		0.26	0.0	0.0	0.31	0.0	0.0
	34	Opposing left turn	0.12	0.33	0.15	0.058	0.34	0.050
	46		0.08	0.33	0.11	0.053	0.34	0.046
	34	Through cross traffic	1.42	1.67	1.54	1.28	1.32	0.65
	46		0.70	0.33	0.44	0.77	0.34	0.24
Low volume	27	Through cross traffic	0.93	1.00	0.96	0.94	1.00	0.48
	33		0.87	1.33	1.06	0.94	1.32	0.55

tion conflicts are common, as are opposing-left-turn conflicts. Cross-traffic conflicts at signalized intersections can occur only if a driver violates the red signal phase and are exceedingly rare (with the exception of the right-turn-from-right conflict, which is observed more frequently although it still indicates a violation of the usual right-turn-on-red ordinances). At unsignalized intersections, all-same-direction conflicts are also common, except for those resulting from lane changes. Cross-traffic conflicts are much more prevalent at such intersections compared with signalized intersections.

3. Considering the rarity of certain conflict types and the infrequent occurrence of some accident types, emphasis in applying the TCT as a safety indicator must be placed on a limited subset of conflict types. It is not practical to use conflict types that require excessively long periods to observe adequate samples. Likewise, there is little incentive to collect data on conflict types for which corresponding accidents hardly ever occur. Thus, the practical, usable conflict types are the following:

1. Signalized intersections
   a. Same direction (pooled types 1, 2, 3, and 4)
   b. Opposing left turn (type 5)
2. Unsignalized intersections [through cross traffic from left and right (pooled types 7 and 10)]
3. Unsignalized intersections, medium volume only
   a. Opposing left turn (type 5)
   b. Left turn same direction (type 1)

4. An estimate of the expected rate of accidents of a specified type and for a specified class of intersections can be computed from data obtained in a field conflict study. If the conflict study at the intersection produces an average conflict rate of C_0, the expected accident rate is $\hat{A}_0 = C_0\hat{R}$. Values of $\hat{R}$, which are the accident/conflict ratios obtained in this research for the various conflict types and intersection classes, are presented in Table 8, along with their variances. The latter can be used to estimate the variance in the expected accident rate by using Equation 7.

5. Accident/conflict ratios differ substantially from conflict type to conflict type, ranging from as low as 1 or 2 accidents/million conflicts for same-direction conflicts at signalized intersections to as high as about 700 accidents/million conflicts of the opposing-left-turn and through-cross-traffic types. (The latter ratios are for unsignalized intersections only; at signalized intersections there are about 10,000 accidents/million through-cross-traffic conflicts, but the rarity of this type of conflict precludes an accurate estimate.)

6. The variation in accident/conflict ratios is generally quite large (CVs up to about 200 percent), indicating a substantial difference among intersections of normally the same type. This variance arises primarily from the intersection-to-intersection differences in accidents, whose CVs match those of the ratios quite well. The CVs of the conflicts, on the other hand, are only about half as large.

7. Comparisons of accident/conflict ratios between classes of intersections suggest that there are differences, but statistical tests, for the most part, are not able to establish this with confidence. This is because of the large variances noted earlier, as well as the substantial number of intersections having no accidents of a specified type during the 3 years analyzed. Despite the lack of proof of such differences between intersection classes, it is probably unwise to combine the data from different classes of intersections to obtain universal ratios.

8. The conflict rates obtained and used to determine the accident/conflict ratios are the average or expected values. Procedures were developed to determine values that could be considered abnormally high. Basically, the procedure utilized calculated probability distributions (the gamma distribution) and accepted as abnormal by definition those rates that exceeded the 90th percentile (alternatively, the 95th percentile). The values obtained are given in Tables 4 through 7.

9. If a potential TCT user determines that his conflict rates and variances differ substantially from those obtained in the U.S. Midwest during this study, he will have to adjust the values given in Tables 4 through 7. The procedure is described in the text.

10. The proper use of conflicts is to estimate an expected rate of accidents as opposed to predicting the actual number that might occur in a particular year. Accident data fluctuate greatly from year to year; the best one should expect is to be able to estimate the average (expected) value with acceptable accuracy and precision.

11. An additional year of accident data (1982) for eight intersections was used to determine the validity of the proposed accident estimation procedure. Accident estimates based on conflicts were compared with accident estimates based on accident history. Overall, for the eight intersections, both methods produced about the same estimates--18.20 accidents on the basis of conflicts, 19.67 on the basis of previous accidents. (There were actually 20 conflict-related accidents in 1982 at the eight intersections.) Breaking these down to the 16 possible combinations of intersections and conflict types indicated that both procedures sometimes overestimated and sometimes underestimated the actual number of accidents. In this respect, the accident-based procedure yielded closer estimates more often than the conflict-based procedure, but only marginally so.

12. Of the 13 out of 16 sets of accident estimates for which CVs could be calculated, those based on accidents were more precise in 8 cases and those based on conflicts were more precise in 5 cases. This difference is not statistically significant; in other words, the conflicts procedure produces estimates equally as precise as those based on accident histories.

13. If one has estimates of expected accidents based on both accident history and conflict data, they can be combined to produce an estimate that is more precise (smaller variance) than would be obtained by using either one separately.

14. Overall, traffic conflicts of certain types are indeed good surrogates of accidents in that they produce estimates of average accident rates nearly as accurate and precise as those produced from historical accident data. Therefore, if there are insufficient accident data to produce an estimate, a TCT study should be very helpful.

ACKNOWLEDGMENT

The authors gratefully acknowledge the contributions made to this research by Ezra Hauer of the University of Toronto and a consultant to Midwest Research Institute, who provided many helpful suggestions, particularly during the planning stages of the research.

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Publication of this paper sponsored by Committee on Methodology for Evaluating Highway Improvements.

Stopping Sight Distance Parameters

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ABSTRACT

A review of stopping sight distance parameters has recently been completed for NCHRP. AASHTO currently recommends a driver perception-response time of 2.5 sec and this value was found to be satisfactory. AASHTO currently uses braking distances based on locked-wheel skidding on poor-condition wet pavement surfaces. It was concluded that this is not appropriate for speeds above 30 mph if a vehicle with minimum legal tire tread is to be stopped in its own lane on a wet pavement of this type. For a vehicle to be able to make such a stop it was concluded that braking distances should be increased. At a speed of 40 mph, the distance increases to 360 ft and at 80 mph it increases to 1,630 ft. Examination of recently measured speed distributions showed that drivers continue to select the same speeds on wet pavements as they do on dry roads and that the AASHTO policy of using the same initial speed for both wet and dry conditions should be retained. Lowering the driver eye height to 40 in. from the current AASHTO value of 42 in. would accommodate more than 95 percent of the automobile driver-vehicle combinations expected to be in use late in this decade. Such a change was recommended because a 42-in. eye height would not accommodate 25 percent of the vehicles. No research on the appropriate height of the object was performed. Ten vertical curve locations at which there was less than AASHTO policy minimum available stopping sight distance were found to have an average of about 40 percent more accidents than nearby locations with adequate sight distance. Several horizontal- and vertical-curve geometric design aids based on derivations made in the research are presented.

Stopping sight distance (SSD) is one of the most important criteria in geometric design, affecting both operations and safety. It is defined as the minimum sight distance that will allow a vehicle traveling at or near the design speed to stop just before reaching an object in its path, and it is important that this design element be frequently reviewed in response to changing vehicle and driver characteristics. The University of Michigan's Transportation Research Institute (UMTRI) was selected to carry out such a study. The final report was recently published in the NCHRP series (1). This paper summarizes the research, emphasizing those findings believed to be of particular importance in highway design and traffic control.

SSD application involves considering two concepts, the stopping distance (STD) and the available sight distance (ASD). The ASD depends on the locations of the eye of the driver, the object to be seen on the road, and obstructions to the line of sight caused by the geometry of the road and roadside. SSD is adequate when ASD is greater than STD and inadequate when the opposite condition exists.

STD consists of a perception-response distance (PRD) added to the braking distance (BD). When the speed (V) of the vehicle is considered, PRD is derived from the perception-response time (PRT). The STD on a level road is expressed as follows:

$$STD = 1.47\ V\ PRT + V^2/30\ f \qquad (1)$$

where f is the average deceleration from V to a stop (g). Although every significant parameter in the STD model is stochastic, the model is treated deterministically and the parameters used are drawn from that end of the probability distribution that accommodates poorer performance and results in greater STD values.

This paper is organized into three sections. A study of the effects of ASD on safety is summarized first. Next the three STD elements--initial vehicle speed (V), PRT, and BD--are discussed. The effects of grade and horizontal curvature on BD are considered. The studies concerned with ASD elements, eye and object height and road geometry, are described in the last section. The effects of vertical curvature on ASD for passenger cars and trucks and the sensitivity of ASD to the location of the object and eye in the lane on both horizontal and vertical curves are treated. Night effects on ASD are also considered.

SAFETY STUDIES

It is accepted that SSD has impacts on highway safety but the relationship has not been identified or recently quantified with enough accuracy to be used in evaluation studies. A review of the several studies of the relationship between SSD and safety is included in NCHRP Report 270 (1).

The problem with most of these studies is that it is difficult to separate sight distance effects from other roadway design elements and to maintain proper controls. A limited study of the effects of ASD on safety on tangent sections was carried out as a part of the research.

The number of accidents over a 6-year period was compared at 10 pairs of two-lane rural road segments in close proximity. The sites are located in Oakland and Washtenaw counties in southeastern Michigan. They were matched for traffic characteristics, road design factors, roadside features, traffic control, and abutting land use. The two segments were within 1 mi of each other on the same road with no major intersections between them. One segment was on a vertical curve and had an ASD that was less than the 1965 AASHTO policy value, the current minimum value (2), whereas the other had an ASD exceeding this value. Each limited-ASD (LSD) site had a standard warning sign with a speed advisory plate. Table 1 presents a description of the sites and a summary of the accident data.

TABLE 1 Summary of Safety Study

Site Pair	Site Type	Length (mi)	Speed Limit (mph)	Advisory Speed at LSD Site (mph)	ASD (ft)	No. of Accidents
1	LSD	0.50	45	40	118	11
	Control	0.50	45		>700	3
2	LSD	0.23	50	40	276	1
	Control	0.23	50		536	0
3	LSD	0.40	50	25	188	2
	Control	0.40	50		>700	2
4	LSD	0.25	45	30	174	7
	Control	0.25	45		>700	6
5	LSD	0.22	45	30	118	11
	Control	0.22	45		>700	3
6	LSD	0.25	45	30	250	17
	Control	0.25	45		>700	26
7	LSD	0.24	45	35	262	24[a]
	Control	0.24	45		>700	13[a]
8	LSD	0.15	50	40	308	5
	Control	0.15	50		>700	2
9	LSD	0.17	50	40	280	2
	Control	0.17	50		>700	1
10	LSD	0.20	25	-	223	0
	Control	0.20	25		>700	0
Total	LSD					80
	Control					56

Note: LSD = limited sight distance.
[a]Only 4 yr of accident data were available.

There was a total of 136 accidents for 30.28 mi-years of exposure. Of these, 80 accidents occurred on the LSD sites and 56 occurred on their matched control sections. At seven of the site pairs there were fewer accidents on the control section; in two cases there was a tie; and at only one site were there more accidents on the section with greater ASD. No accident-type differences were apparent.

The group totals were analyzed by standard contingency table techniques. The hypothesis of no significant difference in accident frequency between the LSD sites and the control sites was rejected at the 0.05 level. Hence, it was concluded that the approximately 40 percent more accidents at the LSD sites were not due to chance. It is believed that a larger study of this type should be conducted to confirm and develop a more reliable quantification of the effect of vertical-curve ASD on safety. Studies should also be made on horizontal-curve pairs with only STD varying.

STD ELEMENTS

In this section studies of the three parameters of the STD equation--initial speed (V), perception-response time (PRT), and braking distance (BD)--are summarized. In addition, sensitivity analysis and some interactions with geometric design elements are explored. Recommended changes in AASHTO policies are presented and discussed.

Initial Speed

It was once assumed that motorists travel at a slower speed, the operating speed, on wet pavements than they do on dry roads. The 1965 AASHTO policy (2) used the design speed for dry pavements and the operating speed was assumed for wet pavements for V in STD calculations. Since 1971 the policy (3) has been to use the same speed (the design speed) for both wet and dry conditions. A study of motorist speed behavior was conducted to test the current validity of this policy. Speed distributions were analyzed from 106 rural sites with 55-mph speed limits in five states. The data had been recently collected for the national speed-limit monitoring program (4) for rural Interstates, principal and minor arterials, and major collectors.

Statistical tests of a 10 percent sample of the available 900 daylight hourly distributions indicated that they could be treated as normally distributed at the 0.05 level of confidence. Visual inspection of cumulative plots of the remaining data confirmed this conclusion. This supported the finding that speeds on rural highway facilities are often normally distributed and in this case permitted the use of statistical techniques based on the assumption of normality.

The daylight speed distributions recorded at a set of 25 permanent Illinois speed-monitoring stations for which reliable weather information was also available were compared under wet and dry pavement conditions. Speed data were obtained for up to 3 days per site on days on which it was known to have rained for the whole day and for adjacent days when there had been no rain. An analysis of variance of the hourly speeds revealed no difference in the average and accordingly they were aggregated to provide a daylight total.

The daily cumulative speeds at a site were then compared for rainy and dry days. Generally, the differences between wet and dry pavements were not statistically significant and were never practically important. Figure 1 shows the distribution of the differences in 85th- and 95th-percentile speeds for wet and dry conditions at 25 sites. At these important higher speeds, those of most concern in the determination of STD, the wet and dry pavement speeds are practically indistinguishable. This confirmed the validity of the AASHTO policy not to treat wet and dry pavements differently.

Driver PRT

SSD PRT covers four steps. The driver must detect an obstacle, identify it as a significant hazard, decide to stop, and begin the stop. The case of particular interest in the STD context is the surprise situation in which the motorist is not aware of the presence of an object on the road ahead. In the primary PRT study, subjects drove an instrumented vehicle for several miles for familiarization. They then crested a sharp vertical curve on a tangent section and encountered a surprise in the form of a low-contrast obstacle shaped like a short railroad tie centered in the lane of travel on the reverse slope of the crest. Time and distance measurements were made from when the obstacle first became visible to when the subject removed his foot from the accelerator (perception time) and then from the accelerator release to brake pedal contact (response time). After the surprise encounter the test was repeated several times on the same subjects under "alerted" conditions. These trials required only

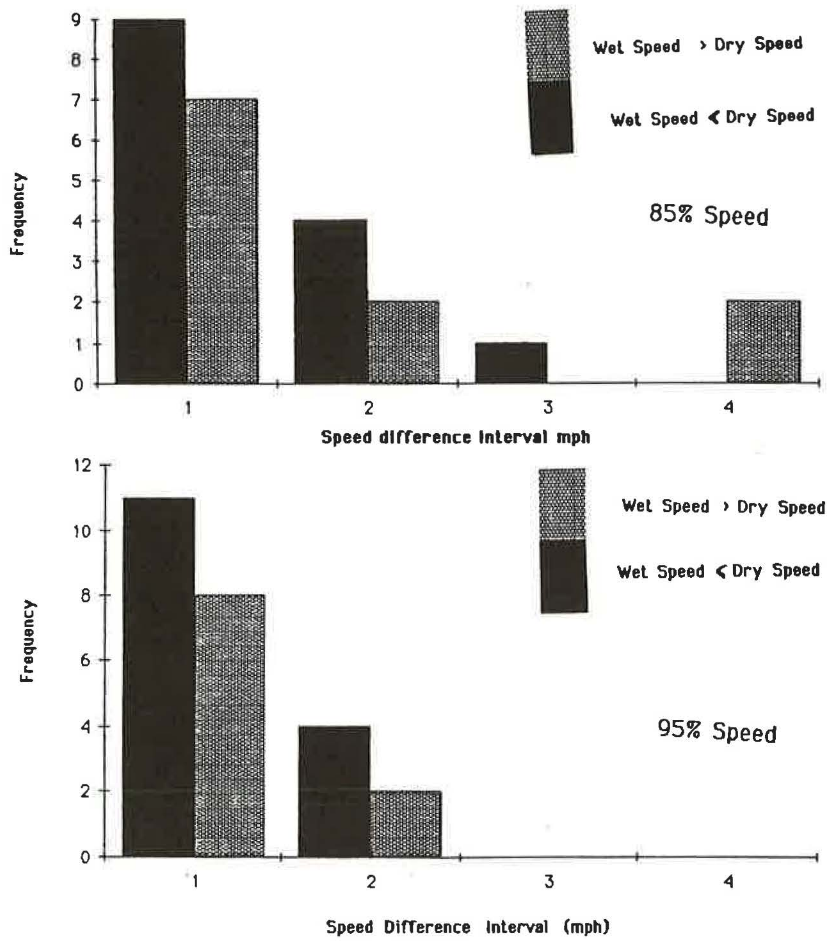

FIGURE 1 Speed differences under dry and wet conditions at the 85th and 95th speed percentiles.

that the subject tap the brake pedal. Finally, in a different driving environment, the subjects released the accelerator and tapped the brake pedal in response to the lighting of a red lamp mounted on the hood of the test car (brake trials).

A total of 64 subjects, 49 younger than 40 years of age and 15 older than 60, was studied. The data for the younger drivers from this study are presented in Figure 2 on a cumulative normal probability scale. The most relevant finding is that for the surprise condition, the 5th- and 95th-percentile values of the PRT were 0.85 and 1.6 sec, respectively. The PRT for the older drivers was substantially the same.

The subjects used in this study, however, were not fully representative of the normal driving population. Their driving times before the tests were short, they knew that they were involved in an experiment of some kind, and they did not appear fatigued or under the influence of alcohol or drugs. Such conditions would be expected to affect the PRT. Studies of the effects of drugs and alcohol indicate that a 50 percent increase in PRT is reasonable (5). Such a correction leads to a 95th-percentile value of 2.4 sec. This is a reasonable percentile for design and is so close to the current 2.5-sec AASHTO policy value that it was concluded that the current value should be retained.

An important factor not considered quantitatively here is the object contrast. The foregoing data are based on a relatively low-contrast condition. However, worse values are possible and this would cause a further increase in the required PRT. There is no information on the distribution of contrasts for real obstacles encountered in actual driving situations, and hence no estimate of the magnitude of this additional correction was made. However, a limited field study of the response time to some object characteristics was made. Seven widely varying conditions with different obstacle height, width, and contrast were evaluated by using 26 observers. The difference in response time among the seven conditions had a range generally of about 0.2 sec, except that for the 95th-percentile observers the range was 0.4 sec and the 98th-percentile subjects had a range of about 0.5 sec. Where there was a great contrast between the obstacle and the background the response time was shorter. It was also observed that a high narrow object that was in poor contrast to the natural background foliage found at this study site required a longer response time.

BD

BD was viewed in the research as being made up of three parts--the basic capability of the tire-road interface to decelerate the vehicle, a measure of the efficiency of the vehicle's braking system under varying loads, and a driver control strategy, which may not use all of the available braking capability, depending both on driver skill as well as on choice. In stops from high speeds the contribution to the

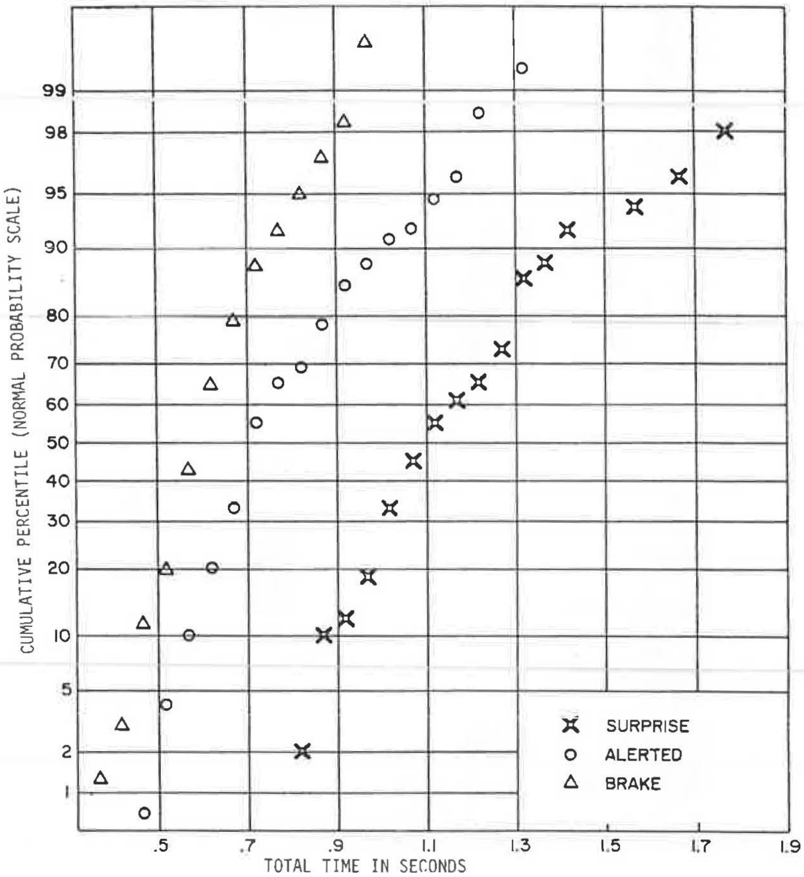

FIGURE 2 Perception-response times for younger drivers.

braking distance from aerodynamic drag also becomes important.

AASHTO policies view the driver as applying the brakes sufficiently hard to lock the wheels; the deceleration then depends only on the condition of the pavement and tires. The road condition is measured by the skid number, a function of the velocity and the pavement texture depth. The condition of the tires is measured by the depth of the treads. In a locked-wheel stop it is assumed that all the available friction is utilized for deceleration.

However, it has been found that drivers generally do not decelerate by locking the wheels but modulate their braking effort in an attempt to minimize BD and maintain directional control and stability. This appears to be particularly true at high speeds on wet pavements. The question then becomes one of determining how deceleration depends on the capability of the vehicle brake system to utilize the friction available at the interface among vehicle, tire, and pavement and the ability of the driver to modulate braking control.

The maximum friction available at the tire-pavement interface in controlled deceleration is greater than that available in the locked-wheel situation, but vehicle braking systems are not capable of utilizing all of the available friction. The term braking efficiency (BE) is used to express the percentage of tire-pavement friction that a perfect driver could achieve and yet maintain control over the vehicle. The braking capability of passenger vehicles has improved significantly over the last decade. The average BE of a 1982 model passenger car is 0.91 (6). The BE of heavy trucks is not as great as that attained by passenger cars. Because truck BE depends on the vehicle geometry, weight, and load distribution, it is best determined separately for each truck configuration.

The ability of a driver to bring the vehicle to a controlled stop is measured by the control efficiency (CE). Analysis of experimental data collected (7) shows that the CE for passenger car drivers decreases with increasing initial speed. In addition a limited set of experiments performed in this research indicates that professional drivers of heavy trucks do not achieve a CE of more than 0.62.

The relationships developed to calculate the instantaneous coefficient of friction (μ) between the road and tires for locked-wheel and controlled decelerations for passenger cars and trucks are given as follows. (The aerodynamic drag deceleration component, which is not shown, is a function of the vehicle velocity and its frontal area and weight.) For a locked-wheel stop, the coefficient for passenger cars is

$$\mu = 0.012\ A\ SN_V \tag{2}$$

For trucks it is

$$\mu = 0.0084\ A\ SN_V \tag{3}$$

where

$SN_V = SN_{40} \exp[-0.0016(MD^{-0.47})(V - 40)]$,
V = velocity (mph),
MD = mean pavement texture depth (in.) (sandpatch method), and
$A = 1 + (5.08MD - 0.008045V)[1 - (x/12)^{1/2}]$ where x is tire tread depth in 1/32 in. (except for tread depths > 12/32 in., x = 12).

For a controlled stop, the coefficient for passenger cars is

$$\mu = (0.2 + 0.01344SN_V)\ A\ BE_{car}\ CE_{car} \quad (4)$$

For trucks it is

$$\mu = 0.01218\ SN_V\ A\ BE_{truck}\ CE_{truck} \quad (5)$$

where

BE_{car} = 0.91,
BE_{truck} = BE (truck geometry, weight, load distribution) determined for each truck configuration,
CE_{car} = $0.267 + (0.0808 + 0.00543SN_{V_I})A_{V_I}$,
V_I = initial velocity,
A_{V_I} = value of A evaluated at V_I, and
CE_{truck} = 0.62.

The calculation of BD requires integration of the deceleration function over the appropriate range of velocity. The results of this integration can be satisfactorily approximated by using an appropriate average deceleration to solve for the BD. This average deceleration, f in Equation 1, is related to the coefficient of friction and aerodynamic drag by the following formula:

$$f = \mu(0.707V_I) + C_{aero}(0.5)(V_I)^2 \quad (6)$$

where C_{aero} for passenger cars is 10^{-5}. The instantaneous aerodynamic drag is approximately equivalent to a deceleration of only 0.004 g at 20 mph but increases to about 0.064 g at 80 mph. These relationships were used to estimate BD (1) and differ greatly from those in the recently published AASHTO policy (8).

A poor, wet road with a grade change of 15 percent (SN_{40} = 28) was selected for use in illustrating braking performance for both controlled and locked-wheel stops. Figure 3 shows the BD curves for this road for various initial speeds for new tires and for tires that are barely legal, with a 2/32-in. tread depth. It also shows the current AASHTO policy values (8), which can be seen to be very close to those for a locked-wheel stop with barely legal tires. New tires reduce BD by up to 100 ft, whereas controlled stops take up to twice as far as locked-wheel stops. These results make it clear that the current BD values should be increased from 275 to 360 ft at 40 mph and from 625 to 1,200 ft at 70 mph if passenger cars with worn tires are to make controlled stops on wet roads with a 15 percent grade change.

It is believed that the findings of the BD analysis are of the greatest significance among the findings of this research because they affect the STD so significantly. One alternative to lengthening the ASD to the required STD at critical locations is to improve the surface skid capability. For example, increasing the SN_{40} from 28 to 35 (approximately equivalent to a road with a 39 percent grade change) would yield a controlled-stop BD of 414 ft at a speed of 60 mph on a wet road with average partially worn tires (8/32-in. tread), a value consistent with current AASHTO policies. For such tires SN_{40} values from 32 to 37 would achieve desirable AASHTO STD values over the full range of important speeds used in highway design.

ROAD ELEMENTS AND STD

Grades, vertical curves, and horizontal curves all affect BD.

Grades

There are two effects of constant grades (G) on BD. Lengths are based on plane surveying practices that ignore gradients; the actual road extent is greater. On constant grades the additional distance per sta-

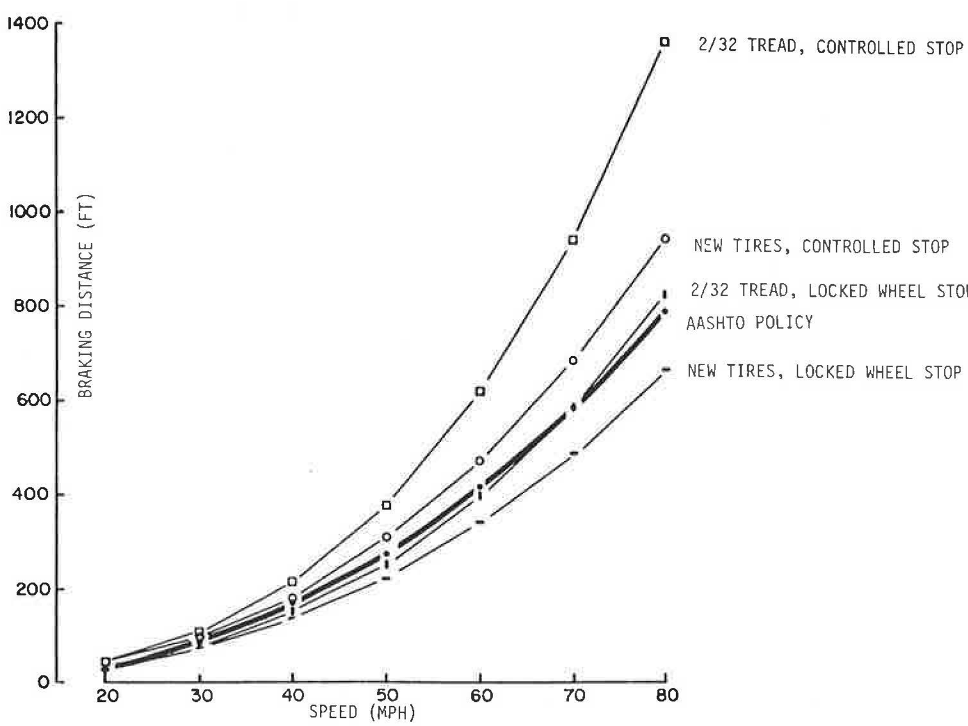

FIGURE 3 Passenger car braking distance on wet, poor road.

tion available with grades of 5 percent is only 0.1 ft/station, whereas for G = 10 percent the value is only 0.5 ft/station. This is clearly of no practical consequence. The second effect comes from the change in resistance to movement as the vehicle climbs or descends a constant grade. This effect can be important when grades exceed 3 percent, and when the increase in BD recommended earlier is taken into account, it should be incorporated into the calculations.

Vertical Curves

Vertical curves affect BD in three ways. In this research it was shown that the true length of a vertical curve (L) is greater than its horizontal projection by a factor of

$$(1 + A/100)^{1/2} \tag{7}$$

where A is the absolute value of the algebraic difference of grades expressed as a percentage. This value is about A/2 percent and therefore gives an increase in effective curve length; hence there is an ASD of about 3 percent for A = 6 percent and 5 percent for A = 10 percent.

A vehicle stopping on a vertical curve faces a continuously varying grade, and this can be taken into account in determining BD. The effect on BD can be substantial and lies between that of the two grades separately. A relation is provided in an NCHRP report (5) that makes a calculation of this value possible.

Finally a vehicle on a vertical curve experiences a centrifugal force that reduces its effective weight on crest curves and increases it on sags. This directly affects the BD because the effective weight affects the braking force, which in turn affects the deceleration and hence the BD. It is shown in an NCHRP report (5) that when a vehicle moves along a parabolic vertical curve, it follows a nearly circular path with a radius approximately equal to 100K, where K is the widely used number of feet along the curve for a 1 percent change in grade. This effect changes BD less than 1 percent for speeds of 50 mph or greater and only 2 percent at speeds of 30 mph. This small effect can be ignored in most applications.

Horizontal Curves

It is well known that the lateral acceleration experienced on a horizontal curve (f_e) decreases the available deceleration for stopping (f). The available total friction (f_t) is related approximately to the others, and by using the force equilibrium relationship for horizontal curves and the BD relationship, one obtains

$$f^2 = f_t^2 - [V^2/(15R) - e]^2 \tag{8}$$

where e is the superelevation of the curve with radius R. At high speeds this effect can be significant on curves with minimum radius designs, as has been recently documented by Neuman (9).

Discussion

The recommended STD distances can be compared with those associated with decision sight distance (DSD) (10). This research tends to bring these values closer together and may lead policymakers to use DSD in preference to STD in certain high-speed applications where alternatives to stopping are clearly available.

ASD ELEMENTS

In this section the geometric relationships developed for the engineer concerned with ASD are considered, with particular emphasis on crest vertical curves and the needed clearances for obstacles to the line of sight on horizontal curves.

Driver Eye Height

A study was made of the distribution of driver eye height for the near-term population of drivers and vehicles from which a desired percentile value could be selected to serve as a possible replacement for the current AASHTO policy of 42 in. (8). Driver eye height clearly varies with several factors, including the vehicle type, seat characteristics, and the size, position, and posture of the driver.

Experimental measurements were beyond the scope of this research and an approach based on recommendations of D. Hammond of the Ford Motor Company was used. This approach uses the Society of Automotive Engineers (SAE) eyellipse data, which provide vertical distances from the vehicle seating reference point (SgRP) to various population percentiles of eye height. In order to determine the driver eye height, SgRP-to-eyellipse distance must be added to the SgRP-to-ground distance, a vehicle-specific characteristic.

Ground-to-SgRP distances were determined for almost all domestic and foreign passenger vehicle models sold in the United States in 1981. Because the two distributions are approximately normal and it is assumed that driver and vehicle distributions are independent, the two distributions were added as shown in Figure 4.

Estimates of 1990 fleet sales by weight, as developed by NHTSA (11) with the assumption that the same weight vehicle would have the same SgRP-to-ground height as the 1981 vehicle did, were then used. The results were close enough to the 1981 values that no change was made. Accordingly, a change in the eye height value from 42 in., which is too high for 25 percent of the vehicles, to a value of 40 in., which will accommodate more than 95 percent of the passenger cars, is recommended.

Object Height

No original research was accomplished on object height. However, a good recent summary of ground clearance data for small cars has been provided by Woods (12). These data indicate that 30 percent of such vehicles would not clear a 6-in. obstacle. A 4-in. obstacle height is required to provide clearance for all these small vehicles. The research report shows the effects of such a value on vertical curve design.

Vertical Curves

SSD affects vertical alignment on tangent roadways on both crest and sag vertical curves. During the day the line of sight from the eye of the driver to the obstacle is broken by the road surface for the crest curve and by an overhead structure for the sag curve. After dark, headlamp illumination affects ASD on both types of curves. In this section the crest

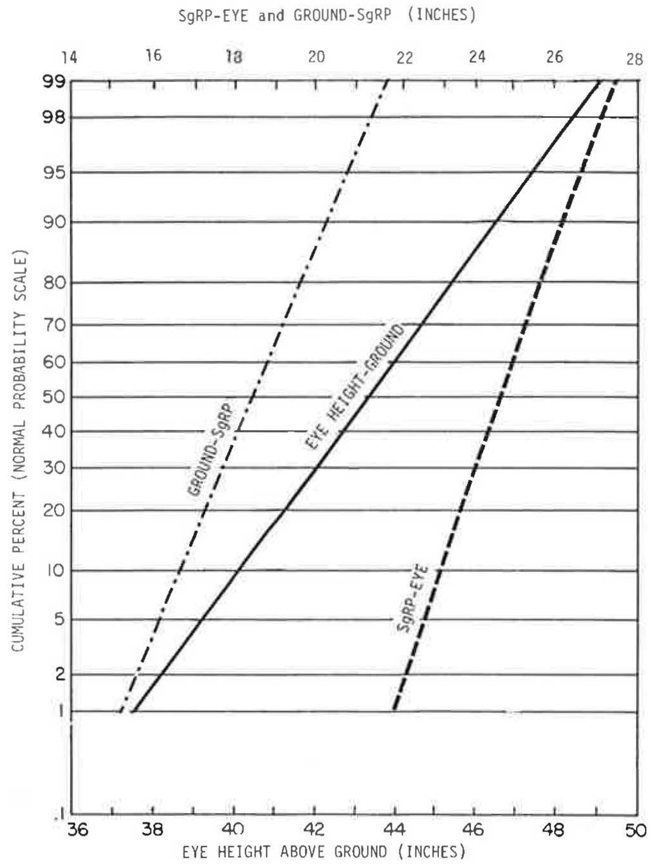

FIGURE 4 Cumulative distribution of eye-SgRP distance, SGRP-ground distance, and eye height above ground.

vertical-curve geometry and the results of an analysis of crest and sag vertical curves are given along with certain important truck and night vision elements.

Figure 5 shows the basic ASD elements for crest vertical curves. The ASD is divided into two components. S_e is the distance from the eye of the observer to the tangent point of the line of sight on the curve, and S_0 is the distance from the tangent point to the top of the object. The difference in grades, $a = 0.01A$, is here defined as $|0.01G_1 - 0.01G_2|$. The symmetry assumption shown in Figure 5 does not affect the final algebraic relationships developed.

In the general case the total sight distance can be expressed and simplified as follows, called the general sight distance formula:

$$ASD = L/2 + 100\{h_eL/(Ax) + h_0L/[A(L - x)]\} \quad (9)$$

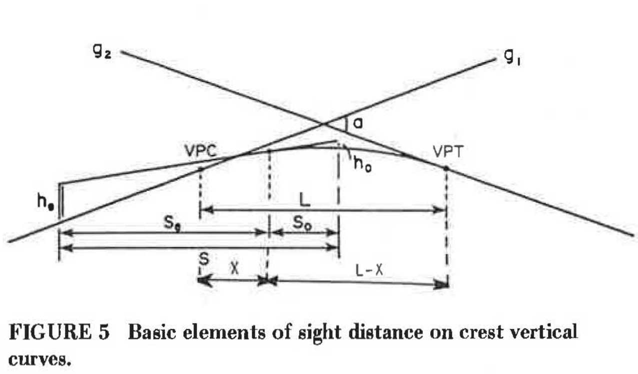

FIGURE 5 Basic elements of sight distance on crest vertical curves.

where

h_e = eye height (ft),
h_0 = object height (ft),
A = absolute value of the algebraic difference in grades (%),
L = curve length (ft), and
x = point of tangency of the line of sight measured from the point of vertical curvature (VPC).

Solutions for all cases can be obtained by using Equation 9 with the results given in Table 2.

With these relationships SD graphs for crest vertical curves can be generated and plotted by computer. Figure 6 shows an example of such an SD graph [see also the paper by Neuman and Glennon (13)]. Such graphs can be used to evaluate the variation in ASD and to compare the STD with the ASD, the time a driver spends on the curve with minimum ASD available, and the locations on the crest vertical curve where the minimum ASD occurs. Computer programs were prepared to generate the data and plot these SD graphs.

Night Visibility

The ASD in the case of a sag vertical curve has been defined by AASHTO as the distance from the eye of the driver to the point on the road where a headlamp beam with an upward divergence of 1 degree from the vehicle axis strikes the road surface (2). The study showed that this model is useful only when the object to be seen has retroreflective properties, because the headlamp illumination above the vehicle's axis is too weak for the driver to see any other object at these distances.

The problem of night visibility on crest vertical curves was also considered. An object beyond a crest vertical curve that would be visible under daytime conditions is shadowed by the road crest at night. The effect of a typical headlamp mounting height on ASD at night was analyzed. Data on the visibility of small, low-contrast objects under headlamp illumination with high beams were used. This effect was concluded to be important only for speeds of 30 mph or less.

Trucks

Experiments were conducted in which the performance of professional truck drivers in stopping their vehicles on wet pavements under various load conditions was studied. For locked-wheel stops on poor, wet roads, trucks require from 1.20 to 1.22 the STD of passenger cars for speeds from 40 to 70 mph. For controlled stops the ratio is from 1.39 to 1.47. With typical values of eye heights for conventional truck and passenger cars of 93 and 40 in., respectively, and a 6-in. object height, calculations show that the required truck STD should be less than 1.35 times that for cars if trucks are to be able to stop within the ASD on crest vertical curves designed for cars. It can be concluded that the greater ASD for trucks compensates fully for the disadvantage in STD in locked-wheel stops. However, trucks require about a 7 percent greater ASD than do passenger cars for controlled stops.

Horizontal Curves

The ASD on horizontal curves is concerned with lines of sight across the inside of such curves as well as

TABLE 2 Formulas for Crest Vertical Curves

Case	Location of Observer/Object	Point of Tangency (x)	Sight Distance Formula[a]
$S < L$	Grade/curve	$0 < x < X_1$	$S = h_e L/(ax) + x/2 + (2h_0 L/a)^{1/2}$
	Curve/curve	$X_1 < x < X_2$	$S = (2h_e L/a)^{1/2} + (2h_0 L/a)^{1/2}$
	Curve/grade	$X_2 < x < L$	$S = (2h_e L/a)^{1/2} + (L - x)/2 + h_0 L/[a(L - x)]$
$S > L$	Grade/curve	$0 < x < X_2$	$S = h_e L/(ax) + x/2 + (2h_0 L/a)^{1/2}$
$S_0 < L$	Grade/grade	$X_2 < x < X_1$	$S = h_e L/(ax) + x/2 + (L - x)/2 + h_0 L/[a(L - x)]$
	Curve/grade	$X_1 < x < L$	$S = (2h_e L/a)^{1/2} + (L - x)/2 + h_0 L/[a(L - x)]$
$S_0 > L$	Grade/grade	$0 < x < L$	$S = h_e L/(ax) + x/2 + (L - x)/2 + h_0 L/[a(L - x)]$

[a] $X_1 = (2h_e L/a)^{1/2}$ and $X_2 = L - (2h_0 L/a)^{1/2}$.

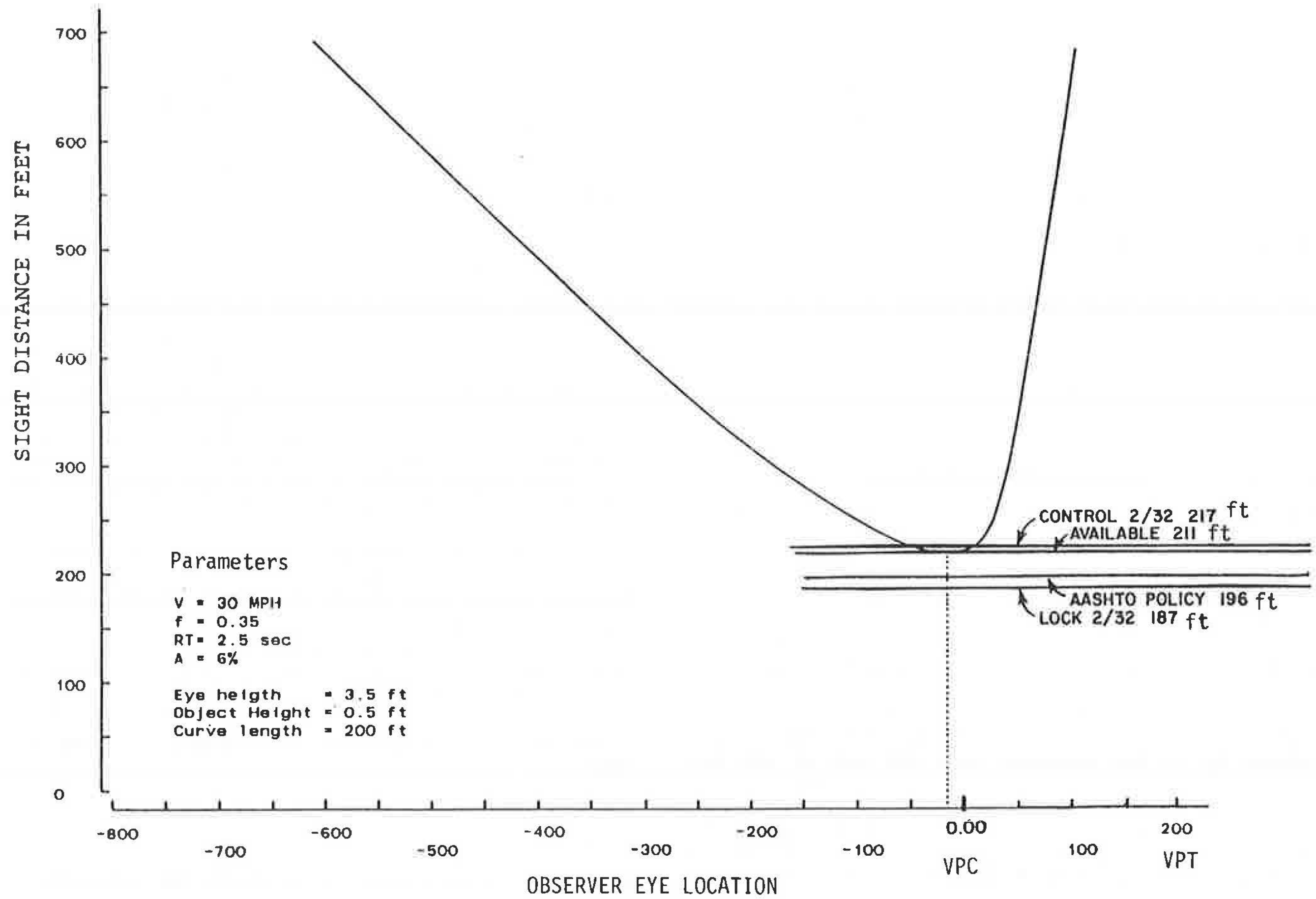

FIGURE 6 SD graph for crest vertical curve.

the location of the eye and object. Of particular importance is the location of the critical obstacle to vision, expressed typically as the clearance (m) along a radial direction from the path of the driver's eye as shown in Figures 7 and 8. Where this clearance is a maximum, which occurs when the STD is less than the length of the curve, M is used in the formulas. Elements that were considered include the changing values of m near the end of the curve as well as the effect of designs using spiral transition curves linking the tangents with the circular portion of the curve.

AASHTO presents clearance requirements for sight obstructions inside horizontal curves only for the case when STD < L and both observer and object are on the curve (2). The other cases all require less clearance for a given STD. This is of particular importance if the longer STD values recommended in this research are used in place of current AASHTO policy values.

Table 3 gives chord approximation relationships for determining the maximum needed clearance M. The chord approximation has less than 0.5 ft error in M for radii of 400 ft or more and is easier to use than the trigonometric relationship commonly encountered.

The case when STD > L has not been treated analytically or summarized in current AASHTO publications. It was found that this value of m can be expressed as a simple function of M for the case when STD < L. The results are shown in Figure 9.

Figure 10 was prepared as a design aid to relate ASD to m when STD > L. It can be used to determine the critical value of any parameter--m, R, I, or ASD--when the other three are given.

When the observer is on the tangent within a distance STD or less from the point of curvature (PC), there is also a required clearance (m) on the tangent section. It varies approximately as a quadratic

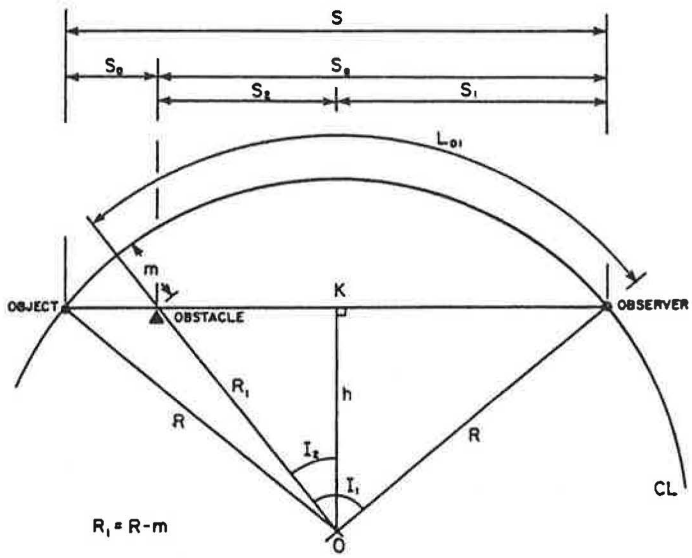

FIGURE 7 Observer and object in the horizontal curve.

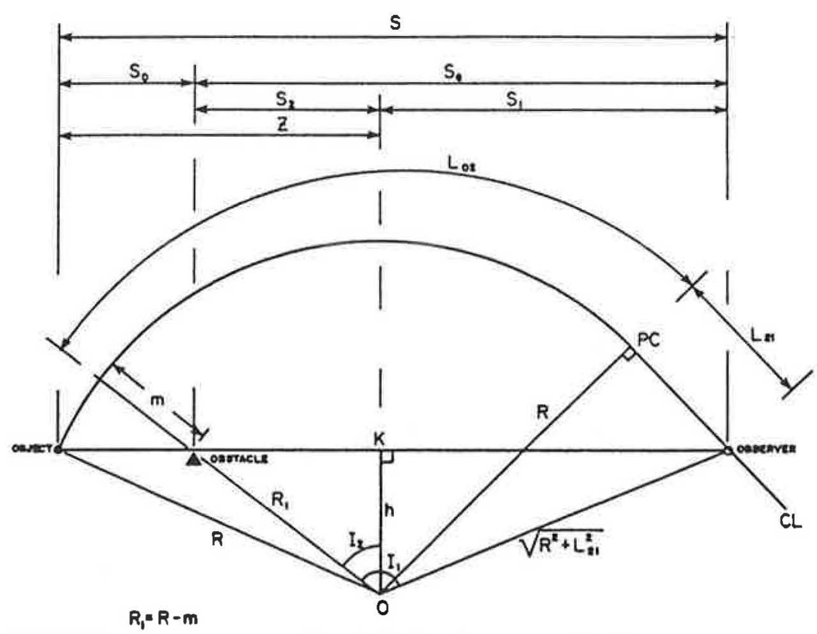

FIGURE 8 Observer on tangent and object in the horizontal curve.

TABLE 3 Horizontal Curve Clearance M (STD < L)

Case	Exact Solution	Chord Approximation
ASD < L	M = R[1 - cos(I*/2)]	M = (STD)^2/(8R)
ASD = L	M = R[1 - cos(I*/2)]	M = L^2/(8R)
ASD > L	M = R sin(I/2)tan[(I* - I)/2]	M = L(2STD - L)/(8R)

Note: I = central angle of horizontal curve; I* = central angle subtended by STD.

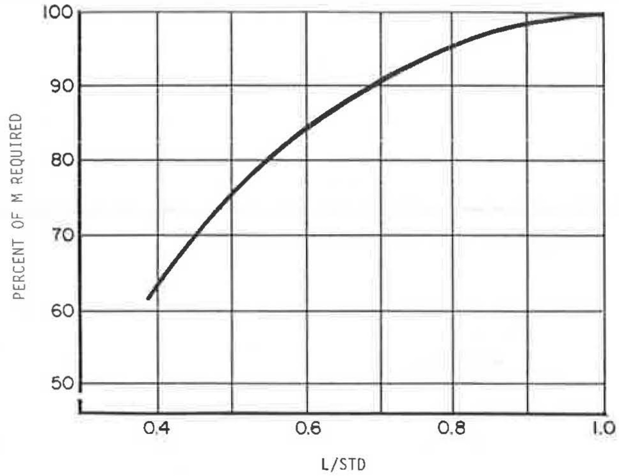

FIGURE 9 Required horizontal curve clearance as percentage of M when STD > L for ASD < L.

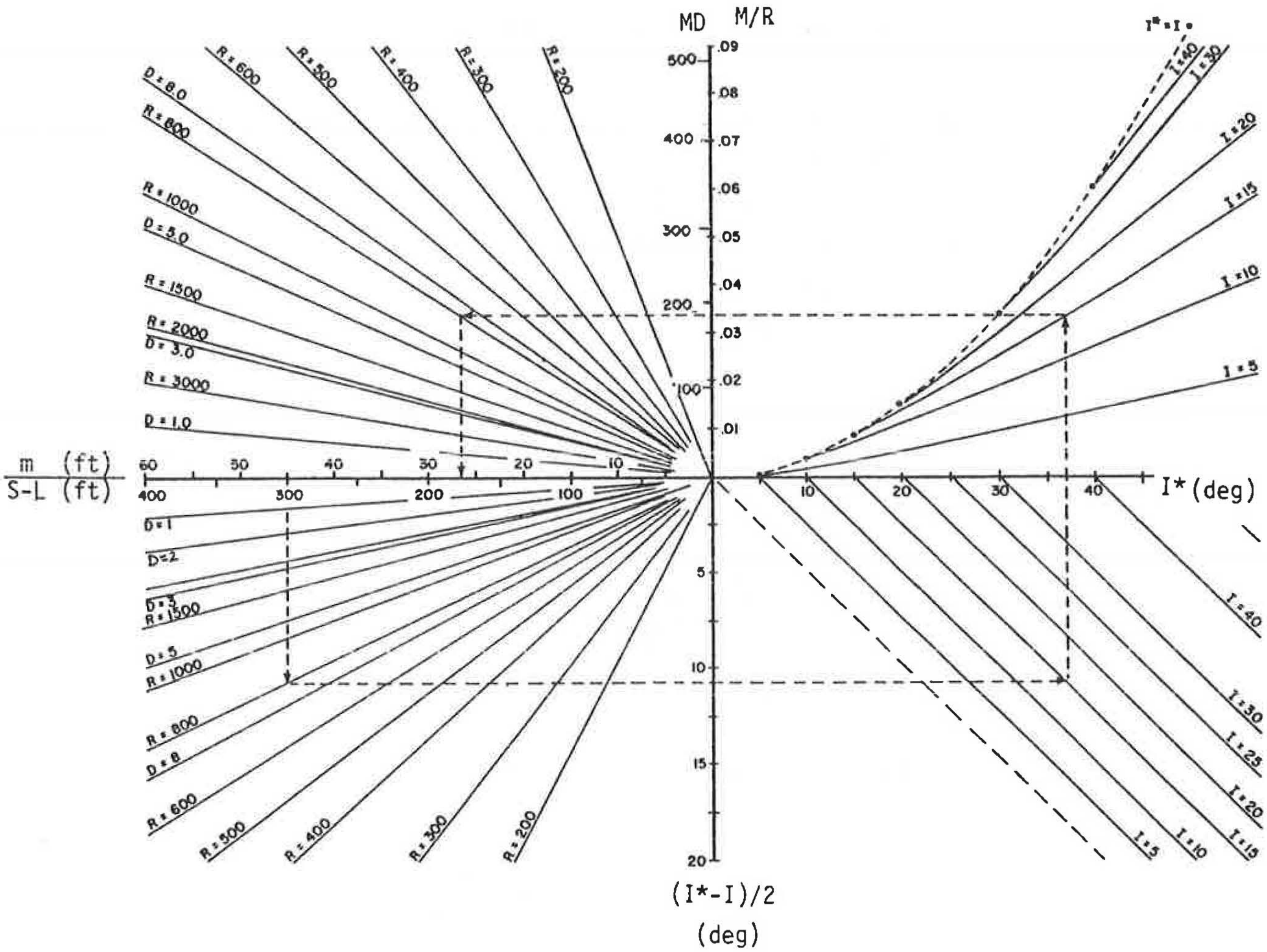

FIGURE 10 Critical lateral clearance on horizontal curves.

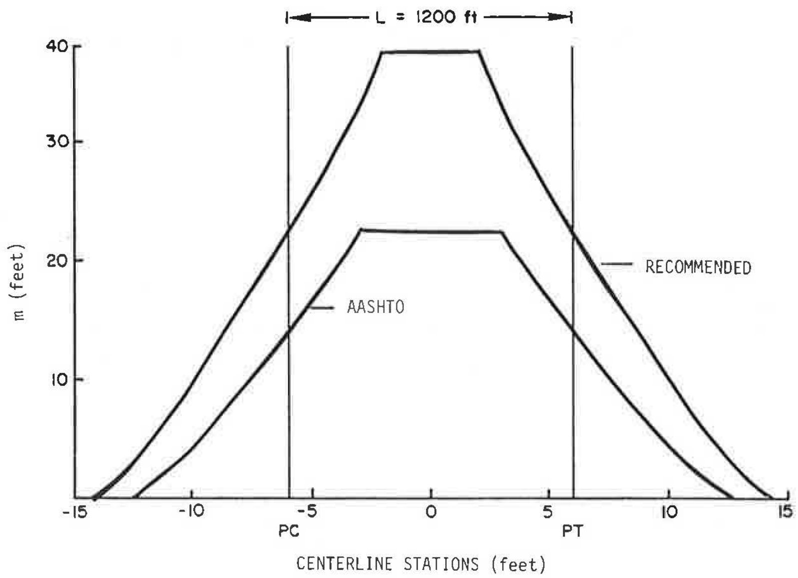

FIGURE 11 Example of needed horizontal curve clearance.

from zero at a point STD in advance of the PC to the full M required on the curve when the observer reaches the PC. Examples of this effect for STD values found in this research and AASHTO recommendations for a 1,200-ft curve with a design speed of 60 mph are shown in Figure 11.

Spiral Transition Curve

A spiral transition curve reduces the needed m while the driver is on the tangent and on the spiral. The magnitude of this effect was studied for typical spirals and it was found that this decrease in the needed m-value would range from about 1 to 4 ft as design speeds increase from 50 to 80 mph.

Position of Eye and Object

Current design practice places the eye and object on the centerline of the critical lane of travel. A sensitivity analysis showed that other reasonable positions of eye and object have no important effect on ASD.

ACKNOWLEDGMENTS

Human factors research was done by P.L. Olson and M. Sivak; vehicle studies were accomplished by P.S. Fancher; the safety and speed studies were made by L.P. Kostyniuk and D. Godfrey; L.W. Schneider developed the eye-height distributions for the 1981 car fleet. The other studies were made by D.E. Cleveland, and several of the derivations were made by G.R. Waissi.

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The opinions and conclusions expressed or implied in this paper are those of the research agency and are not necessarily those of AASHTO, FHWA, NCHRP, or the individual states participating in NCHRP.

Publication of this paper sponsored by Committee on Operational Effects of Geometrics.

Geometric Design of Exclusive Truck Facilities

JOHN M. MASON, Jr., and ROBERT C. BRIGGS

ABSTRACT

Past truck research is studied to determine the applicability of AASHTO geometric design policies to exclusive truck facilities. The policies addressed include those with respect to vehicle characteristics, sight distance, horizontal alignment, vertical alignment, and cross-section elements. Each existing AASHTO design policy is described, the applicability of the policy to exclusive truck facilities is discussed, and alternative design criteria are recommended where past research warrants possible changes.

Rapid traffic growth has prompted the Texas State Department of Highways and Public Transportation (SDHPT) to examine various techniques to handle the corresponding increase in truck traffic demands. The SDHPT sponsored a study to evaluate the needs of a special truck lane along the I-35 corridor between Dallas and San Antonio. The objectives were to identify areas with a high volume of trucks, establish operational and design procedures to deal with truck traffic, and evaluate the effects of the proposed recommendations.

One specific alternative of interest was the feasibility of using existing median areas to accommodate exclusive truck facilities (ETFs). These lanes would be located on intercity corridors where high volumes of truck traffic existed or were projected. The I-35 corridor was selected as the initial segment for evaluation. Findings of this initial study will be used to establish procedures for evaluating other high-volume truck corridors in the state.

The analysis procedure involved two distinct phases. The first documented the physical problems associated with placing ETFs in the existing right-of-way. The second phase consisted of the review of current geometric design policy to determine its applicability to ETFs. Major elements of the study included geometrics, right-of-way availability, operations, safety, pavement requirements, and costs of the potential improvements.

Roadway geometry was a primary consideration in the analysis. Geometric design was addressed initially because it affects right-of-way limits, operational efficiency, safety, and construction costs. Current roadway design policies largely reflect those outlined in AASHTO's Green Book (1). However, these policies are based on the assumption that the majority of the design traffic will be automobiles, with a relatively small percentage of large trucks.

No publication exists that provides specific guidelines for the geometric design of ETFs. A detailed literature review of truck-related information was conducted to determine the feasibility of applying the findings to the design of ETFs. This paper summarizes the review of the pertinent design elements and identifies areas where additional design criteria are necessary. The following elements were examined: vehicle characteristics, sight distance, horizontal alignment, vertical alignment, and cross-section elements. Further research is needed to satisfactorily address the design requirements of ETFs.

VEHICLE CHARACTERISTICS

There are numerous publications dealing with vehicle characteristics and their effect on roadway design. The literature generally provides guidance on geometric requirements for several specific vehicle characteristics.

AASHTO (1) policy addresses two distinct classes of vehicles--passenger cars and trucks. Passenger car characteristics should be excluded in the design of ETFs. The AASHTO truck class is categorized by single-unit trucks, buses, truck tractor-semitrailer combinations, and trucks or truck-tractors with semitrailers in combination with full trailers. Current vehicle dimensions are shown in Table 1. Truck characteristics can be further divided into two categories--size and performance. The size category includes vehicle height, width, and length and driver eye height. The performance category includes weight-to-horsepower ratios, braking ability, acceleration, and deceleration. A summary of truck characteristics and the geometric features that they affect is shown in Table 2 (2).

TABLE 1 AASHTO Design Vehicle Dimensions *(1)*

	Vehicle Dimensions (ft)		
Design Vehicle Type	Height	Width	Length
Single-unit truck (SU)	13.5	8.5	30
Intermediate semitrailer (WB-40)	13.5	8.5	50
Large semitrailer (WB-50)	13.5	8.5	55
Double-bottom semitrailer with full trailer (WB-60)	13.5	8.5	65

Vehicle height is generally 13.5 ft because of clearance restrictions on U.S. highways. Truck operators and manufacturers have expressed little interest in raising limits of vehicle height because of existing loading-dock dimensions, stacking limitations of most commodities, and vehicle instability on sharp curves in high wind situations (3). No change in AASHTO policy for design vehicle height appears necessary for the design of truck facilities.

AASHTO recommends a design vehicle width of 102 in. The Surface Transportation Assistance Act of 1982 requires states to allow the operation of 102-in.-wide trucks on the Interstate system regardless of the maximum-vehicle-width laws in the individual states. The 102-in. width should represent a minimum design vehicle width. Larger widths could be used, depending on the amount of oversize permits issued along the particular corridor. Increased vehicle widths directly affect pavement costs because of lane-width requirements. Therefore increasing design vehicle width may require cost/benefit analyses on an individual-corridor basis.

TABLE 2 Geometric Features and Related Vehicle Characteristics *(2)*

Geometric Feature	Related Vehicle Characteristic
Sight distance	
Stopping sight distance	Braking distance, eye height
Passing sight distance	Vehicle length, acceleration
Horizontal alignment	
Superelevation	Vehicle height (C.G.)
Degree of curve	Vehicle height (C.G.)
Widths of turning roadways	Vehicle length, width
Pavement widening on curves	Vehicle length, width
Vertical alignment	
Maximum grade	Weight-to-horsepower ratio
Critical length of grade	Weight-to-horsepower ratio
Climbing lanes	Weight-to-horsepower ratio
Vertical curves	Eye and headlight heights
Vertical clearance	Vehicle height
Cross-section elements	
Lane widths	Vehicle width
Shoulder widths	Vehicle width
Traffic barriers	Vehicle mass and C.G.
Side slopes	Vehicle height (C.G.)

Note: C.G. = center of gravity.

AASHTO design vehicle length varies according to the vehicle type. The longest design vehicle is a WB-60, which is 65 ft. Since the 1982 Surface Transportation Assistance Act, vehicles up to 65 ft long are permitted access to the Interstate system. Several states have allowed combinations of greater than 65 ft to operate on their roadways for a number of years (4). Walton and Burke (5) have assembled a series of configurations for various truck types of differing size and weight that represent feasible maximum vehicle lengths. The longest vehicle configuration presented is a triple combination that is 105 ft long. This configuration is legal in some states at this time. Because this vehicle type is already in service, it is recommended as the minimum design vehicle configuration for ETFs.

A study of truck driver eye height yielded values of 94 in. for cab-over and 101 in. for cab-behind-engine truck configurations (6). These heights were determined for an individual of average height seated in each type of truck. Six trucks from three manufacturers were used in this study. However, Middleton et al. (7) reported a different relationship for truck driver eye height: 107 in. for a cab-over truck, 93 in. for a cab-behind configuration, and 91 in. for a low-cab-over configuration. This study was based on an average of eye heights provided by five different truck manufacturers. The difference in these findings demonstrates the need to determine the range of truck driver eye height. Once an appropriate range has been established, a sensitivity analysis should be performed to determine the significance of the variations.

Current AASHTO policy (1) uses a weight-to-horsepower ratio of 300 lb/hp to represent the characteristics of heavy vehicles operating on grades. Previous versions of the policy (8,9) used a 400:1 ratio. Figure 1 shows the changes in the average weight-to-horsepower ratio for vehicles operating on U.S. highways between 1949 and 1973. Walton and Gericke (10) state that today's trucks perform better than national representative trucks of the past because of superior engines and transmissions.

AASHTO policy argues that 300-lb/hp trucks have operating characteristics that are acceptable to the highway user, that carrier operators are voluntarily using this value in the determination of maximum truck loading, and that the manufacturers of trucks find this value acceptable for the design of the vehicle. However, a 1984 study (11) found a larger portion of multiple combination trucks operating in

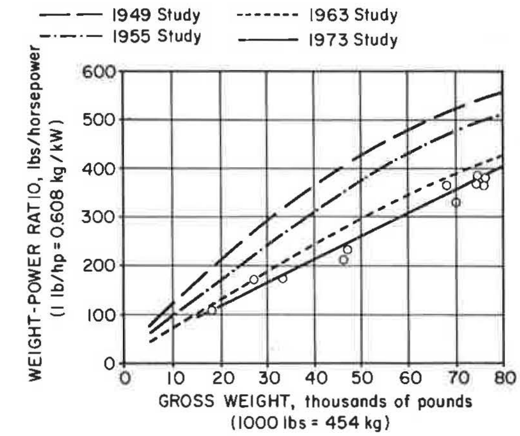

FIGURE 1 Trend in weight-to-horsepower ratios from 1949 to 1973 *(1)*.

the range of 0 to 100 lb/hp (see Figure 2). The 300-lb/hp value, nonetheless, appears appropriate for the design of ETFs.

Heavy-vehicle braking performance depends primarily on tire type and condition, weight of the vehicle, road surface characteristics, number of axles, and number of tires per axle. Several researchers

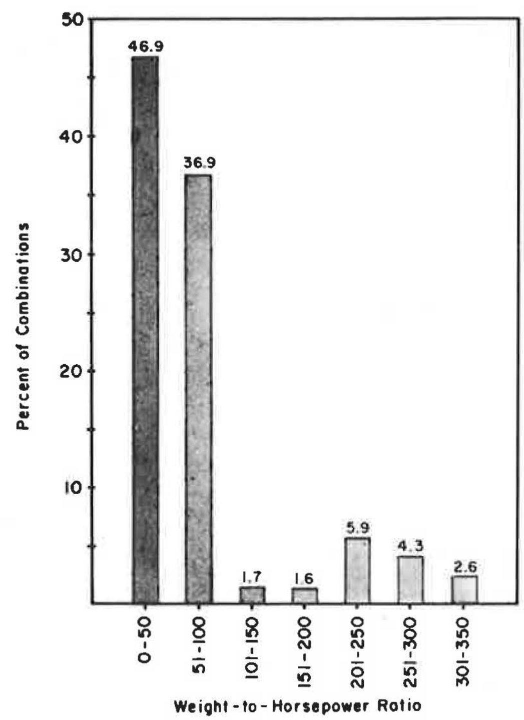

FIGURE 2 Distribution of weight-to-horsepower ratios for combinations operating in the United States *(11)*.

have measured heavy-vehicle braking distance. However, because pavement friction, driver selection, vehicle condition, and test procedures varied among researchers, caution must be exercised in interpreting the results of previous vehicle braking studies.

Peterson and Gull (12) conducted braking tests in Utah to determine the braking performance of single, double, and triple combination trucks. The tests were performed on both wet and dry pavement surfaces. The wet and dry coefficients of friction were 0.64 and 0.92, respectively. They noted that the FHWA Motor Carrier Safety Regulations specify deceleration rates of 21 ft/(sec)^2 for passenger cars and 14 ft/(sec)^2 for truck combinations on dry pavements. Federal regulations also require that a truck stop within a distance of 40 ft from an initial velocity of 20 mph. On the basis of the 40-ft stopping distance requirement and the 14-ft/(sec)^2 deceleration rate, the relationship of required braking distance versus initial speed is plotted in Figure 3. Also shown are the passenger car stopping distances predicted by using the AASHTO braking-distance equation. The FHWA truck stopping-distance curve illustrates the longer braking-distance requirements. For example, a truck traveling 30 mph on dry pavement requires approximately 50 ft more braking distance than does a passenger car traveling at the same speed on the same pavement. It will be necessary to develop braking-distance criteria for ETFs to reflect truck braking characteristics.

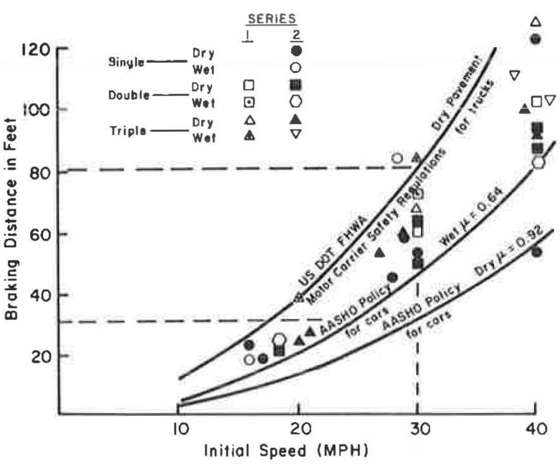

FIGURE 3 Braking distances of various combinations compared with AASHTO and FHWA stopping distance values *(12).*

Truck performance on grades has been routinely investigated. Many studies have been conducted to describe truck deceleration on upgrades and acceleration on downgrades. This information is used to determine maximum permissible grades, critical lengths of grades, and climbing-lane design. Deceleration curves are shown from the 1965 AASHTO Blue Book (Figure 4 (9)], from the state of Texas in 1976 [Figure 5 (10)], from 1979 California studies [Figure 6 (13)], and from the AASHTO Green Book [Figure 7 (1)]. The improved performance indicated in these curves is attributable to decreasing weight-to-horsepower ratios. Increased performance of trucks on grades allows shorter, less frequent auxiliary truck lanes on uphill sections and greater permissible grades throughout the system. In short, a higher performance design vehicle results in lower construction costs because of minimized cut-and-fill operations and a reduced need for climbing lanes.

SIGHT DISTANCE

AASHTO (1) assumes that the increased stopping distances that trucks require are compensated for by

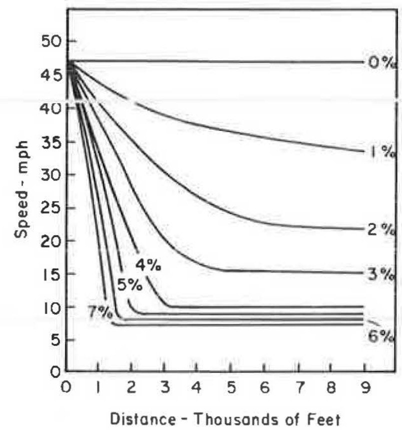

FIGURE 4 Deceleration curves from 1965 AASHTO Blue Book *(9).*

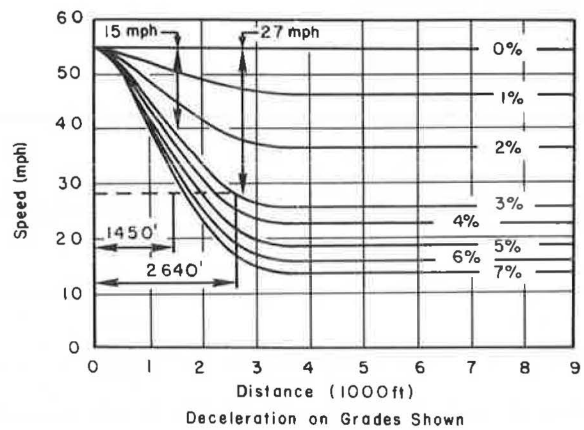

FIGURE 5 Deceleration curves from Texas in 1976 *(10).*

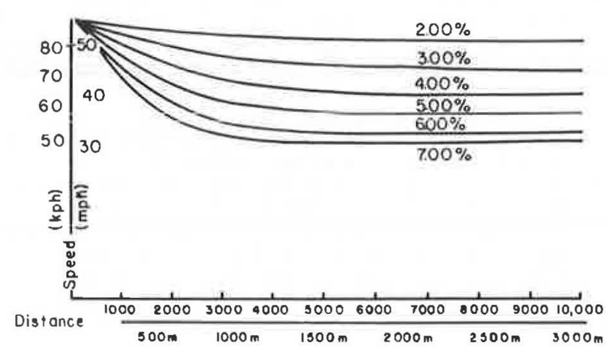

FIGURE 6 Deceleration curves from 1979 California studies *(13).*

the greater sight distance of the truck drivers because of higher eye heights. Studies indicate that this may not always be the case, especially where heavily loaded trucks are concerned (7,14).

A study of truck sight distance requirements (14), for example, concluded that heavily loaded trucks require stopping distances of such magnitude as to eliminate any sight distance advantages over current AASHTO criteria. Sight distance advantages of trucks on crest vertical curves were calculated relative to sight distances provided for passenger cars. Braking distances were then calculated by us-

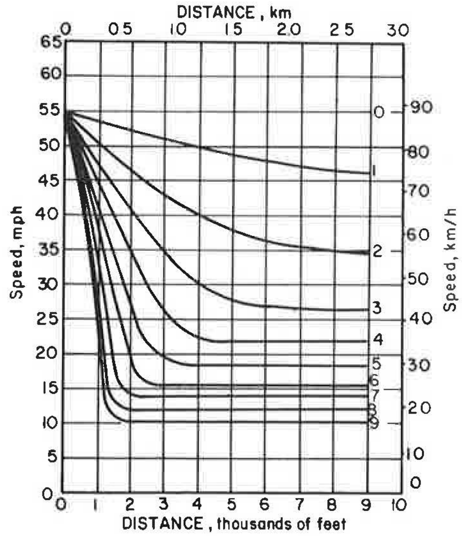

FIGURE 7 Deceleration curves from AASHTO Green Book *(1)*.

ing data from on-the-road vehicle braking tests conducted by the Bureau of Motor Carrier Safety. It was found that the upper range of truck braking distances obtained in the study was large enough to negate the advantages of the commanding view afforded truck drivers.

The foregoing study also addressed the sight distance requirements for truck passing zones. Trucks generally enjoy a 17 to 27 percent increase in sight distance relative to that of passenger cars on crest vertical curves. In current practice, passenger car operating characteristics are used in the determination of passing zones for cars passing cars on two-lane highways. However, passing-zone requirements for cars passing trucks are 1.25 to 2 times the distance required for cars passing cars. Trucks passing trucks unfortunately require even greater distances. It is therefore necessary to revise passing-zone design to reflect the truck-passing-truck situation.

The horizontal sight distance criteria on curves used by AASHTO may also need to be reformulated (2). AASHTO assumes that on vertical curves, the increase in truck driver eye height relative to that in passenger cars compensates for the increased braking requirements of heavy trucks. However, the sight distance requirement on a horizontal curve is not a function of driver eye height alone. It is primarily a function of the distance of an obstruction from the center of the inside travel lane. Thus, the direct application of a safe stopping sight distance based on passenger car driver eye height cannot be used for ETFs.

Specific eye height criteria will have to be established for ETFs. The selected criteria will be reflected in the design of vertical curves, passing-zone markings, and horizontal curves.

HORIZONTAL ALIGNMENT

AASHTO uses the minimum-radius equation for the design of horizontal curves:

$$e + f = V^2/15R \qquad (1)$$

where

V = vehicle design speed (mph),
e = superelevation rate,
f = limiting side friction factor, and
R = radius of curvature (ft).

The side friction factor (f) was established based on comfort of the driver while negotiating a turn. One weakness of this method has been identified by Weinberg and Tharp (15), who state that f fails to take into account the tendency of the vehicle to overturn on a curve. A side friction factor that has not exceeded the driver comfort range may be of sufficient magnitude to cause a heavily loaded vehicle with a high center of gravity to overturn while it is negotiating a curve (3).

The determination of the distribution of the actual centers of gravity of commercial vehicles is necessary to properly evaluate the sensitivity of on-the-road variations. Certain computer programs that model heavy-vehicle responses to various inputs could possibly be used to redefine the f-value in terms of overturning moments of a variety of vehicle configurations.

The maximum values of superelevation used in practice are primarily limited by climatic conditions, terrain characteristics, and rural or urban design considerations rather than by vehicle characteristics. For ETFs, the rate of superelevation may need to be revised to reflect the limiting f-values associated with rollover thresholds (16). Preliminary review indicates that the critical value of f may be near 0.25 for low-speed turning maneuvers. Superelevation on turning roadways at intersections and interchanges may need to be increased relative to current practice so that excessive friction requirements associated with these maneuvers do not result in vehicle turnovers.

VERTICAL ALIGNMENT

In 1969, Glennon and Joyner (17) reevaluated the AASHTO design criteria that related truck operating characteristics on grades to the implementation of truck climbing lanes. They found that the 400-lb/hp ratio used for truck speed-distance curves represented a reasonable lower boundary for trucks operating on the roadway at that time. They recommended that the AASHTO 15-mph speed reduction criterion be reduced to 10 mph. In addition, they recommended that the downhill portion of the auxiliary truck lanes be extended to allow reentry speeds closer to average running speeds. The current AASHTO design policy (1) has adopted the 10-mph speed reduction and a 300-lb/hp ratio for critical length-of-grade determination. These criteria can be reasonably applied to ETF design.

Middleton et al. (7) studied the relationship between available stopping sight distance of heavy trucks and the required braking distance on crest vertical curves. They concluded that on such curves where there were large differences in tangent grades, drivers of heavy trucks would not always have the required sight distance needed to stop in time to avoid hitting a 6-in. obstacle on the road. The same was true for a 15-in. obstacle, which was chosen to represent the taillights of a passenger car. Vertical-curve design policy will need to consider critical combinations of tangent grades to avoid sight distance deficiencies on ETFs.

Gordon (14) found that because visibility on sag curves is a function of headlight heights and beam angles, trucks would experience no sight distance deficiencies on sag curves designed according to AASHTO policy.

CROSS-SECTION ELEMENTS

Lane Widths

Weinberg and Tharp (15) state that lane widths on tangent and comparatively flat curves have been determined by the summation of safe lateral clearance between opposing vehicles, the clearance between the vehicle and pavement edge, and the width of the vehicle. On turning roadways, offtracking and front and rear overhang characteristics are added to the foregoing variables to obtain needed lane widths. In short, adequate pavement widths are a function of body and edge clearances for meeting and passing vehicles.

The Red Book (8), Blue Book (9), and Green Book (1) state that lanes 12 ft wide are preferred for high-type multilane facilities and two-lane highways. For freeways, the assumption is made that traffic conditions that dictate the use of a multilane configuration also dictate the use of 12-ft lane widths. For two-lane highways, a 12-ft lane width is considered essential in maintaining adequate clearance between commercial vehicles. For ETFs a 13-ft lane width may be desirable, especially if large volumes of oversize vehicles are to use the facilities. Walton and Gericke (10) state that the need for adequate clearance between vehicles necessitates providing 12-ft lanes for the operating of 102-in.-wide trucks.

In 1945, Taragin (18) studied the relationship between lane widths and vehicle operation. He collected data on lateral placement of passenger cars and trucks on various types of two-lane highways. The roadway widths of these highways varied from 18 to 24 ft and shoulder width varied from 2 to 10 ft. Lateral placement data were collected for cars and trucks traveling freely, encountering opposing vehicles, and passing vehicles traveling in the same direction. Saag and Leisch (2) utilized these data to determine the desired left and right lateral clearance for cars and trucks on rural highways. They concluded that truck drivers desire 2.5 ft of clearance between the left side of the truck and the left edge of the traffic lane when they are meeting or passing other trucks. In addition, for the same maneuver, the driver desires a clearance of 2 ft from the right side of the truck to the right edge of the traffic lane. They further concluded that with a truck width of 8 ft or more, trucks on 12-ft-wide pavements did not have enough lateral pavement width to achieve these clearances. Saag and Leisch presented an equation to determine desirable pavement lane widths as a function of vehicle width:

$$L = 4.5 + W_v \qquad (2)$$

where

W_v = vehicle width (ft),
L = lane width (ft), and
4.5 = sum of desired right and left clearances (ft).

Thus for an 8.5-ft-wide truck, the desired lane width is 8.5 + 4.5, or 13 ft.

Taragin assumed that drivers were satisfied with lane widths when the lateral position of the vehicle within the traveled way remained constant for free-moving, opposing, and passing maneuvers. In addition, he assumed that drivers positioned their vehicles near the center of the traveled lane when they were satisfied with the lane widths provided. On the basis of these assumptions, certain studies indicate that truck drivers are not satisfied with lanes 12 ft wide.

For example, a study was conducted by Canner and Hale (19) to determine vehicle encroachment on bituminous shoulders and lateral placement of vehicles within the right-hand lane of four-lane divided highways. The vehicles studied were trucks with dual tires on the back axle, tractor-trailer combinations, and buses. The highway sections were edge striped so that the effective lane width was 12 ft. However, the pavement extended 3 ft outside the right-edge stripe in some sections. At these sections, heavy vehicles moved toward or crossed over the right-edge stripe more often than on sections where the edge stripe was located at the edge of the pavement.

Lee (20) conducted studies of lateral placement of trucks on four-lane divided highways with 12-ft traffic lanes. His data indicate that the largest percentage of observations of wheel placement were within 2 ft or less from the right pavement edge. As the size of the truck increased, the percentage of observations within the 2-ft distance increased. Also, the frequency of placement within the 2-ft distance increased on curved sections of roadway.

The foregoing two studies support the statement by Saag and Leisch that truck drivers are not satisfied with 12-ft lane widths. Thus, as a consequence, lane widths of 13 ft or greater are recommended for exclusive truck operations.

Width of Shoulders

AASHTO (1) defines highway shoulders as a portion of the roadway for the accommodation of stopped vehicles, emergency use, and lateral support of surfaces and base courses of the roadway. Shoulders are recommended to be of sufficient width to provide 2 ft of clearance between the edge of the traffic lane and the stopped vehicle.

Right shoulders are commonly 10 ft wide on freeways and other high-type facilities; in areas with a high volume of truck traffic, 12-ft right shoulders are recommended. For sections with many through lanes, 10-ft-wide left shoulders are recommended. Shoulders should be continuous and full width across all structures.

AASHTO policy (1) distinguishes between graded and usable shoulders. The graded shoulder width is the distance from the edge of the travelled way to the intersection of the shoulder slope and the front slope of the roadside. The usable shoulder width is that which can be used when a driver makes an emergency or parking stop. A distance of 2 ft from the outer edge of the usable shoulder to roadside barriers, walls, or other vertical elements is recommended. Adequate shoulder widths reduce the potential for collisions with fixed obstacles, overturning of vehicles, running off the roadway, and pedestrian accidents.

Authorization of 102-in.-wide trucks on roadways should not affect AASHTO's current policy on shoulder widths because a 102-in. vehicle width is assumed in its design vehicle. For special-use truck facilities with high percentages of oversize trucks, it may be necessary to reevaluate shoulder width criteria.

Seguin et al. (21) mention shoulder characteristics as a source of potential truck problems on urban freeways. Right shoulder widths averaged 8 to 10 ft; more than 85 percent of right shoulders were 6 ft or wider. Left shoulders averaged 3 to 5 ft in width, and over 50 percent were narrower than 6 ft. In most cases, left shoulders were not adequate to handle trucks making emergency stops. The inadequate widths did not allow trucks to clear the through lanes without running into the median areas. Problems with narrow shoulder widths were often compounded by narrow median widths, which eliminated the possibility of shoulder widening. As more 102-in.-wide trucks use the urban Interstates, these problems will probably worsen. To avoid these types of problems, shoulders of adequate width should be provided on truck facilities. No change in AASHTO policy is considered necessary at this time; nevertheless, attention should be given to oversize-vehicle operation, which may warrant increases in shoulder width.

Guardrails

The Green Book (1) states that guardrails should be used where vehicles leaving the roadway would be subject to hazard, but only if the roadside hazard constitutes a greater threat to safety than striking the guardrail itself. Guardrails are designed to redirect the impacting vehicle, reduce its velocity, and guide it along the rail while it decelerates. Current design standards for guardrails assume a design vehicle of 4,500 lb traveling 60 mph and striking the rail at a 25-degree angle (22). No provisions for heavy vehicles are made in the design of most guardrails. As a consequence, most of the roadside hardware in existence today is proving to be inadequate for heavy vehicles such as trucks and buses (23). Facilities designed exclusively for heavy vehicles will require the redesign of roadside hardware.

Several types of guardrails and bridge rails are in use today that will successfully redirect heavy vehicles with minimal property damage. The most common is the concrete median barrier, or safety shape. Full-scale impact testing with heavy vehicles resulted in the successful restraining and redirection of a vehicle at speeds of up to 45 mph at a 15-degree impact angle (24). Concrete bridge rails have also been developed for redirection of errant trucks on elevated structures (25). However, because these rails are somewhat expensive ($41 per foot in 1980), research is needed to develop less costly barriers for heavy vehicles.

Drainage Channels and Side Slopes

Drainage channels, while performing the vital task of directing water away from the highway, should not pose a serious safety hazard to errant vehicles. Extensive studies have been performed to determine optimum ditch designs for highways using passenger cars as test vehicles (26). Because of obvious cost problems, few, if any, studies have been performed on the effects of ditches on the recovery of errant heavy vehicles.

Roadway side slopes are a similar matter. In most cases, vehicle testing on side slopes has been performed with passenger cars as test vehicles. Published data are lacking concerning the controllability of heavy vehicles on roadside slopes. Current criteria provide a starting point in the determination of safe roadside cross sections for heavy vehicles.

CONCLUSIONS AND RECOMMENDATIONS

On the basis of a literature review of truck studies, the following additions to current highway design policy should be considered in the development of criteria for the design of ETFs:

1. Vehicle characteristics
   a. A 105-ft double or triple combination design vehicle should be incorporated into design policy.
   b. Ranges of truck driver eye heights for different truck classes are necessary.
   c. Standardized brake testing of vehicles is needed to produce accurate braking distance requirements for different truck classes.
2. Sight distance
   a. A design driver eye height representing a worst-case scenario should be considered in predicting sight distance requirements for cab-under-truck configurations.
   b. Sight distance requirements on horizontal curves should be calculated and increased stopping distance requirements of heavy vehicles should be accounted for.
3. Horizontal alignment
   a. The side friction factor (f) may warrant modification in consideration of truck overturning moments.
   b. Superelevation rates on turning roadways may need to be increased at low speeds to compensate for vehicle rollover.
4. Vertical alignment
   a. Provisions for auxiliary truck climbing lanes should reflect the 10-mph speed reduction criterion recommended in the revised AASHTO policy.
   b. Crest vertical curve length criteria should be examined for the stopping distance requirements of heavily loaded trucks.
   c. Passing-zone design on ETFs must consider truck performance limitations.
5. Cross-section elements
   a. A design vehicle representing a heavily loaded vehicle with a high center of gravity is needed for designing barriers for ETFs.
   b. Little information is available to predict behavior of errant heavy vehicles on varying roadside slopes. Research into this area is needed in order to develop criteria for a safe roadside environment on truck facilities.

These recommendations provide a starting point in developing geometric criteria for ETFs. They do not represent an opposing viewpoint to current AASHTO policy; rather, they identify areas of concern in the design and construction of unique truck roadways.

ACKNOWLEDGMENT

This paper has been developed as part of an ongoing research project entitled Study of Truck Lane Needs sponsored by the Texas State Department of Highways and Public Transportation. The findings are the result of an initial literature review of past truck studies.

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The views, interpretations, analyses, and conclusions expressed or implied in this report are those of the authors. They are not necessarily those of the Texas State Department of Highways and Public Transportation.

Publication of this paper sponsored by Committee on Geometric Design.

Operational and Safety Effectiveness of Passing Lanes on Two-Lane Highways

DOUGLAS W. HARWOOD, ANDREW D. St. JOHN, and DAVEY L. WARREN

ABSTRACT

Passing lanes and short four-lane sections are installed to provide increased opportunities for passing slow-moving vehicles on two-lane highways. An operational and safety evaluation of these treatments was performed by using traffic operational field data collected at 15 sites and traffic accident data for 76 sites. It was found that passing lanes decrease the percentage of vehicles platooned on two-lane highways and that the magnitude of this benefit varies with passing-lane length, traffic volume, and the level of platooning upstream of the passing lanes. Passing lanes increase the rate of passing maneuvers on two-lane highways but have only a small effect on mean travel speeds. Passing lanes and short four-lane sections do not increase accident rates above the levels found on comparable untreated two-lane highways; in fact they probably improve safety.

An operational and safety evaluation (1) is presented of two closely related treatments used to improve traffic service on two-lane highways: passing lanes and short four-lane sections.

A passing lane is defined as an added third lane in one direction of a normally two-lane highway to provide opportunities for passing slow-moving vehicles where passing opportunities would otherwise be limited by sight distance and opposing traffic. A passing lane may be used either alone or as part of a series of passing lanes in alternating directions. Where sight distance is adequate, some agencies permit passing by vehicles traveling in the opposing direction to that of a passing lane, whereas other agencies prohibit all passing maneuvers by vehicles in the opposing direction.

Passing lanes in level or rolling terrain are a primary focus of this paper because they have not been evaluated extensively in the United States. However, added lanes of this type are also used extensively on steep grades in hilly or mountainous terrain, where they are generally known as truck climbing lanes. Climbing lanes located on grades long and steep enough to reduce trucks to crawl speeds have been evaluated more thoroughly than passing lanes in previous research and are therefore not addressed in this paper.

A short four-lane section is part of a four-lane highway, generally less than 3 mi long and bounded by two-lane sections at both ends. A short four-lane section on a normally two-lane highway could represent the ultimate design for a particular site or could represent the first step in staged construction of a four-lane highway. Whatever the purpose for which a short four-lane section was constructed, it provides additional passing opportunities and operates essentially as two passing lanes in opposite directions at the same location. A short four-lane section requires greater pavement and right-of-way width than a passing lane, but has the potential advantage that there is no need to permit vehicles traveling in either direction to cross the marked centerline in order to pass. Short four-lane sections are usually either undivided or divided with a narrow, flush median, although four-lane divided sections with a raised or unpaved median could operate in a similar manner.

Figure 1 shows a typical passing lane with passing prohibited in the opposing direction, a passing lane with passing permitted in the opposing direction, and a short four-lane section.

STUDY SITES

Passing lanes and short four-lane sections were evaluated by using data collected at selected sites in 12 states that participated in the study: Arkan-

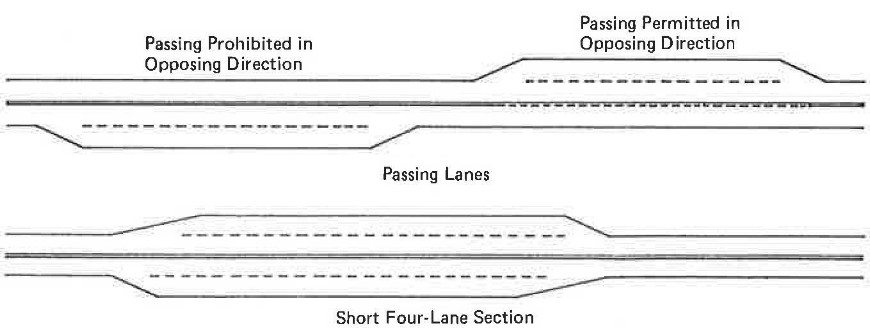

FIGURE 1 Typical passing lane and short four-lane sections.

sas, California, Kentucky, Michigan, Mississippi, Nevada, New York, Oklahoma, Oregon, Pennsylvania, Utah, and Washington. A traffic operational evaluation was based on field data collected at 12 passing-lane and 3 short four-lane sites. A safety evaluation was based on 1 to 5 years of accident data for each of 66 passing-lane and 10 short four-lane sites.

OPERATIONAL EVALUATION

An operational evaluation was performed for 12 passing lanes and 3 short four-lane sites by using traffic performance data collected in the field. The objectives of this evaluation were to determine the effectiveness of these treatments in improving traffic operations on two-lane highways and to determine the influence of traffic volume, geometrics, and treatment length on the operational effectiveness of the treatments.

Data Collection

The field data collection plan for passing lanes used automatic traffic data recorders (TDRs) at six locations and three manual observers. The TDRs were used to record traffic volumes, vehicle mix, speeds, accelerations, headways, and platooning characteristics. The manual observers counted passing maneuvers in both directions in the treated section, counted traffic conflicts or erratic maneuvers in the lane-drop transition area, and performed part of the vehicle classification by entering a code for each recreational vehicle into one of the TDRs. Figure 2 shows a typical data collection setup for a passing lane, including the location of TDR traps and the observers.

The data collection plan was structured to determine the effectiveness of passing lanes by a comparison of traffic operational conditions at three key locations: Location 1 (upstream of the passing lane); Location 3 (in the middle portion of the passing lane); and Location 5 (downstream). In addition, comparisons between Locations 5 and 6 (approximately 1 mile downstream from the passing lane) were intended to determine the rate at which operational benefits of the passing lane are lost downstream. The operational data collected at short four-lane sections were essentially equivalent to those collected at passing lanes, except that they were collected in four lanes rather than in three in the middle of the treated section.

Measures of Effectiveness

Three primary measures of effectiveness were used in this study to assess the operational benefits of passing lanes and short four-lane sections on two-lane highways. These measures were

- Traffic speed,
- Percentage of vehicles platooned, and
- Passing rate.

The mean speed and various percentiles of the speed distribution were used as measures of effectiveness. Speed descriptors were obtained separately for passenger cars, trucks and buses, recreational vehicles (RVs), unimpeded vehicles (free vehicles and platoon leaders) and the traffic stream as a whole.

A key measure of effectiveness in this study is the percentage of traffic traveling in platoons. Research by Messer (2) has found vehicle platooning to be more sensitive to traffic flow rate than mean speed, and the percentage of time spent following in platoons has been proposed as the primary criterion for defining level of service on two-lane highways in the current revision of the Highway Capacity Manual (HCM) (3).

Each vehicle recorded at a TDR trap was classified as a free vehicle, a platoon leader, or a platoon member. Each vehicle with a time headway of 4 sec or less was classified as a platoon member. The choice of the 4-sec headway criterion to define platooning was made after careful consideration of the criteria used by other researchers. The revised HCM procedures recommend a platoon definition based on a 5-sec headway (2). Morrall (4), a Canadian contributor to the revised HCM procedures, used a platoon definition based on a 6-sec headway. Hoban (5), who has conducted extensive operational research on two-lane highways and passing lanes in Australia, has recently recommended a 4-sec headway criterion. In this study, it was considered critical to avoid classifying a vehicle as platooned unless this was clearly the case. For this reason, the shortest of the criteria frequently cited in the literature, 4 sec, was selected.

The final measure of effectiveness used for the evaluation of passing lanes and short four-lane sections was the passing rate, defined as the number of completed passes per hour per mile in one direction of travel. The passing rate is an appropriate measure of effectiveness because passing lanes are intended to increase the passing rate above that which would occur on a normal two-lane highway.

Operational Analysis Results

A combined operational analysis of passing-lane and short four-lane sections was conducted. Each direction of travel in the short four-lane sections was treated as a separate passing lane, so the combined data for the operational analysis represent, in effect, 18 passing lanes.

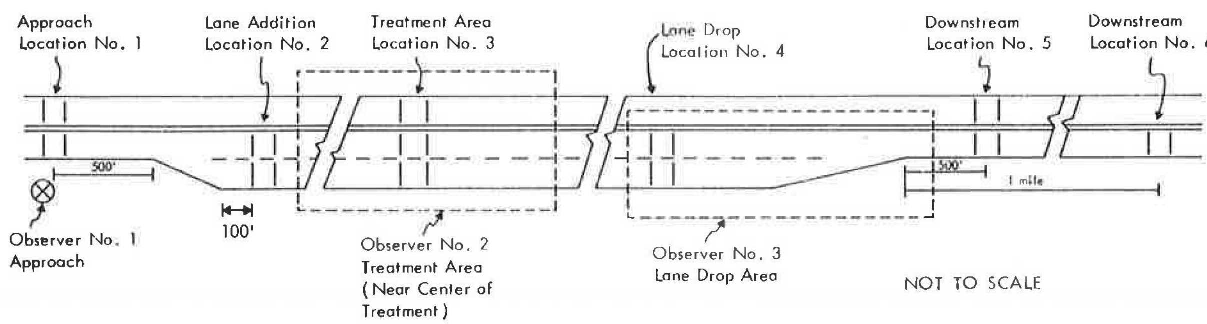

FIGURE 2 Locations of TDR traps and observers for data collection at passing lanes.

Up to 6 hr of operational data were collected at each study site. The traffic flow rates observed at the passing lane and short four-lane sites ranged from 26 to 710 vehicles per hour (vph) in the treated direction. However, the results reported in the following are not necessarily valid for flow rates above 400 vph because very little data at flow rates above that level were obtained. All the conclusions presented are statistically significant at the 95 percent confidence level unless otherwise stated.

Percentage of Vehicles Platooned

Passing lanes were found to reduce the percentage of vehicles that are members of platoons. Table 1 reveals the effect of passing lanes on vehicle platooning. The percentage of vehicles platooned decreased, on the average, from 35.1 percent immediately upstream of a passing lane to 20.7 percent within the passing lane. Immediately downstream of the passing lane, the percentage of vehicles platooned had increased to 29.2 percent, on the average, which is still 5.9 percent lower than the upstream level. This decrease in the percentage of vehicles platooned represents a major improvement in traffic service within a passing lane and a small improvement in traffic service downstream of a passing lane.

Table 1 also shows that the operational benefits from the introduction of a passing lane can vary greatly from site to site. These variations are even greater than those shown in the table when each hour of data from each site is examined separately. The prediction of these variations as a function of geometric and traffic operational variables will be addressed later.

An issue of interest to the evaluation of passing lanes is how far downstream the operational benefits of the added lane persist. It is expected, for example, that any reduction in platooning produced by a passing lane would gradually disappear downstream as faster vehicles overtake slower vehicles and are unable to find passing opportunities. Data were collected in the field approximately 1 mi downstream from each passing lane to determine the persistence of the reduction in platooning provided by a passing lane. Table 1 shows that on the average the percentage of vehicles platooned 1 mi downstream of a passing lane is still 3.5 percent lower than that upstream of a passing lane (31.6 versus 35.1 percent). However, the results obtained from the analysis of these data were inconclusive; the persistence of operational benefits from a passing lane appears to be highly dependent on the geometrics and traffic flow conditions in the downstream area.

The previous discussion has emphasized that this effectiveness of passing lanes varies over a range of values. Several predictive models were developed by using multiple regression analysis to investigate these variations in effectiveness as a function of geometric and traffic variables. A model was developed to predict the change in platooning from the upstream percentage of vehicles platooned and the passing-lane length. This model is

$$\Delta PL = 3.81 + 0.10UPL + 3.99LEN \quad (1)$$

where

- ΔPL = difference in percentage of vehicles platooned upstream and downstream of passing lane,
- UPL = percentage of vehicles platooned upstream of passing lane, and
- LEN = length of passing lane (mi).

This model explains 33 percent of the variation in the dependent variable (i.e., $R^2 = 0.33$). A positive value of ΔPL represents a reduction in platooning.

The percentage of vehicles platooned upstream of a passing lane (UPL) has the strongest correlation with ΔPL of any of the independent variables considered. UPL represents the combined influence of traffic volume, vehicle mix, and upstream geometrics on the traffic entering the passing lane. The use of UPL as a predictor of passing-lane effectiveness is quite appropriate because by using the revised HCM procedures for two-lane highways, UPL can be interpreted directly as the upstream level of service. The positive sign on the regression coefficient of UPL in Equation 1 indicates that the effectiveness

TABLE 1 Effect of Passing Lane on Percentage of Vehicles Platooned

		Percentage of Vehicles Platooned[b]				
Site	Avg Flow Rate (vph)	Immediately Upstream	Within Passing Lane[a]	Immediately Downstream	Downstream 1 mi	Upstream-Downstream Reduction (ΔPL)
1	140	27.4	14.6	18.7	23.0	8.7
2	560	61.9	44.6	57.1	51.5	4.8
3	120	28.0	11.0	21.7	21.3	6.3
4	120	43.4	33.3	40.7	41.8	2.7
5	80	11.7	11.0	8.0	10.7	3.7
6	150	26.7	13.4	25.5	25.0	1.2
7	300	41.2	34.4	36.7	40.9	4.5
8(NB)[c]	410	51.4	31.1	45.8	45.3	5.6
8(SB)[c]	415	46.1	28.7	42.6	-	3.5
9	130	34.2	18.5	31.4	25.0	2.8
10	150	24.1	15.4	22.0	21.6	2.1
11	35	9.2	2.8	8.0	10.7	1.2
12	300	49.1	22.2	37.3	41.6	11.8
13	305	39.0	21.6	44.1	47.2	-5.1
14(NB)[c]	280	41.7	24.1	-	-	-
14(SB)[c]	330	43.6	24.2	35.4	36.9	8.2
15(NB)[c]	340	50.9	22.8	38.4	-	12.5
15(SB)[c]	250	36.4	19.6	23.0	30.9	13.4
Avg[d]		35.1	20.7	29.2	31.6	5.9

[a]Combined data for right and left lanes in treated direction near center of passing-lane section.
[b]Platooned vehicles include following vehicles that are members of platoons but not platoon leaders.
[c]Short four-lane section; remainder of sections are passing lanes.
[d]Average of hour-by-hour data rather than site-by-site data tabulated above.

of a passing lane increases as the traffic entering the passing lane becomes more congested.

The model presented in Equation 1 also demonstrates that the effectiveness of a passing lane in reducing platooning also increases with passing-lane length. The influence of passing-lane length has been represented in Equation 1 as a linear term; however, it is expected conceptually that passing-lane length will have a nonlinear relationship to the effectiveness of a passing lane in reducing platooning, with shorter lanes being more effective per unit length than longer ones. The data currently available are not sufficient to model this nonlinear aspect of passing-lane length, but it merits further investigation.

Figure 3 shows the predictive model represented by Equation 1 and the variation of the reduction in the percentage of vehicles platooned as a function of the upstream percentage of vehicles platooned and the passing-lane length. For example, it can be seen that a 1-mi passing lane with 40 percent of the entering traffic platooned would be expected to reduce platooning by 11.8 percent.

Several additional models were used in an effort to find a model that explained more of the variance in ΔPL than Equation 1. It was found that when flow rate was added to the model presented in Equation 1, the resulting model explained 55 percent of the variance in ΔPL (i.e., $R^2 = 0.55$). This model is

$$\Delta PL = 7.64 - 0.04FLOW + 0.45UPL + 4.82LEN \quad \text{for } FLOW \leq 400 \text{ vph} \qquad (2)$$

where FLOW is the flow rate in the treated direction in vehicles per hour and the remaining variables are as previously defined.

A conceptual drawback of Equation 2 is that the negative sign of the regression coefficient for flow rate implies an inverse relationship between flow rate and ΔPL, which seems counterintuitive; however, it should be noted that such an inverse relationship applies only if UPL and LEN are held constant. The unexpected negative sign for the coefficient of the flow rate term results because flow rate and UPL are strongly correlated with one another ($r = 0.89$, $p < 0.0001$). When two variables are so strongly correlated, it is best to use only one of them in a regression model. In this case, UPL is the better predictor of ΔPL and therefore Equa-

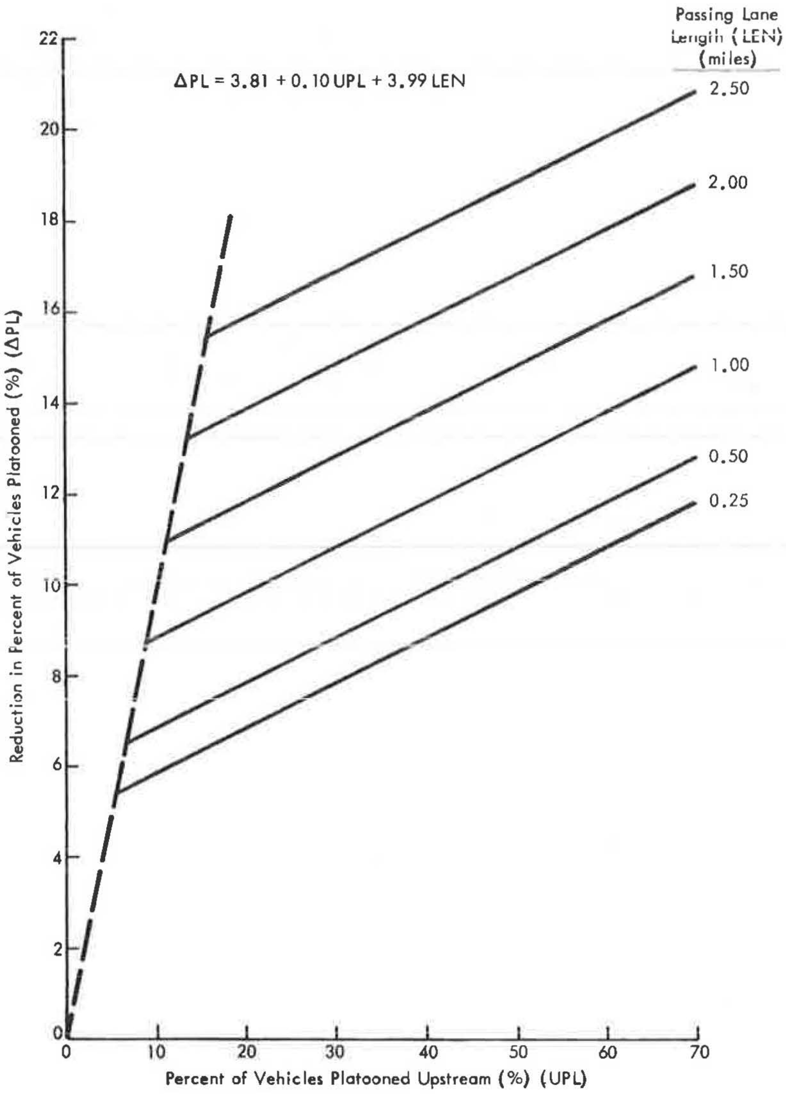

FIGURE 3 Relationship to predict reduction in percentage of vehicles platooned as a function of upstream percentage of vehicles platooned and passing-lane length.

tion 1 is recommended as the best predictive model for ΔPL.

Traffic Speed

An analysis of traffic speed was based on comparisons among the mean speed immediately upstream of the passing lane, within the passing lane, and immediately downstream of the passing lane. Mean speeds were found to be affected, on the average, only slightly by the presence of the passing lane. Mean speeds were approximately 2.2 mph higher within a passing lane than upstream of the lane and 0.9 mph higher downstream of a passing lane than upstream of it. These results indicate a small operational benefit in increased speeds because of the passing lane, although as suggested in the revised HCM, it appears that vehicle platooning is a more sensitive measure of traffic service than is mean speed.

The effect of a passing lane on traffic speed was found to vary widely from site to site. The variations in mean speed upstream and downstream of a passing lane can range from an increase of 8.3 mph to a decrease of 6.7 mph. This wide range of speed differences between upstream and downstream suggests that vehicle speeds are influenced more strongly by local geometrics at the upstream and downstream measurement sites than by the presence of a passing lane. Spot speeds are more sensitive to local geometrics than platooning measures because drivers can quickly adjust their speed in response to an external influence, whereas vehicle platoons require time to develop.

Several attempts were made to model the effect of passing lanes on mean speed, in a manner similar to Equations 1 and 2 for vehicle platooning. However, the relationships obtained from these analyses were considered to be unreliable for predicting the effectiveness of passing lanes, because the underlying data are influenced so strongly by local geometrics.

Passing Rate

The rate of completed passes per hour per mile was determined for all or a selected portion of each passing lane and short four-lane section. The following analysis is based on the assumption that where passing maneuvers were observed for only a portion of an added lane, the portion of the lane studied is representative of the lane as a whole.

Treated Direction

The passing rates in the treated direction were found to range from 0 to 219.3 passes per hour per mile. The passing rate was found to have a strong relationship to flow rate, represented by the following regression model:

$$PR = 13.0 + 0.223FLOW \text{ for } 50 \text{ vph} \leq FLOW \leq 400 \text{ vph} \quad (3)$$

where PR is the passing rate in the treated direction in completed passes per hour per mile. This model explains 47 percent of the variance in the dependent variable (i.e., $R^2 = 0.47$).

Figure 4 compares the passing rate predicted by Equation 3 for passing lanes with a corresponding relationship for one direction of a conventional two-lane highway adapted from a relationship presented in the 1950 HCM (6). Although the latter relationship is of questionable value and was omitted from the 1965 HCM (7), the comparison serves to illustrate that passing lanes provide much higher passing rates than would be possible on a conventional two-lane highway.

An improved regression model for predicting the passing rate in the treated direction was obtained by adding two independent variables--passing-lane length and upstream percentage of vehicles platooned--to the model. The revised model for passing rate in the treated direction is

$$PR = 0.127FLOW - 9.64LEN + 1.35UPL \text{ for } 50 \text{ vph} \leq FLOW \leq 400 \text{ vph} \quad (4)$$

This model explains 83 percent of the variance in the dependent variable ($R^2 = 0.83$).

The model presented in Equation 4 shows that the passing rate increases with increasing flow rate and with increasing upstream percentage of vehicles platooned. The model also shows that the passing rate decreases with increasing passing-lane length. This finding tends to confirm the hypothesis that the passing rate is highest near the beginning of a passing lane and decreases to a lower, steady-state level at some distance into the lane.

Untreated Direction

Passing rates in the untreated direction were also studied for the 12 passing-lane sites. Passing by opposing-direction vehicles is permitted at 6 of the 12 passing-lane sites and prohibited at the remaining 6.

The passing rate on passing lanes where passing is permitted in the untreated direction varied from 0 to 50.0 passes per hour per mile. At these sites, there is a strong linear relationship between the passing rate and the flow rate in the untreated direction. The regression model for this relationship is

$$OPR = -6.97 + 0.13OFLOW \text{ for } 50 \text{ vph} \leq OFLOW \leq 400 \text{ vph} \quad (5)$$

where OPR is the passing rate in the opposing direction in passes per hour per mile and OFLOW is the flow rate in the untreated direction in vehicles per hour. This model explains 71 percent of the variation in the dependent variable (i.e., $R^2 = 0.71$).

Figure 4 shows that the passing rate in the untreated direction of a passing lane is substantially less than that in the treated direction but is higher than that for a conventional two-lane highway. Apparently more passes occur in the opposing direction of a passing lane than on a conventional two-lane highway because there are more passing opportunities available when the oncoming traffic can use two lanes rather than one.

The prohibition of passing in the opposing direction of a passing lane places that direction of travel at a distinct operational disadvantage. Despite the prohibition, a limited number of passing maneuvers do occur. The passing rates in the opposing direction ranged from 0 to 18.5 passes per hour per mile. No statistically significant relationship was found between opposing direction passing rate and flow rate for passing lanes where opposing-direction passing is prohibited.

SAFETY EVALUATION

A safety evaluation of the effectiveness of passing lanes and short four-lane sections was also performed. The purpose of this evaluation was to quantify the safety performance of these treatments in relation to comparable untreated sections and to detect any accident patterns or other safety prob-

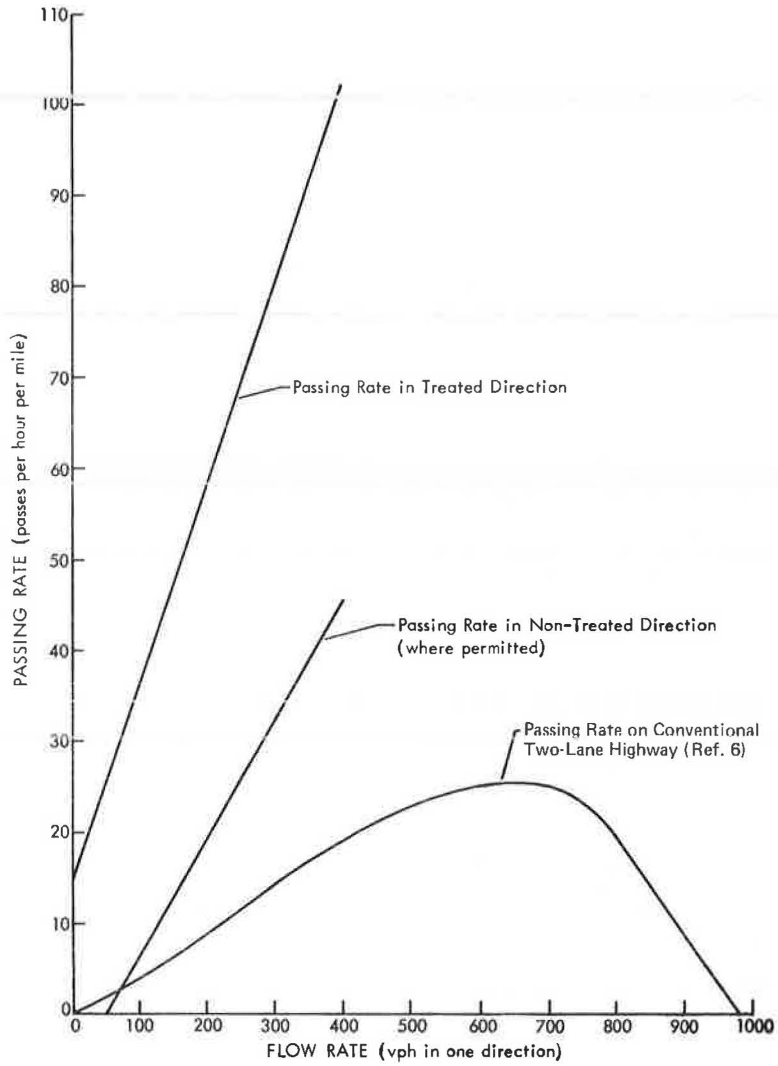

FIGURE 4 Passing rates in treated and untreated directions of passing lanes compared with conventional two-lane highways.

lems that might limit use of these treatments. Separate safety evaluations were performed for passing-lane and short four-lane sections.

Passing Lanes

Accident data were obtained from the participating states for a period of 1 to 5 years for each passing-lane site. The average length of the accident study period for the 66 passing-lane sites was 3.59 years. The results obtained from the analysis of these data are presented in the following paragraphs.

Comparisons Between Treated and Untreated Sites

Table 2 compares the mean accident rates for the treated and untreated directions of passing lanes and for comparable sections of untreated two-lane highway. The data presented indicate that the accident rates in passing lanes are slightly higher in the treated than in the untreated direction and that passing lanes have slightly lower accident rates than untreated two-lane highways. However, none of the differences between the means shown in Table 2 are statistically significant.

A matched-pair comparison was performed between 13 passing-lane sites and 13 corresponding untreated sites. The untreated sites were matched to the treated sites by the states that participated in the study. In all but two cases, the treated sites had a lower accident rate than the comparable untreated sites. The total accident rate of the passing-lane sites was, on the average, 38 percent less than that for comparable untreated sites and the fatal and injury accident rate was 29 percent less than that for comparable untreated sites. The observed difference in total accident rates was statistically significant at the 95 percent confidence level, but the difference in fatal and injury accident rates was not statistically significant.

Lane-Addition and Lane-Drop Transition Areas

A separate investigation was made of accidents in the lane-addition and lane-drop taper areas of passing lanes to determine whether there are any particular safety problems in those areas. Of the 305 accidents that occurred in the treated direction of passing lanes, 48 were found to occur in the first 800 ft of the passing lane and 51 in the final 800 ft. Figure 5 shows the distribution of accidents be-

TABLE 2 Comparison of Accident Rates for Passing Lanes and Untreated Two-Lane Highways

Type of Location	No. of Sites	No. of Accidents: Total	No. of Accidents: Fatal and Injury	Exposure (MVM)	Mean Accident Rate[a] (accidents/MVM): Total	Mean Accident Rate[a] (accidents/MVM): Fatal and Injury
Passing lane						
Treated direction[a]	66	305	133	271.0	1.13	0.49
Untreated direction	66	227	95	242.5	0.94	0.39
Both directions combined	66	532	228	513.5	1.04[b]	0.44[b]
Untreated two-lane highway (both directions combined)	13	430	226	273.5	1.57	0.83

Note: MVM = million vehicle miles.
[a]Including lane-addition and lane-drop transition areas.
[b]Based on average of accident rates for treated and untreated directions.

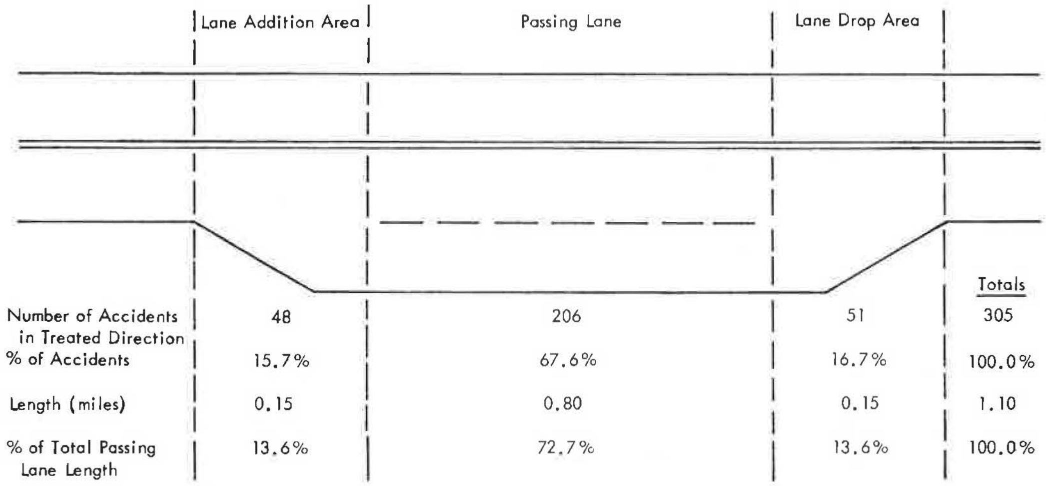

FIGURE 5 Distribution of accidents along a passing-lane section.

tween different areas of a typical passing lane. There is no indication that accidents are more likely in one transition area than in another. A slightly greater proportion of accidents occur in the transition areas than would be expected from their relative length alone, but the differences are not large. Thus, there is no indication of any marked safety problem in the lane-addition and lane-drop transition areas of passing lanes.

Studies of traffic conflicts and erratic maneuvers performed in the lane-drop transition areas of 10 passing-lane sites found no indication of safety problems associated with the transition area.

Although there is no evidence of a safety problem in lane-drop transition areas on the basis of the studies on accidents, traffic conflicts, and erratic maneuvers presented here, it is obvious that such transition areas should be carefully designed to prevent safety problems from developing. Many agencies that use alternating passing lanes either overlap them in the opposite direction or provide buffer areas between them to avoid a direct taper transition between passing lanes in opposite directions.

Cross-Centerline Accidents

Some agencies have been reluctant to install passing lanes on two-lane highways because of concern that such lanes might increase the likelihood of accidents between vehicles traveling in opposite directions, which are generally quite severe. In Table 3 the accident rates for cross-centerline accidents are compared for passing lanes with opposing passing prohibited, passing lanes with opposing passing per-

TABLE 3 Comparison of Cross-Centerline Accident Rates for Passing Lanes and Comparable Untreated Sections

Accident Severity Level	Passing-Lane Sections, Opposing Passing Prohibited: No. of Accidents	Exposure (MVM)	Accident Rate/MVM	Passing-Lane Sections, Opposing Passing Permitted: No. of Accidents	Exposure (MVM)	Accident Rate/MVM	Comparable Untreated Sections: No. of Accidents	Exposure (MVM)	Accident Rate/MVM
Fatal	6	234.7	0.026	5	278.8	0.018	7	273.5	0.026
Injury	15	234.7	0.064	12	278.8	0.043	39	273.5	0.143
Property damage only	10	234.7	0.043	14	278.8	0.050	28	273.5	0.102
Total	31	234.7	0.133	31	278.8	0.111	74	273.5	0.271

Note: MVM = million vehicle miles.

mitted, and comparable untreated sections. Cross-centerline accidents are defined here as all accidents that involve vehicles traveling in opposite directions; such accidents are predominantly head-on and opposing-direction sideswipe collisions. No substantial differences in accident rate were found at any severity level between passing-lane sections with opposing passing permitted and those with opposing passing prohibited, but both types of passing-lane sections have lower accident rates than do untreated two-lane highways. Thus, the provision for passing by vehicles traveling in the opposing direction to that of a passing lane does not appear to lead to any safety problems at the types of sites and the flow rate levels (up to 400 vph) where it has been permitted by the participating states.

Left-Turning Accidents

Accidents involving left-turning vehicles are a potential safety problem on passing-lane sections. A vehicle turning left into an intersection or driveway from the treated direction of a passing-lane section is in an exposed position if it must slow or stop in the left lane, which is normally the higher-speed lane, and yield to opposing traffic before completing a turn. However, it was found that only 8 accidents on the 66 passing-lane sections involved vehicles turning left from the treated direction. These accidents were not very severe: none were fatal, two were injury accidents, and six were property-damage-only accidents. Two of the eight accidents involved intersections and the remaining six were presumably driveway-related. On the other hand, the sample of untreated two-lane highways experienced 29 left-turn accidents of which none were fatal, 18 were injury accidents, and 18 were property-damage-only accidents. The untreated sections experienced virtually the same total travel as the treated direction of the passing-lane sections (273.5 and 271.0 million vehicle-mi of travel, respectively), so the two types of overall exposure data are comparable. Unfortunately, no complete data on left-turn volumes or the number of driveways and intersections are available to permit more precise exposure measures to be used. However, on the basis of the available data, there does not appear to be a safety problem associated with left-turn accidents in passing-lane sections.

Short Four-Lane Sections

The safety evaluation of short four-lane sections was based on accident data collected for nine short four-lane sections in three states--New York, Oregon, and Washington. Accident data were also available for six untreated two-lane highway sections located near all but one of the nine treated sections.

Comparison Between Treated and Untreated Sites

In Table 4 the overall accident experience for the treated and untreated sites is compared. The total accident rate for short four-lane sections is approximately 34 percent less than that for the untreated sections and the fatal and injury accident rate is 43 percent less, although these differences are not statistically significant. The accident rates for short four-lane sections and untreated sections presented in Table 4 are of comparable magnitude; the accident rates for passing lanes and untreated sections, respectively, are presented in Table 2.

A matched-pair comparison of accident rates for six short four-lane sections and six comparable untreated sections was also performed. In all but one case, the short four-lane sections had lower accident rates than the corresponding untreated sections. The total accident rate of the treated sites was 53 percent lower than that of the comparable untreated sites and the fatal and injury accident rate was 52 percent lower. Because of the small number of sites available, the mean difference in accident rates, although substantial, is not statistically significant for either total accidents or fatal and injury accidents.

Cross-Centerline Accidents

Table 5 shows that the rates for cross-centerline accidents on short four-lane sections are generally less than half of the rates for the same type of accidents on the comparable untreated sections.

CONCLUSIONS

Passing lanes and short four-lane sections were found to provide substantial operational benefits

TABLE 4 Comparison of Accident Rates for Short Four-Lane Sections and Comparable Two-Lane Highways

		No. of Accidents			Accident Rate/MVM	
Type of Location	No. of Sites	Total	Fatal and Injury	Exposure (MVM)	Total	Fatal and Injury
Short four-lane section	9	106	69	89.6	1.18	0.77
Comparable two-lane highway	6	250	189	139.4	1.79	1.36

TABLE 5 Comparison of Cross-Centerline Accident Rates for Short Four-Lane and Comparable Untreated Sections

	Short Four-Lane Sections			Comparable Untreated Sections		
Accident Severity Level	No. of Accidents	Exposure (MVM)	Accident Rate/MVM	No. of Accidents	Exposure (MVM)	Accident Rate/MVM
Fatal	3	89.6	0.033	1	139.4	0.007
Injury	10	89.6	0.112	45	139.4	0.323
Property damage only	4	89.6	0.045	10	139.4	0.072
Total	17	89.6	0.190	56	139.4	0.402

when used as an operational treatment on two-lane highways. Both types of added lanes increase the passing rate in the treated direction to several times the passing rate that would occur on a conventional two-lane highway. By using Equation 4, passing rates in passing lanes and short four-lane sections can be predicted as a function of flow rate, length of treated section, and upstream percentage of vehicles platooned.

The percentage of vehicles platooned is reduced by nearly half (from 35.1 to 20.7 percent of vehicles following in platoons) within a passing lane. The percentage of vehicles platooned immediately downstream of a passing lane is 6 percent less than the upstream value (29.2 versus 35.1 percent); the persistence of these downstream benefits is variable and highly dependent on the characteristics of particular sites. These results imply that at 250 vph (a typical flow rate for a passing lane on a two-lane highway) if 90 vehicles are following in platoons upstream of a passing lane during a given hour, only 50 vehicles will be following in platoons within the passing lane and only 75 vehicles will be following in platoons immediately downstream of the passing lane. The operational benefits of passing lanes can persist for several miles downstream from the treated section.

The reduction in platooning from upstream to downstream of a passing lane can be predicted as a function of the upstream percentage of vehicles platooned and the length of the added lane by using Equation 1. Further research is being conducted through computer simulation of traffic operations on two-lane highways with and without passing lanes. This research will address questions of fundamental importance to designers, including the optimal length and frequency of passing lanes under different conditions of traffic flow and terrain.

A safety evaluation found that the installation of a passing lane on a two-lane highway does not increase the accident rate and in fact probably reduces it. No unusual safety problems were found to be associated with either lane-addition or lane-drop transition areas. The rate of accidents involving vehicles traveling in opposite directions was found to be the same or lower on passing-lane sections than on untreated two-lane highways at all severity levels, even for passing lanes where passing by opposing-direction vehicles is permitted. No safety problems associated with vehicles making left turns from the treated direction of a passing lane were found.

A substantially lower accident rate was found for short four-lane sections than for comparable untreated two-lane highways. The accident rates involving vehicles traveling in opposite directions on short four-lane sections were generally less than half of the rates found on comparable untreated sections. Because of the small sample size available for short four-lane sections, the statistical significance of these conclusions could not be demonstrated.

ACKNOWLEDGMENT

The research reported in this paper was performed as part of an FHWA contract on alleviation of operational problems on two-lane highways. The authors acknowledge this sponsorship.

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The findings and conclusions in this paper are those of the authors and do not necessarily represent the views of FHWA.

Publication of this paper sponsored by Committee on Operational Effects of Geometrics.

Abridgment

Benefit-Cost Evaluation of Left-Turn Lanes on Uncontrolled Approaches of Rural Intersections

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ABSTRACT

Left-turn lanes are provided on uncontrolled approaches of rural intersections to improve the safety and efficiency of traffic operations on these approaches. Although the safety and operational effects of left-turn lanes are well recognized, there are no generally accepted guidelines that define the circumstances under which the costs of these lanes are justified by the benefits that they provide. The objectives of this research were (a) to evaluate the benefits and costs of left-turn lanes on the uncontrolled approaches of intersections on rural two-lane highways and (b) to determine the traffic volumes that warrant these lanes in Nebraska. The road-user cost savings associated with the reductions in accidents, stops, delay, and fuel consumption provided by left-turn lanes were evaluated over a range of traffic volumes and compared with the costs of left-turn lanes over the same range. The safety effectiveness of the lanes was based on accident experience on rural two-lane highways in Nebraska. The NETSIM traffic simulation model was used to determine their operational effectiveness. Volumes for which the road-user cost savings exceeded the lane costs were determined to warrant left-turn lanes. The warrants developed in this research are limited to prevailing conditions typical of those on rural two-lane highways in Nebraska. However, the procedure used to develop these warrants is applicable to other locations.

Left-turn lanes are provided on uncontrolled approaches of rural intersections to improve the safety and efficiency of traffic operations on these approaches. The primary function of these lanes is to remove the deceleration and storage of left-turning vehicles from the through traffic lanes and thereby enable through vehicles to pass by without conflict and delay. Thus, the benefits derived from the provision of these left-turn lanes are reductions in accidents, stops, and delay.

Although the safety and operational effects of left-turn lanes are well recognized by highway engineers, there are no generally accepted guidelines that define the circumstances under which the costs of constructing and maintaining left-turn lanes are justified by the benefits that they provide. Intersection design guides (1-4) currently used by highway engineers contain criteria for the geometric design of the elements of left-turn lanes, such as taper lengths, storage lengths, and lane widths. But these design guides do not contain warrants for left-turn lanes. Without acceptable warrants, the only means highway engineers have to determine the need for left-turn lanes are experience and judgment, which vary considerably among individuals. Acceptable left-turn lane warrants would not only improve the consistency of decisions to construct such lanes, but on the basis of a benefit-cost evaluation would also provide for their cost-effective use. Thus, left-turn lane warrants based on a benefit-cost evaluation would enable the determination of the need for left-turn lanes at specific locations and would promote the most cost-effective allocation of available funds among competing highway projects.

PREVIOUS RESEARCH

Several studies (5-9) have reported the safety effects of left-turn lanes on the uncontrolled approaches of rural intersections. However, few studies have quantified the operational effects of left-turn lanes at these locations. In addition, a review of the literature revealed only three studies that were designed to develop warrants for left-turn lanes on the basis of a benefit-cost analysis (10-12), but the limited scope of these studies made their findings inapplicable for the purposes of this research.

OBJECTIVES

The objectives of the research reported in this paper were (a) to evaluate the benefits and costs of left-turn lanes on the uncontrolled approaches of intersections on rural two-lane highways and (b) to determine the traffic volumes that warrant the construction and maintenance of these lanes in Nebraska. This paper presents the procedure, findings, and conclusions of this research.

PROCEDURE

The benefits provided by left-turn lanes are reductions in accidents, stops, and delay. The road-user cost savings associated with these benefits were evaluated over a range of traffic volumes and compared with the costs of constructing and maintaining left-turn lanes over the same range. Volumes for which the road-user cost savings were greater than the left-turn lane costs were determined to be those volumes that warrant left-turn lanes. A description of the procedure used to evaluate each component of the road-user cost savings and the left-turn lane costs follows.

Accident Cost Savings

An analysis of accidents occurring at rural intersections in Nebraska was conducted to determine the effectiveness of left-turn lanes in reducing accidents on the uncontrolled approaches of intersections on rural two-lane highways. Intersection accident data were obtained from the Nebraska Department of Roads for the 3-year period from January 1, 1977, to December 31, 1979. From these data, the numbers and types of accidents that occurred on each of the intersection approaches were determined. Previous research (6-9) indicated that the primary safety effects of left-turn lanes on rural intersection approaches were reductions in the numbers of rear-end and left-turn accidents. Therefore, rear-end and left-turn accident rates were computed for each approach. Mean rear-end and left-turn accident rates were then computed for each approach category. These mean rates are shown in Table 1. T-tests conducted at the 5 percent level of significance within each shoulder category indicated that there were no statistically significant differences in rear-end and left-turn accident rates between approaches with left-turn lanes and those without left-turn lanes.

TABLE 1 Mean Accident Rates

	No Paved Shoulder		Paved Shoulder	
Accident Type	No LT Lane	LT Lane	No LT Lane	LT Lane
Rear-end	0.44	0.19	0.31	0.28
Left-turn	0.03	0.26	0.000	0.10

Note: Accident rates are expressed as accidents per million entering vehicles. LT = left-turn.

Despite the fact that no statistically significant safety effects of left-turn lanes were found, an accident reduction factor was computed for each accident type from the difference between the mean accident rates with and without left-turn lanes within each shoulder category. The accident reduction factors computed are shown in Table 2 along with accident reduction factors found in the literature (6-8).

TABLE 2 Accident Reduction Factors

	Accident Type	
Source	Rear-End (%)	Left-Turn (%)
Nebraska		
Without shoulder[a]	60	-770[c]
With shoulder[b]	10	$-\infty$[d]
NCHRP *(6)*	20	-
FHWA *(7)*	80	50
Hammer *(8)*	85	37

[a]Approaches without paved shoulders.

[b]Approaches with paved shoulders.

[c]Increase in mean left-turn accident rate was 770 percent.

[d]Undefined percentage of increase in mean accident rate because approaches without left-turn lanes had a zero mean left-turn accident rate.

The accident reduction factors computed for rear-end accidents indicated that left-turn lanes were more effective in reducing rear-end accidents on approaches without paved shoulders than on approaches with paved shoulders. The rear-end accident reduction factor computed for approaches without paved shoulders was within the range of the rear-end accident reduction factors found in the literature, whereas the rear-end accident reduction factor computed for approaches with paved shoulders was lower than those found in the literature. However, it was not apparent from the literature that the effects of paved shoulders had been considered in the previous research. Therefore, the rear-end accident reduction factors computed from the Nebraska data were used for the purposes of this research.

The accident reduction factors computed for left-turn accidents indicated that left-turn lanes were not effective in reducing left-turn accidents but were associated with increases in left-turn accidents. In the Nebraska data, a left-turn accident was defined as a collision between a left-turning vehicle and an opposing vehicle. Consequently, these findings suggested that perhaps sight-distance problems between left-turning and opposing vehicles were created by the provision of left-turn lanes or that the intersection approaches with left-turn lanes had more left-turn accidents merely because they had higher left-turn volumes. However, as shown in Table 2, these findings were contrary to those reported by previous research (7,8), but the definition of left-turn accidents used in these studies may have included collisions between left-turning vehicles and other vehicles than opposing ones. For this reason, and because properly designed left-turn lanes would not be expected to create sight-distance problems between left-turning and opposing vehicles, it was assumed that left-turn lanes provided no reductions in left-turn accidents on approaches with and without paved shoulders.

Therefore, accident reduction benefits of left-turn lanes used in this research were reductions in rear-end accidents only. The accident cost savings provided by left-turn lanes were computed by using the rear-end accident rates and the rear-end accident reduction factors shown in Tables 1 and 2 for uncontrolled approaches at intersections of rural two-lane highways in Nebraska.

Operational Cost Savings

The benefits of reduction in stops and delay provided by left-turn lanes result in operational cost savings to the road users. The operational cost savings are composed of reductions in motor vehicle operating costs and time costs because of fewer stops and less delay. Previous studies (10-14) have found the reductions in stops and delay provided by left-turn lanes to be functions of approach volume, opposing volume, left-turn volume, approach speed, and percentage of trucks. However, none of these studies formulated an expression for the total operational cost savings resulting from these reductions.

In order to determine the effectiveness of left-turn lanes in reducing stops and delay on the uncontrolled approaches of intersections on rural two-lane highways, a series of computer runs was conducted over a range of approach conditions using the NETSIM traffic simulation model (15). One set of runs was made with left-turn lanes on the approaches and a second set without left-turn lanes on the approaches. Both sets of runs were made over the same range of volumes, approach speeds, and truck percentages. The effects of left-turn lanes on stops and delay were then determined by a pairwise comparison of the NETSIM stops and delay output from the two sets of runs for identical combinations of volumes, approach speeds, and truck percentages. Because the NETSIM output included fuel consumption, the effect of left-turn lanes on fuel consumption was also determined in this manner. Thus, for every combination of volume, approach speed, and truck percentage, the effects of left-turn lanes on stops, delay, and fuel consumption were computed as the differences between the respective outputs of the two runs for the approaching traffic. By using a modified response surface experimental design (16,17), more than 2,500 pairs of simulation runs were made. The details of these runs are documented elsewhere (18).

A multiple-regression analysis of the results of the simulation runs was conducted to determine the relationships between the benefits of left-turn lanes and the approach conditions. As a result of this analysis, three regression equations were determined for the prediction of the reductions in stops, delay, and fuel consumption provided by left-turn lanes, which were used to compute the operational cost savings.

The operational cost savings were computed by using (a) unit vehicle operating costs determined by Claffey (19) for passenger cars and updated to the year 1983 on the basis of changes in the national consumer price index (CPI) [private transportation: tires (new, tubeless), motor oil, and automobile repairs and maintenance (20,21)], (b) the unit value of time established by AASHTO (22) for the year 1975 and updated to the year 1983 on the basis of the change in the CPI (20,21), and (c) the fuel economy of the weighted 1971 composite vehicle in the NETSIM model (23) corrected for the increased fuel economy of the 1983 vehicle fleet in accordance with the fuel economy adjustment factors obtained by Apostolos (24). The annual operational cost savings were computed for the average vehicle mix, average vehicle occupancy, and average hourly distribution of daily traffic that existed on rural two-lane highways in Nebraska during 1983 (25).

Left-Turn-Lane Costs

The costs of a left-turn lane were computed to be the additional costs required to construct and maintain a painted left-turn lane on an uncontrolled intersection approach within the existing right-of-way on rural two-lane highways in Nebraska. Based on a review of left-turn-lane projects constructed in 1983, the Nebraska Department of Roads estimated the additional costs of a left-turn lane to be \$6 per square foot of additional pavement required. This unit cost included construction cost items of earthwork, asphaltic-concrete pavement, and drainage, and maintenance cost items of pavement markings and snow removal over the life of the project. Thus, the construction and maintenance costs of a left-turn lane were computed by multiplying the additional square feet of pavement area required by the left-turn lane times the unit cost of \$6 per square foot. The construction and maintenance costs were then annualized by multiplying them by the capital recovery factor (0.11746) for a 10 percent interest rate, 20-year service life, and zero salvage value. The dimensions of the left-turn lane configuration used to compute the additional pavement area required were determined as a function of approach speed and left-turn volume in accordance with Nebraska design standards (26).

CONCLUSIONS

On the basis of the results of the benefit-cost evaluation conducted in this research, the following conclusions were reached in regard to the provision of left-turns on the uncontrolled approaches of four-leg intersections of rural two-lane highways in Nebraska:

1. The approach volumes at which left-turn lanes were warranted were dependent on the prevailing approach conditions, in particular the left-turn percentage, opposing volume, approach speed, and shoulder condition.
2. The approach volumes required to warrant left-turn lanes were considerably higher on approaches with, rather than without, paved shoulders, because of the lower rear-end accident rates and the reduced effectiveness of left-turn lanes in reducing rear-end accidents on such approaches.
3. Under no circumstances on approaches without paved shoulders were left-turn lanes warranted at an approach annual average daily traffic (AADT) of less than 2,500 vehicles per day (vpd). On approaches with paved shoulders, left-turn lanes were never warranted at approach AADTs of less than 4,500 vpd.

It must be remembered that the benefit-cost evaluation conducted in this research was based on accident experience, traffic conditions, road-user unit costs, and left-turn lane costs that were intended to be representative of intersections on rural two-lane highways in Nebraska during 1983. Consequently, on approaches with higher than average accident rates or truck percentages or both, left-turn lanes may be warranted at volumes lower than the warranting volumes found in this study. In addition, the use of different unit road-user costs and left-turn lane costs would also alter the findings of this study. Higher unit road-user costs and lower left-turn lane costs would reduce the warranting volumes. Conversely, lower unit road-user costs and higher left-turn lane costs would increase the warranting volumes.

ACKNOWLEDGMENT

This paper is based on research undertaken as part of Project HPR 79-3, Development of Warrants for Special Turning Lanes at Rural Nonsignalized Intersections, which was conducted by the Civil Engineering Department, University of Nebraska-Lincoln, in cooperation with the Nebraska Department of Roads and FHWA, U.S. Department of Transportation.

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Publication of this paper sponsored by Committee on Operational Effects of Geometrics.

Abridgment

Superelevation and Roadway Geometry: Deficiency at Crash Sites and on Grades

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ABSTRACT

Survey data on roadway superelevation, curvature, and grade collected at the sites of fatal rollover accidents and at comparison sites in New Mexico and Georgia were analyzed to determine the effect of grade on superelevation after adjustment for curvature. These adjusted data were then used to determine the effect of superelevation on accidents. After adjustment for curvature, it was found that in comparison with flat roadway sections (grade +2.5 to -2.5 percent) sections with grade (greater than +2.5 or less than -2.5 percent) had less superelevation. After adjustments for both curvature and grade, fatal rollover accident sections were found to have less superelevation than comparison sections. Inadequate superelevation presents a risk that should be eliminated from the roadway system.

The influence on accidents of superelevation rates--the vertical cross-slope or banking of the pavement on curved roadways--is a feature of highway design that has not been examined in detail. In a correlational analysis of rural highway geometry and accidents in Louisiana, it was reported that roadways with relatively flat cross-slopes have higher accident rates than those with greater cross-slope (1). However, this analysis did not account for the roadway curvature or vehicle speeds. A study of rural isolated curves using surrogate measures for accident experience found the degree of curve and superelevation deficiency to be the best predictors (R^2 = 0.68) of the accident rate for vehicles running off the road (2). Engineering surveys of sites of single-vehicle accidents all found that, on the average, the superelevation rates at these sites were higher than those at the comparison sites, but this result was most likely because of higher frequency of curves at the accident sites (3-6). These studies also noted that the superelevation rates at the sites of fixed-object accidents tended to be greater than those at the sites of rollover accidents.

The results of investigations on two distinct but related questions are reported. First, after adjustment for curvature, what is the relation between superelevation and grade? Second, after adjustments for both curvature and grade, what is the effect of superelevation on fatal single-vehicle rollover accidents? (A more detailed report of the statistical analysis and results is available from the authors at the Insurance Institute for Highway Safety.)

METHODS

Engineering survey data from rural primary roads (principal arterials and Interstates) and secondary roads (minor arterials and collectors) in Georgia and New Mexico were analyzed. Surveys were made at locations centered on a reference point where a fatal single-vehicle rollover accident had occurred, at comparison locations 1 mi upstream from the accident location, and at a stratified random sample of 300 sites representing the rural roadway system of each of the states in terms of average daily traffic.

At each accident and comparison location, 10 curvature and superelevation and 11 gradient measurements were obtained along a 100-ft roadway section centered on the accident or comparison location. At random sites, measurements of curvature, superelevation, and grade were taken 50 ft before and after the reference points. The methods for obtaining these measurements are given in detail elsewhere (5,6).

The basic units for statistical analysis were roadway sections 100 ft long, which were described by one measurement of superelevation rate and curvature and two measurements of vertical alignment. The grade of a section was taken to be the average value of the grade at its beginning and at its end. Sections that were straight, had excessive curvature, or had large increases in curvature relative to adjacent sections (e.g., curve transition sections) were eliminated from the analyses. Sections were classified as downhill, flat, or uphill according to whether the average grade was below -2.5 percent, between -2.5 percent and +2.5 percent, or above 2.5 percent, respectively. Sections were also classified as accident sections (upstream from the actual accident); downstream sections (just past the actual accident); or comparison sections (including sections 1 mi away from the accident site, and those randomly selected).

The effects of grade and section type on the linear relationship between superelevation rate and curvature was studied by using regression analysis [SAS general linear model procedure (7)]. In these analyses the superelevation rate was assigned a negative sign when the edge of the traveled lane was below the center of the traveled roadway (typical for right curves) and a positive sign when it was above the center of the roadway (typical for left curves). Curves turning left were assigned a negative sign and curves turning right were assigned a positive sign.

Equation 1 represents the model for studying the effect of grade:

$$\text{Superelevation}_{sk} = A_{0s} + A_{1s}\ \text{curvature}_{sk} + \text{error}_{sk} \quad (1)$$

where

s = 1 for crash sections,
s = 2 for downstream sections,
s = 3 for comparison sections, and
k = 1,..., K_s corresponds to the different sections.

The model for studying the effect of section type was similar except that s = 1 if the section had a downhill grade, s = 2 if it was flat, and s = 3 if it had an uphill grade. Both of these models were estimated separately for all combinations of parameters, including state (New Mexico or Georgia) and type of roadway (Interstates and principal arterials or minor arterials and collectors).

The regression coefficients in Equation 1 were estimated and the regression lines corresponding to the effect studied (e.g., grade or section type) were compared. To assess the effect of vertical alignment on superelevation, the estimated excess superelevation was calculated for both uphill and downhill sections by using the flat sections as the standard, that is, by subtracting the estimate for the flat section from the estimate for the graded section. Similarly, to assess the effect of superelevation on accidents, the estimated excess superelevation at accident sections was calculated by using the comparison sections as the standard reference. Thus, negative excess resulting from a comparison of a specific section with a standard section indicates deficient superelevation.

RESULTS

The regression coefficient of superelevation on curvature, A_{1s} in Equation 1, was predictably negative and statistically significant for all combinations of state, roadway class, vertical alignment, and section type.

The superelevation deficiency estimates for all uphill and all downhill comparison sections (upstream and random) are plotted in Figure 1 by state and road class. Sections with substantial curvature and grade had deficient superelevation except for primary roads in New Mexico. This was true regardless of the direction of turn. In all four cases, the intercepts of the regression lines describing superelevation as a function of curvature were found to vary significantly by grade. The regression coefficients of curvature were significantly different by grade for all cases where deficient superelevation was found (i.e., except for primary roads in New Mexico). It should be noted that for sections with positive vertical alignment the results display a somewhat erratic pattern for primary roads; this is probably because of the small sample sizes (N = 25 in New Mexico and N = 22 in Georgia).

For accident sections, the regression lines did

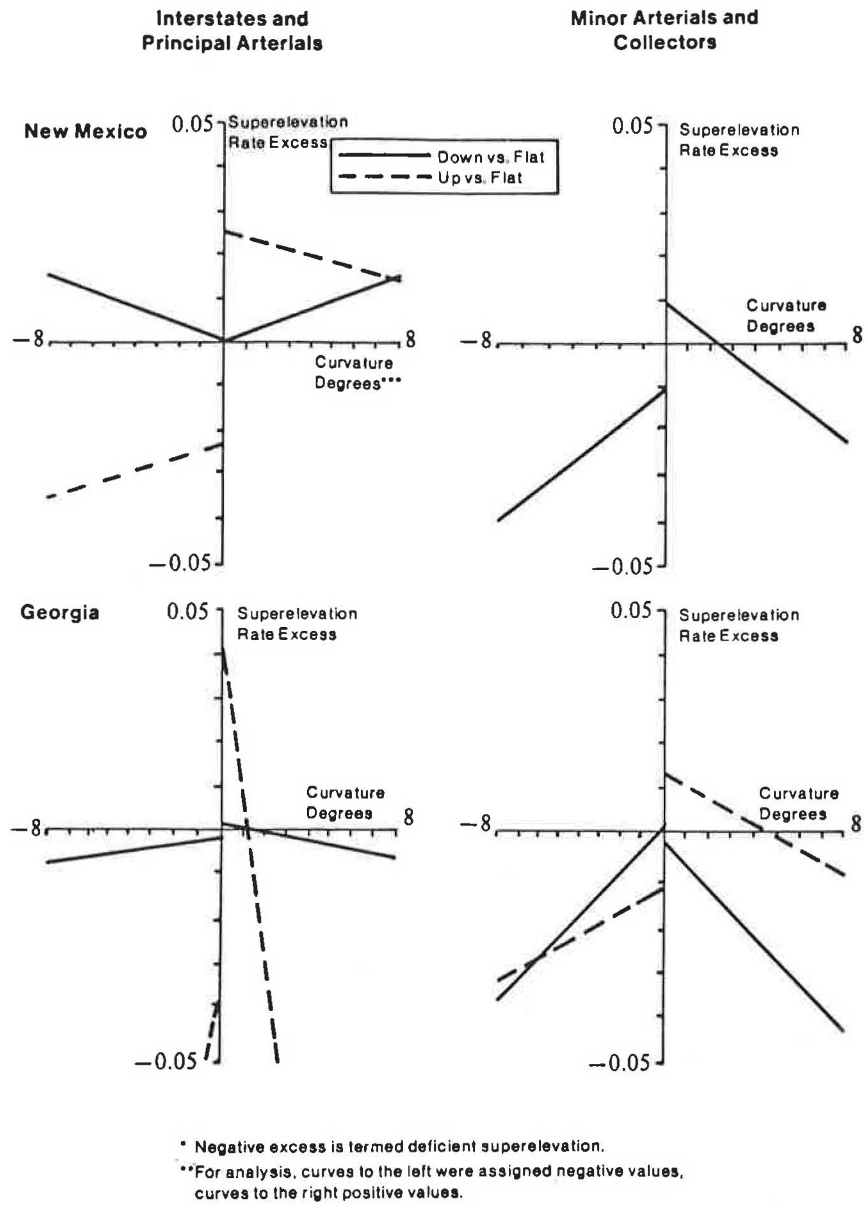

FIGURE 1 Excess of superelevation for roadway sections with uphill or downhill grades versus flat sections based on regression estimates for comparison sites by state, road class, and vertical alignment.

not vary significantly by grade. For downstream sections, there was significant variation in the slopes of the regressions by grade for primary roads in Georgia, and superelevation deficiencies were observed for sections with higher curvature values.

The excess in superelevation for accident sections relative to that of comparison sections is plotted for flat sections in Figure 2 by state and road class. A consistent pattern of deficiency is indicated with the single exception of right-curving sections on secondary roads in New Mexico. In all the other comparisons by state and road class, the regression coefficient of curvature varied significantly by section type.

For sections with grade, the regression coefficients for curvature varied significantly by section type only for primary roads in Georgia. Overall, the results indicated a deficiency for sections with downhill vertical alignment. For sections with uphill vertical alignment the sample size was small (N = 22), and the results showed superelevation excess.

The proportion of accident sections among flat accident and comparison sections was modeled as a function of curvature, grade, and superelevation excess by using the method of logistic regression (8). The results are given by direction of curve, road class, and state in Table 1. As an illustration, among left curves on secondary roads the estimated proportion (p) of crash sections in New Mexico is $p = 1/(1 + e^{-x})$ where $x = -0.78 - 0.31c - 0.12g - 0.26s$ (c = curvature, g = grade, and s = superelevation excess). Note that this proportion of accident sections increases for sharper left curves, steeper downgrades, and increasing superelevation deficiency.

As the chi-square results in Table 1 show, the model accounted for a significant amount of the variation in the proportion of accident sections in all eight analyses. The rank correlations between

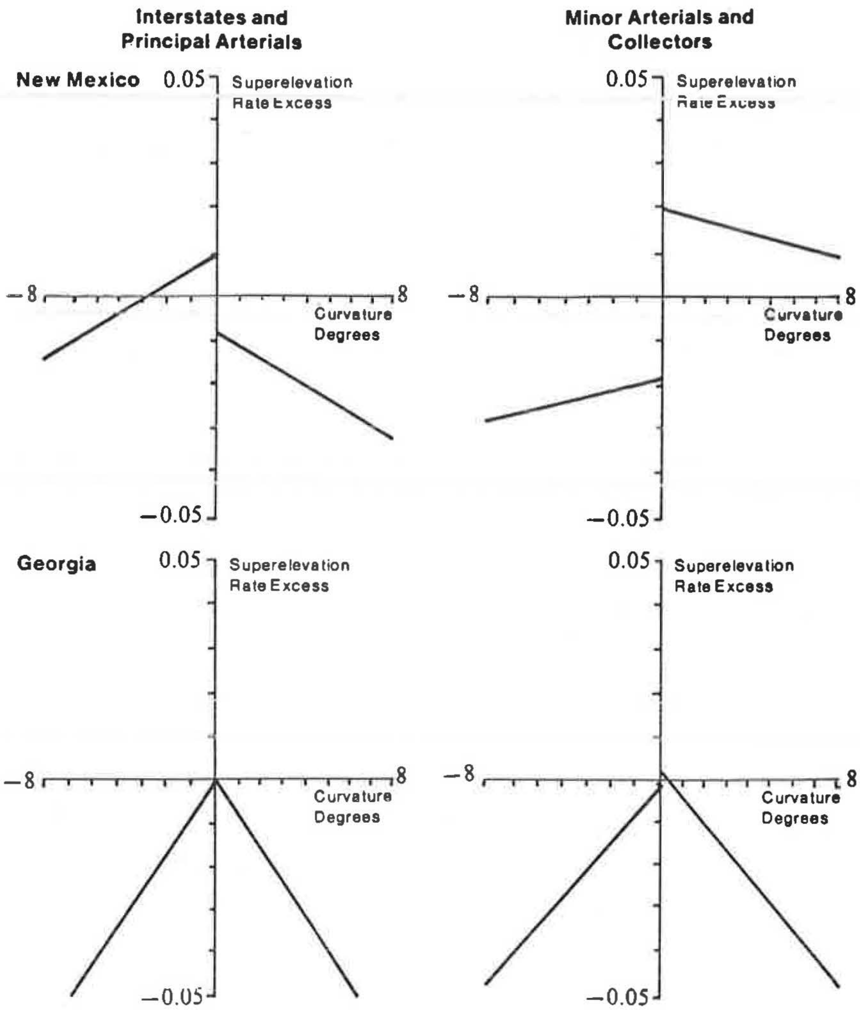

FIGURE 2 Excess of superelevation rate at accident sites over superelevation rate at comparison sites based on regression estimates for flat roads by state and road class.

TABLE 1 Determinants of Accident Sites: Comparison of Accident and Comparison Sections with Flat Vertical Alignment by State, Road Class, and Direction of Curve

		Interstates and Principal Arterials		Minor Arterials and Collectors	
Direction of Curve	Parameter	New Mexico	Georgia	New Mexico	Georgia
Left	Intercept	1.01[a]	-1.21[b]	-1.00[b]	-0.78[a]
	Curvature	-0.08	-0.38[a]	-0.09	-0.31[b]
	Grade	0.67[b]	-0.42	-0.53[a]	-0.12
	Excess superelevation	0.21	-0.22	-0.22[c]	-0.26[b]
	Chi-square (3 DF)	15.7[a]	17.6[b]	18.0[b]	36.6[b]
	Concordant pairs	0.74	0.70	0.71	0.71
	Rank correlation	0.49	0.41	0.44	0.42
	N (accident)	78	58	51	117
	N (comparison)	31	79	74	93
Right	Intercept	-1.21[c]	-0.75[c]	-1.53[a]	-0.76[a]
	Curvature	0.54	0.63[a]	0.44[a]	0.21[a]
	Grade	-0.65[b]	0.13	-0.73[c]	0.01
	Excess superelevation	-0.34[c]	0.01	1.20[b]	-0.19[b]
	Chi-square (3 DF)	20.7[b]	15.2[a]	39.7[b]	17.7[b]
	Concordant pairs	0.75	0.66	0.86	0.69
	Rank correlation	0.50	0.35	0.73	0.40
	N (accident)	45	66	34	72
	N (comparison)	59	52	42	83

Note: Dependent variable y = 1 for accident sites and y = 0 for comparison sites. DF = degrees of freedom.
[a]Significant at 0.01 level.
[b]Significant at 0.001 level.
[c]Significant at 0.05 level.

predicted probability and observed response varied between 0.35 and 0.73. The proportion of accident sections increased for sharper curves in all eight analyses, and this effect was statistically significant in five analyses. Although in these analyses of flat sections grade was limited to the range from -2.5 percent to +2.5 percent, the proportion of accident sections increased for steeper downgrades in five of eight analyses and in three of four analyses where the effect was significant. The proportion of accident sections also increased with increased superelevation deficiency in five of the eight analyses and in four of the five analyses when the effect was statistically significant. (The one anomalous result appears to be statistically unstable.) The adverse effects of sharp curves, downhill grades, and superelevation deficiency are most clearly present on secondary roads.

DISCUSSION AND CONCLUSIONS

The relationship between superelevation and grade was examined for roadways in New Mexico and Georgia. Compared with rates for flat road sections, the rates of superelevation were found deficient on both uphill (grade greater than +2.5 percent) and downhill (grade less than -2.5 percent) sections. Because these results were based on comparisons between the linear regression estimates of superelevation rates as functions of curvature, the deficiency in superelevation cannot be due to curvature differences between flat road sections and those with grade. This finding holds true for many of the parameters examined, including state, road class, and section type, although the strength of the relation did not reach statistical significance in all comparisons. However, in all cases with statistically significant differences the sections with uphill or downhill grades were deficient in superelevation.

Superelevation is intended to counter the outward forces generated when the direction of the vehicle's motion changes along curved paths of travel. Because speeds on downhill grades tend to be higher than on otherwise similar flat grades and the outward forces increase with speed, the superelevation rates on such grades should not be less than those at comparable flat curves. If downhill grades were designed for realistic travel speeds, the rate of superelevation would be higher at curves with downhill grades than that at comparable flat curves because of the higher average speeds of vehicles traveling downhill. Although AASHTO only partially endorses a policy of using increased banking to adjust the design speed on downhill curves to allow for the higher speeds of travel on such curves (9,p.194), the prevalence of reduced superelevation rates at such locations is clearly dangerous. In computer simulation analyses with the highway-vehicle-object simulation model (HVOSM) it was found that the most critical parameter in assessing friction demands on curves was the vehicle speed (10). Increasing the operating speed of the vehicle 12 mph increased tire versus pavement friction needs by at least 50 percent, which was often significantly above AASHTO design values.

The superelevation rates at accident sections were found to be deficient compared with those at comparison sections. Because these analyses were also adjusted for curvature, this deficiency cannot be due to curvature differences. Although statistical significance was reached mostly for flat road segments (-2.5 percent to +2.5 percent grade), this finding is also generally valid regardless of state, road class, and grade. Moreover, a majority of the logistic regression analyses for separating accident sections from comparison sections in terms of grade and curvature were significantly improved when a measure for superelevation deficiency was added to the other alignment measures.

Other roadway characteristics that may be statistically associated with the occurrence of single-vehicle rollover accidents (e.g., pavement condition, maximum superelevation rate, and design speed) were not considered in this paper. Although the effects of these characteristics on accidents, if any, were partly controlled for in that approximately three-quarters of the comparison sections were located on the same roads (1 mi upstream from the accident site) as the accident sections, these results should not be construed to mean that the three roadway alignment components are the only important environmental factors playing a role in single-vehicle accidents.

A possible explanation for the observed deficiency of superelevation at curves on grades is that current design practices were not successfully applied. In general, this does not appear to be the case. However, because many of the roads investigated have been in use over a considerable time, their superelevation deficiencies may be the result of out-of-date design, construction, or maintenance practices. The possibility of the settling of road foundations cannot be excluded. However, the analysis indicated that accident sections had significantly lower superelevation rates (particularly for flat curves) than nearby downstream sections. Alternatively, it is possible that in many instances the design speed is simply set too low, so that the superelevation is nominally adequate but not in line with actual travel speeds.

Regardless of the historical causes, the widespread deficiencies found in the rates of superelevation at locations where challenging road geometry tests both driving skills and vehicle handling present a clearly defined added risk that should be systematically monitored and gradually eliminated from the roadway system.

Discussion

Timothy R. Neuman*

The authors of this paper are to be commended for addressing a subject that receives too little attention. Appropriate design of highway curvature must include consideration of superelevation. Much recent research, including other studies published by these same authors, strongly demonstrates the importance of highway curvature in safe operation of high-speed highways. It is also noted that studies of this nature that address minutely varying design elements are extremely difficult to conduct. It is remarkable that any sensitivities were uncovered at all, given the narrow range of superelevation variance and the many other factors that play a role.

In general, the findings reported here appear reasonable. However, certain important questions need addressing before full acceptance of the research is possible. These questions relate to certain unmentioned but important variables and apparent assumptions that may be imprecise.

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First, the research focuses exclusively on the deficiencies in superelevation at curve sites. It is noted that, among other factors, pavement friction plays a role in vehicle stability. Clearly, available pavement friction and its relationship to vehicle dynamics as well as its distribution at accident versus comparison sites should be considered. Assumed friction factors for design purposes are nominally equivalent to superelevation, as shown by the standard curve formula

$$e + f = V^2/15R$$

where

- e = superelevation (ft/ft),
- f = friction,
- V = design speed (mph), and
- R = radius of curve (ft).

Identically designed curves (in terms of superelevation and radius) would have distinctly different safety and operating characteristics given actual differences in pavement friction. This, in fact, is shown by previous research, including detailed studies of highway curves recently completed by Jack E. Leisch and Associates (JEL) for FHWA. In those studies, available pavement friction was found to be a small but significant variable in prediction of high-accident curve sites. The fact that comparison sites were closely downstream from the curves in the data base does not totally control for pavement friction variances. Pavement wear is variable, with curves (particularly sharper ones) wearing faster than tangent sections.

An additional variable of extreme importance is that of the method of developing superelevation and its effects on dynamics and safety. Analysis of vehicle behavior on approaches to curves shows the transition area (150 ft each side of the PC) to be the most critical part of the curve. Again, identically designed curves in terms of radius and maximum superelevation would operate differently under various methods of developing the superelevation. (It is noteworthy that the JEL curve studies uncovered a slight but statistically significant contribution of amount of superelevation at the PC to high-accident location prediction.)

The statistical analysis itself is predicted on a simplification of the relationship between curvature (defined in terms of degree of curve) and maximum superelevation. The simplification, that the two are linearly related, causes potential problems given that the true relationship is nonlinear. Figure 3, the design curves of superelevation for e_{max} of 0.10, demonstrates the true nonlinear relationship. (It is assumed here that curves in the study sample were designed to a nonlinear policy similar or identical to the relationship shown in Figure 3. This is undoubtedly the case, because design practice in this area has remained essentially unchanged for many years.) If the sample of accident sites is even slightly overrepresented by curves of greater than 5 or 6 degrees, a different linear model would be expected than one created by comparison sites. In other words, differences between the two models may be explained more by the underlying sample distributions of curvature within the accident and comparison sites than by differences in design of superelevation. This point is important given that differences that were observed were very slight (which would be expected given the narrow design range of superelevation).

A far more important question, and one that appears to be at the heart of the authors' findings, is the subject of design speed. In attempting to explain the reasons for accident occurrence, the authors focus on superelevation deficiency. It is more likely, and entirely within reason given the type and age of roads in the study sample, that design speed explains the results. Many of the curves are undoubtedly designed for a speed much too low for prevailing operating conditions. Such curves could be characterized as deficient in terms of superelevation. More likely, and more to the point in terms of design, the deficiency is in the curvature itself.

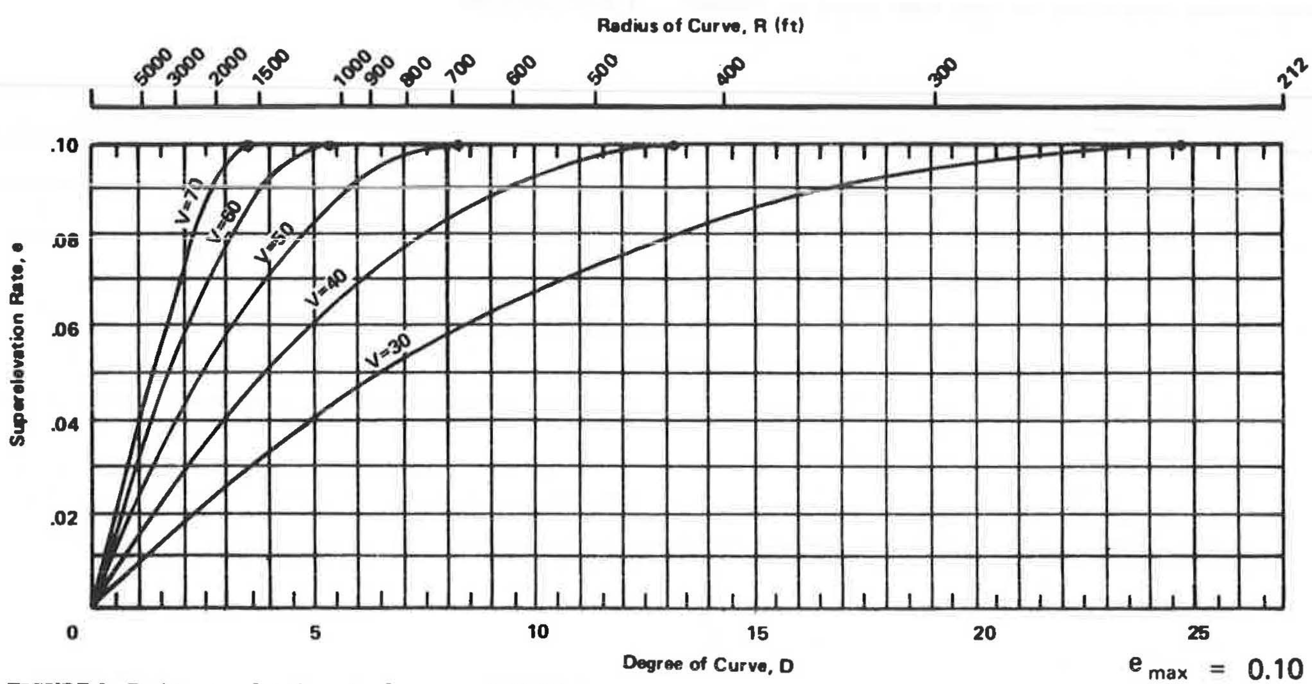

FIGURE 3 Design superelevation rates for e_{max} = 0.10 *(9)*.

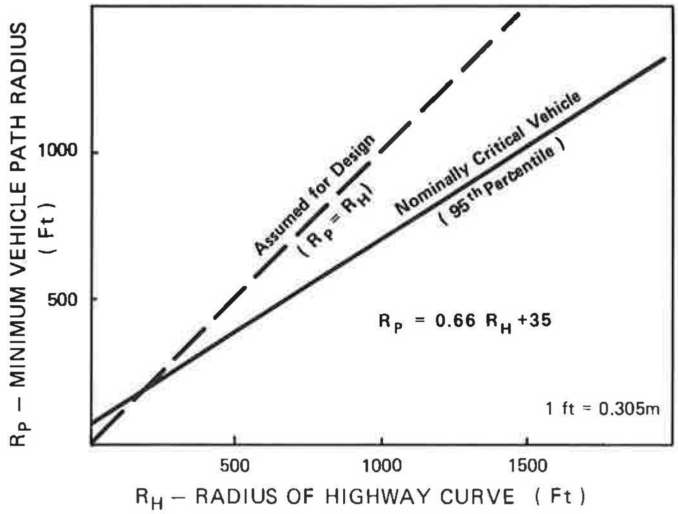

FIGURE 4 Relationship of vehicle path curvature to highway curvature *(11)*.

Glennon et al. demonstrated, in a recently published study of operations on highway curves (11), that the combination of entering speed and curvature is by far the most critical factor in vehicle control. Figure 4 shows the results of vehicle tracking behavior, which is strongly related to roadway curvature. To summarize, drivers tend to "overdrive" curves, that is, to track transient paths sharper than the curvature of the roadway. Furthermore, drivers' approach speeds are influenced very little by the impending curve, whether it is visible, signed, or not evident. Drivers also tend not to adjust their speed completely until they are well within the curve. Curves that are too sharp for the prevailing speeds of a given highway are thus prime candidates for the types of overturn and run-off-the-road accidents discussed here. And, in general, such curves tend to be so underdesigned (i.e., have nominal design speeds much lower than the operating speed) that marginal improvements to superelevation would be of little or no help.

To conclude, I agree that proper superelevation design is critical to safe operations on curves. However, more fundamental questions that appear to be addressed here are what the relationship is between curvature and design speed and what factors determine a safe and reasonable design speed for a curve.

Authors' Closure

We agree with Neuman that design speeds should be in line with the prevailing operating conditions. Slow design speeds are compatible with sharp curves, and a series of studies (3-6) sponsored by the Insurance Institute for Highway Safety over the last decade, including the study on which this paper is based, have demonstrated that the likelihood of single-vehicle crashes is greatly increased on curves of greater than 6 degrees, even where these curves are adequately superelevated. The new findings in this study were that inadequate superelevation poses additional hazard to drivers and that superelevation tends to be inadequate on roadways with grades. The first of these two findings was apparently also confirmed in the JEL curve studies referred to by Neuman.

We also agree with Neuman that the choice of proper curves to match operating conditions and the choice of proper superelevations for these curves are fundamental to safe road design. If these choices are not correct, the curve will be underdesigned both in terms of its curvature and its superelevation. This, as Neuman points out, could be especially hazardous for drivers whose actual travel path is even sharper than that of the curve.

In his discussion of the statistical methods, Neuman notes that if the sample of accident sites contains more curves of greater than 5 or 6 degrees than does the sample of comparison sites, the reported superelevation deficiency at the accident sites may have been caused by incorrectly modeling the concave superelevation-curvature relationship by linear regressions (Figure 3). This observation is correct in theory, but it does not apply to most of the data analyzed in the paper (see Table 2). For the four matched comparisons between left and right curves in the two states that involved accident and comparison sites on flat (between -2.5 percent and +2.5 percent grade) primary roads, it can be seen from Table 2 that the 95th percentiles of curvature for the comparison sample always exceed those for the accident sample. Also, all four of the latter were below 5 degrees, which is well within the range over which the design superelevation-curvature function is linear.

For secondary roads, the situation is less clear-cut because all eight 95th percentiles of curvature exceeded 6 degrees and the 95th percentile of curvature for the accident sample was below the 95th percentile of the comparison sample only for right curves in New Mexico. It should be noted, however, that the operating speeds and the design speeds of these roads would most likely be lower than those on primary roads, and consequently the relationship between curvature and superelevation would remain linear over a larger range (Figure 3). In any case, most of the linear models fit the data quite well and explained about 60 percent of the variability in the superelevation rate.

Commenting on the study design, Neuman noted that it did not totally control for possible differences in pavement friction between the accident and comparison sites. This is true; however, some measure-

TABLE 2 Summary of 95th Percentiles of Curvature and Superelevation Rate Distributions for Flat Curves by State, Road Class, Section Type, and Direction of Turn

		Accident Site		Downstream Site		Comparison Site	
Road Class	Type of Curve	Curvature	Superelevation Rate	Curvature	Superelevation Rate	Curvature	Superelevation Rate
New Mexico							
Primary	Left	4.8	7.2	5.5	5.6	4.8	8.4
	Right	2.9	4.8	2.3	5.8	6.6	7.2
Secondary	Left	14.7	11.3	9.4	8.2	9.4	7.7
	Right	7.0	6.4	7.1	5.6	10.0	8.0
Georgia							
Primary	Left	4.5	9.1	4.5	8.4	5.5	8.8
	Right	2.6	4.6	8.8	8.2	2.8	6.3
Secondary	Left	12.0	8.0	10.5	7.5	6.3	8.9
	Right	12.8	12.6	10.0	6.6	7.4	8.2

Note: Primary road class = Interstates and principal arterials; secondary = minor arterials and collectors. Vertical alignment was defined as follows: down = grade < -2.5 percent, flat = -2.5 to +2.5 percent grade, and up = grade > 2.5 percent.

ments of friction were made in New Mexico and these indicated that no substantial differences in friction existed at matched accident and comparison sites (5). Moreover, even if such differences did exist, it could be argued that inadequate superelevation would tend to result in harder braking in the curve and therefore the superelevation inadequacy itself was the cause of the lowered friction.

It should be noted that the method of developing superelevation may be important for vehicle dynamics, but it is likely that this method was typically the same at accident and comparison sites that were only 1 mi apart. Because most of the comparison data in this study were collected at these matched sites, the effect on the results of such differences should be minimal. More generally, the matching technique used in these studies controlled for the effects of most other design-related differences as well.

The main findings are as follows:

1. After adjustment for curvature, it was found that in comparison with flat roadway sections (grade +2.5 to -2.5 percent) sections with grade (greater than +2.5 or less than -2.5 percent) had less superelevation.
2. After adjustments for both curvature and grade, sections with fatal rollover accidents were found to have less superelevation than comparison sections.

As stated in the paper, it is not yet known why these differences in superelevation exist. Possible explanations are being researched, but the lack of explanation for the differences does not diminish their importance. In addition, the adequacy of the key geometric design features (e.g., design speed, curvature, gradient, and superelevation) should be carefully assessed when roadway maintenance and rehabilitation are undertaken, and deficiencies should be corrected regardless of their source.

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Publication of this paper sponsored by Committee on Operational Effects of Geometrics.

Design and Safety on Moderate-Volume Two-Lane Roads

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ABSTRACT

The effects of geometric design and traffic characteristics on accidents on two-lane rural roads with a moderate average daily traffic (ADT) between 400 and 3,000 vehicles per day were studied for two data sets. Geometric design elements were aggregated into "bundles," or groups frequently found together in the field as a result of design policies. Advanced multivariate analysis techniques were used to explore their relationships with accident experience. It was found that accidents interact in such a complex way with ADT that the rate of accidents per mile-year is superior to the conventional rate of accidents per vehicle-mile for models developed from typical accident data files. Models using almost 20 geometric and traffic variables explained about 70 percent of the variance in accident frequency on several hundred road sections. Of this, ADT accounted for 30 percent and intersection and driveway frequency, about 25 percent more. Models using only ADT, access density, and geometric bundles made up of up to 5 design variables performed as well as did the 20-variable models. Simple categorical models using only the bundles explained approximately 50 percent of the variance in accident density in both data sets. Illustrative mathematical models were also developed. Off-road accidents increase with ADT exponentiated to .5 to .9; the exponent increases as bundle characteristics become worse. Comparison of sections with the best and worst accident records showed the strong influence of the geometric design bundles on accident experience. Results of the research indicate that treating geometric and roadside elements as clusters rather than individually is a worthwhile approach for safety improvement programs.

Safety on moderate-volume rural highways is an important issue. Two-lane roads with an average daily traffic (ADT) between 400 and 3,000 vehicles per day (vpd) constitute approximately 90 percent of the paved rural collector and arterial highway systems in the United States (1). On these roads accident rates are high; fatal and injury vehicle-mile exposure accident rates (VMER) are up to 3.5 times as great as those on Interstate highways (2). Their vast mileage and relatively low use present a problem to highway agencies concerned with safety improvement priorities.

Geometric design, traffic use, intersectional, and access characteristics on these routes vary widely and hence choices among the many possible improvements are particularly difficult. The understanding of the effects of geometric elements on safety has not been adequate to predict the accident response to individual geometric design element changes with reasonable accuracy.

The objective of this research was to explore the interactive effects of geometric design elements and traffic characteristics on accidents on these roads and to identify some promising prediction models useful in engineering decisions. Groups of design elements frequently used together at the same time as a result of design policies or construction practices, called bundles in this research, were formed and explored as an alternative to the study of individual geometric design elements.

PREVIOUS STUDIES

Accidents on two-lane rural roads with an ADT less than 3,000 vpd have been examined as part of studies concerned with broader ranges of ADT (3) and in studies concerned specifically with moderate ADT levels (4-6). The findings from these studies about the effects of geometric design elements on safety are mixed and conflicting (5,7,8). For example, no clear and consistent effects of shoulder and pavement width emerge for this volume range (9). Although access points (driveways and intersections) have been found to influence accident rates (10,11), they have also been reported not to have much effect on accident occurrence at lower ADT levels (5,12).

The effect of ADT on accidents is understood best and it is generally accepted that there is a positive relationship between VMER and ADT. When accident measures other than VMER are used, such as accidents per mile-year (MYER), the effects of traffic volume are even stronger (9-11,13).

The effect of a single geometric element is difficult to identify with typical data sets because of the confounding of geometric and operating elements in actual highway installations (8,14). The interacting effects of the individual elements were clearly shown in a 1960 study (10). Factor analysis showed strong correlations among the various road elements, forming four factors, the first of which captured horizontal and vertical alignment effects, the second the conflict effect of ADT and access density, the third the cross-sectional elements, and the fourth the roadside elements. Together these factors explained 70 percent of the accident variance. However, no categorical or mathematical model summarizing this capability was presented.

The model fit of the relatively few reported mathematical models relating accident occurrence to any of the flow characteristics or geometric design elements in the moderate-volume ADT range (15,pp. 103-109; 16-20) has generally been poor. An exception to this is a model constructed by Cleveland and Kitamura (21), which predicted off-road accidents. Because of large interactions between ADT and the geometric elements the model was stratified for three ranges of ADT below 3,500 vpd. The explanatory variables included ADT and percentage of passing-sight distance restricted. A roadside obstacle measure was added for the highest ADT category. A validity check performed on an independent data set found the model performance satisfactory.

Roy Jorgensen and Associates (19) analyzed relationships for such highways. The model was also stratified for three ranges of ADT: <1,000 vpd, 1,000 to 2,500 vpd, and 2,500 to 5,000 vpd. The variables included in their noninteractive regression analysis were lane width, shoulder width, shoulder surface type, and curvature for VMER models. ADT and section length were added for total accident predictor models. The regression models explained little of the accident data variance, generally less than 8 percent. This is seen as confirmation of the need to take into account interactions among design elements.

METHODOLOGY

The two original data sets that were used in the study of high-volume highway safety (3) were analyzed--the FHWA skid file (16) and the file of off-road accidents along Michigan State highways (21). AASHTO policies (22,23) consider rural routes with design hourly volumes up to 400 vph as moderate-volume rural roads that would be equivalent to an ADT of 2,500 to 3,000 vpd. Accordingly, this analysis is extended to routes with an ADT of up to 3,000 vpd.

As in the high-volume safety study, urban-type segments were eliminated from the FHWA skid file. To minimize the correlation between accidents and segment length (24), only those sites between 3 and 12 mi long were retained for this study. The final sample in the file contained the 1-year accident history and descriptions of the geometric elements for 109 rural two-lane segments from 11 states. In the file there is a total of 1,302 accidents with an average of 11.9 accidents per year per road segment and an average MYER of 1.80 accidents per mile per year on about 724 mi of road.

The second data set, referred to as the Michigan State route set, contains a sample of 164 rural paved 2-mi sites with a 4-year accident experience of more than 610 off-road accidents. Besides accident frequency, the data include ADT and intersection information, geometric characteristics, and some data on roadside obstacles.

A sequence of models was built for each data set by reducing the number of independent variables as much as possible and keeping the power of the model as measured by accident variance explanation as high as possible. The following analysis methodology was used:

1. Statistical examination of data to identify the variables that contribute most to the explanation of variation of accident occurrence by using Automatic Interaction Detection (AID) (25),
2. Identification of reasonable groups of correlated variables by using factor analysis,
3. Grouping of interrelated roadside and geometric design elements into reasonable "bundles,"
4. Determination of the ability of the bundles to explain accident variance and comparison of that with the variation explanation of several individual geometric design elements,
5. Exploration of the characteristics of the segments with the best and worst accident experience, and
6. Development of a promising illustrative categorical or mathematical model.

The smallest AID split used in this research has a minimum of five segments in a group. Differences in means are statistically significant at the 90 percent confidence level. In the figures presented in this paper only differences in average accident experience of at least 30 percent are illustrated.

Analysis of Michigan-FHWA Skid File

Table 1 lists the variables used and summarizes the results of 12 AID analyses. The first step in the analysis was the selection of the accident exposure figure of merit from vehicle miles traveled (VMER), annual length exposure (MYER), and total annual accidents per section per year (SYER). Previous modeling in broader ranges of ADT (3,9,11,17,26) has shown that VMER usually requires adjustment for ADT. This was confirmed for this data set (Model 1 in Table 1). An important split occurred on ADT at a level of 2,500 vpd, indicating that the differential effect of ADT was still very important, even after being included as a part of the dependent variable. This model captured 65.1 percent of the variance. An AID analysis of SYER (Model 2 in Table 1) with all 19 possible individual elements as variables explained 73.2 percent of the variance but still showed segment length to be an important AID descriptor.

The analysis of the third candidate measure, MYER, as the dependent variable (Model 3 in Table 1) explained 74.4 percent of the variation, and segment length did not appear in any of the important AID splits, as shown in Figure 1. Tests of the independence of MYER accident density and segment length showed that segments with lengths between 3 and 12 mi had no significant bias, whereas the longer sections had lower MYER values.

Factor analysis was then conducted to explore which variables tended to vary together. Five factors with reasonable physical interpretations were identified. The first factor showed that 85th-percentile speed, shoulder width, percentage of length with guardrail, and percentage of length on curves are related to the state in which the site is located. This is a result of state design and operating policies and terrain. The second factor captured horizontal alignment variables. Mean skid number at the 85th-percentile speed was the only variable in the third factor, indicating that the skid number is reasonably independent of the other road features. The fourth factor was composed of access-point density variables and vertical alignment. The fifth factor captured flow characteristics and had important contributions from ADT--85th-percentile speed and pavement width. Results of this analysis were used to help in AID interpretations and in the initial bundle definitions.

An AID MYER run using only ADT (Model 4) explained 32.0 percent of the variance. When only ADT and intersection density were used (Model 5), 40.5 percent of the variance was explained. When ADT, intersection density, and driveway density were considered (Model 6), 55.0 percent of the variance was explained, an increase of 14.5 percent. The remaining explained variance of 19 percent is attributable to the other 16 variables of Model 3, 8 of which appear in significant splits.

For the initial AID analysis using MYER (Model 3, Figure 1) the first major splits were on the location of the segment (state) for ADT less than 2,500 and number of curves per mile for segments with an ADT between 2,500 and 3,000. Because factor analysis had indicated that the state of location was highly correlated with cross-section and operating descriptors, it was eliminated from further consideration, resulting in a decrease in the variance explanation of only 2.3 percent (Model 7). Of the 18 candidate independent variables, 10 were significant and 72.1 percent of the variance was explained. These in-

TABLE 1 AID Summary: Michigan-FHWA Skid Accident File

		Model											
Variable Name	Symbol	1	2	3[a]	4	5	6	7[a]	8	9	10[a]	11[a]	12
Dependent variable													
VMER (accidents/million vehicle-miles)	–	●											
SYER (accidents/section/yr)	–		●										
MYER (accidents/mile/yr)	–			●	●	●	●	●	●	●	●	●	●
Flow and location													
State	State	●	●	●									
Length	Len	○	●	●				○					
ADT	ADT	●	●	●	●	●	●	●	●	●	●	●	●
Overall intersection density	OID	○	○	○		●		○					
Overall access-point density	Acc Den	●	○	●			●	●	●	●	●	●	●
85th-percentile speed	85% Sp	●	○	○				●		●			●
Mean skid number at 85th-percentile speed	Skid #	○	○	●				●					
Time	–	●	●	○				●					
Cross section													
Pavement type	Pv Typ	○	○	○				○					
Pavement width	Pv Wid	○	○	●				●	●	●			
Shoulder treatment	Sh Tr	○	○	○				○					
Percentage of length with shoulder narrower than 6 ft	%Sh <6 ′	●	●	●				●	●	●			
Alignment													
Percentage of no-passing zone in both directions	PSR	○	○	○				●	●	●			
No. of curves per mile	NC	●	●	●				●	●	●			
Percentage of length on curves	PCL	●	○	●				●		●			
No. of sag curves per mile	NSC	●	○	○				○					
Percentage of length on significant grades	PSG	●	○	●				○					
Roadside hazard													
No. of obstacles within 10 ft	OB 10	●	●	●				○					
Percentage of guardrail in both directions	PGR	○	○	○				○					
Bundles													
Cross-section	Xs Bun										●		
Alignment	Al Bun										●		
Geometric	Geo Bun											●	●
Variation explained (%)	–	65.1	73.2	74.4	32.0	40.5	55.0	72.1	66.8	70.0	62.1	65.4	66.2
No. of final groups	–	16	18	18	8	13	16	18	16	18	16	17	18

Note: ○ = variable did not appear in significant split; ● = variable appeared in significant split.
[a]AID branch diagrams shown in Figures 1-4.

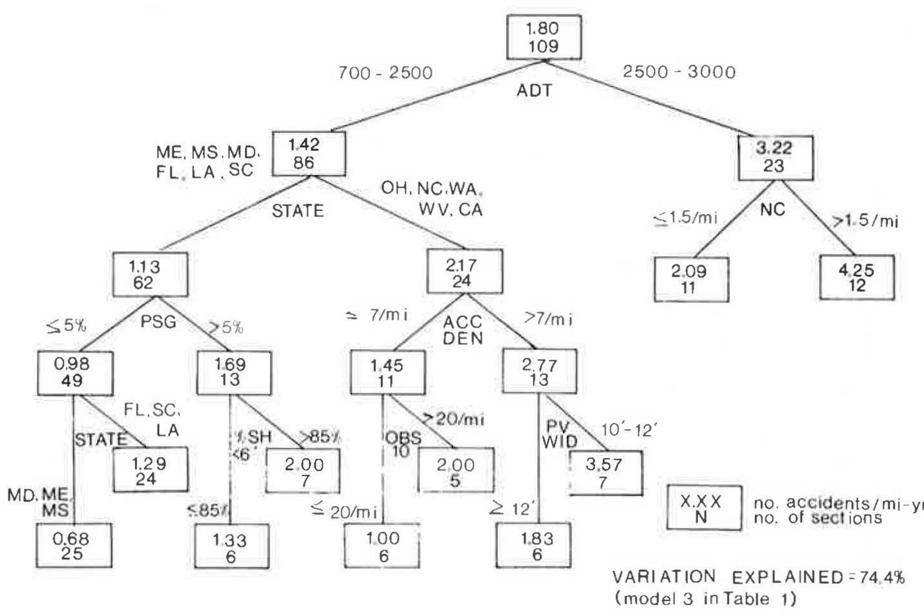

FIGURE 1 Michigan-FHWA skid AID branch diagram using all variables.

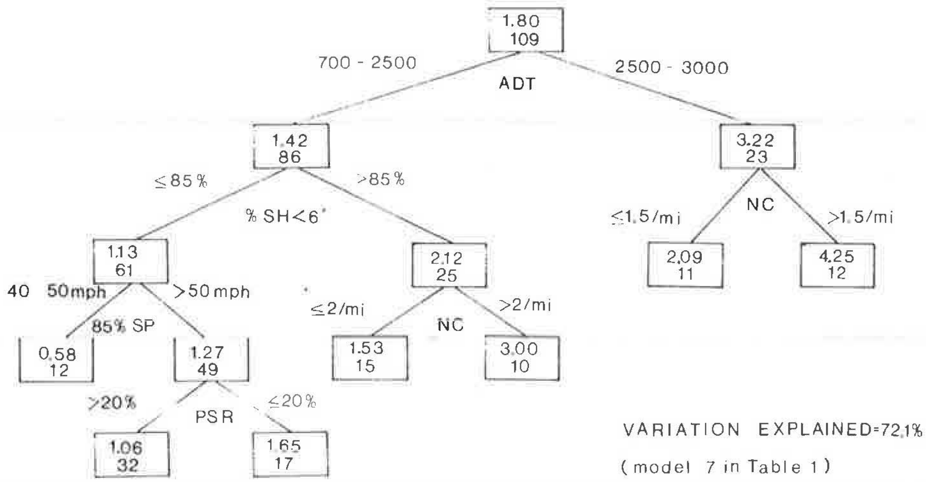

FIGURE 2 Michigan-FHWA skid AID branch diagram using all variables except state.

cluded ADT, speed, access-point density, skid number, two cross-section design elements, and three longitudinal alignment elements. The roadside hazard measures made no significant contribution to explaining these results.

As shown in Figure 2 for the sections with 700 to 2,500 vpd, those segments with more length with shoulder wider than 6 ft had about one-half the accident experience of the other sections. Among these wider shoulder segments those with lower 85th-percentile speeds again had half the MYER of those sections with higher speed. Another large difference in MYER was found for the higher-speed segments with better shoulders where the passing restriction in the segment was less for sections with higher accident experience. In all the AID analysis in this research this was the only result in which better geometrics were associated with higher accident experience. Further analysis of this subgroup showed the confounding effects of horizontal curves, which were overrepresented in sections with passing-sight distances.

Simplifying the model further, an AID analysis was next run with six of the most significant explanatory variables (Model 8), including ADT, access-point density, two alignment variables, curve frequency, and passing-sight distance restriction, and two cross-sectional variables, pavement and shoulder width. The variation explained by these six variables was 66.8 percent, only 5.3 percent less than that of Model 7 with its 10 significant variables.

In Model 9 the 85th-percentile speed and percentage of road with curves, other important variables from Model 7, were added. The results gave only a small variance improvement to 70.0 percent.

The four Model 8 geometric variables with their total of 36 significant levels were then grouped into six cross-sectional and five alignment bundles as defined in Tables 2 and 3. Of the 30 possible combinations, 24 of them existed in this data set. The bundles were loosely ordered and numbered from best to worst. An AID analysis using only four variables--ADT, access density, and the cross-sectional and alignment bundles (Model 10)--explained 62.1 percent of the variation. The AID branch diagram is presented in Figure 3, which shows the same first split on ADT at 2,500 vpd. For sections with 2,500 to 3,000 vpd, those segments with alignment bundles characterized by more than three curves per mile had the highest MYER. Segments in the same ADT category but with no more than two curves per mile experienced fewer accidents. Among these segments those with cross-sectional bundles characterized by wider shoulders had lower accident experience. In the category of 700 to 2,500 vpd, the first split in MYER occurred on the cross-sectional bundles; those 12 segments with the worst designs had the highest MYER. Differences in access density were also important.

The next step was to determine whether further reduction in the number of variables by combining the cross-sectional and alignment elements into one overall geometric bundle would affect the explanatory power of the model. An overall geometric bundle with five categories was created and the definitions are given in Table 4. Figure 4 shows the results of the AID analysis using only ADT, access density, and the overall geometric bundle (Model 11). The variation explained was 65.4 percent, better than the alignment and cross-section bundles, nearly as good as the individual elements themselves, and less than 7 percent weaker than Model 7, which used 10 variables rather than 3. When the 85th-percentile speed was considered along with the geometric bundle

TABLE 2 Cross-Section Bundles for Michigan-FHWA Skid Accident File

Bundle Designation	Length with Shoulder <6 ft (%)	Lane Width (ft)	No. of Sections
Xs-1	≤55	≥12	24
Xs-2	56-85	≥12	12
Xs-3	86-100	≥12	15
Xs-4	≤55	10-12	29
Xs-5	56-85	10-12	10
Xs-6	86-100	10-12	19

TABLE 3 Alignment Bundles for Michigan FHWA Skid Accident File

Bundle Designation	No-Passing Zone (%)	No. of Curves per Mile	No. of Sections
Al-1	0-15	0-2	22
Al-2	16-40	0-2	36
Al-3	>40	0-2	18
Al-4	0-40	≥3	13
Al-5	>40	≥3	20

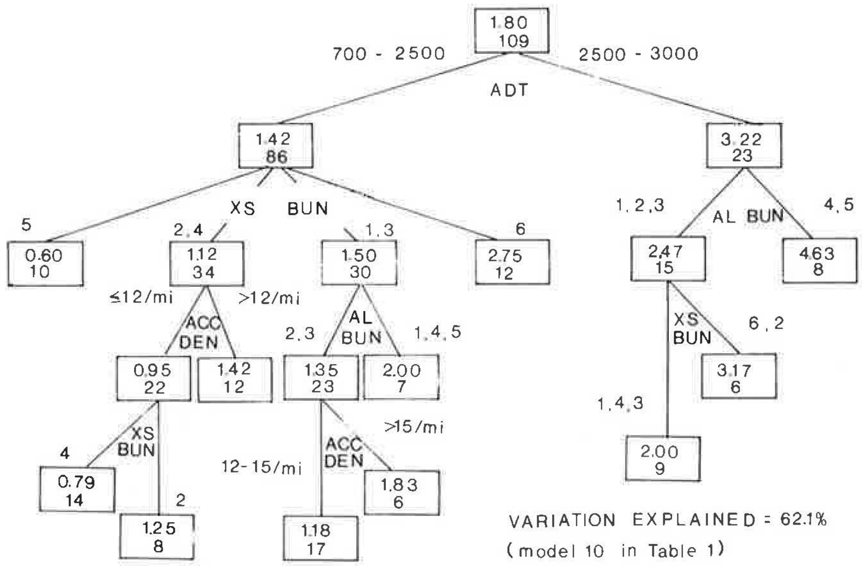

FIGURE 3 Michigan-FHWA skid AID branch diagram using six cross-section and five alignment bundles.

TABLE 4 Definition of Geometric Bundles for Michigan-FHWA Skid Accident File

Bundle Designation	Length with Shoulder <6 ft (%)	Lane Width (ft)	No. of Curves per Mile	No. of Sections
Geo-1	All	≥12	0-2	43
Geo-2	0-85	10-12	0-2	25
Geo-3	All	≥12	≥3	8
Geo-4	0-85	10-12	≥3	14
Geo-5	85-100	10-12	All	19

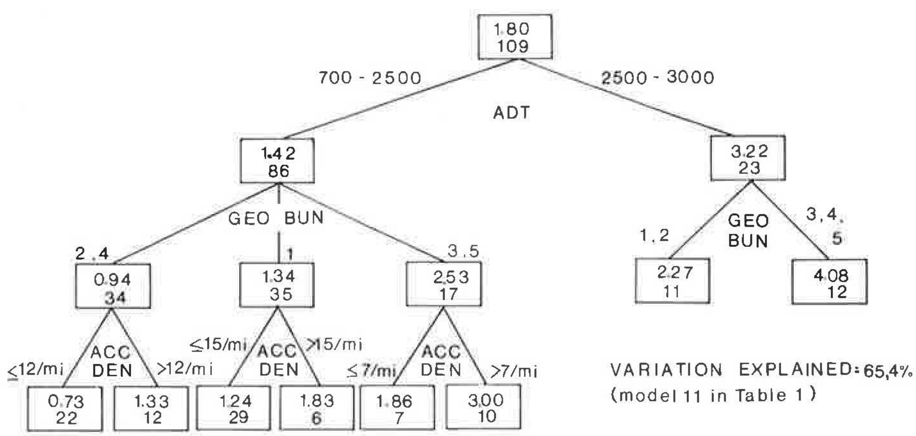

FIGURE 4 Michigan-FHWA skid AID branch diagram using overall geometric bundle and ADT and access density.

(Model 12), a small improvement to 66.2 percent was detected.

A comparison of the 11 sections with the worst accident experience (MYER of 4 to 7, more than twice the average of 1.80) against the 13 sections with no accidents was made. The worst accident sections had the following significant (α = 0.05) relative characteristics:

- More traffic (2,500 versus 1,600 vpd),
- More length with narrower shoulders (75 versus 39 percent),
- More curves (8.2 per mile versus 5.0 per mile),
- More length with significant grades (12 versus 6 percent),

- More length with guardrails (14 versus 9 percent), and
- Lower skid number (37.5 versus 45.1).

The variables associated with significant grades and guardrails were not important in the AID analyses. The factor analysis showed that both of these were in the first factor, which was associated with state operating policies and terrain effects. Other variables from this factor that were highly correlated with these variables appeared in significant AID splits and made a greater contribution to the explained variance. Other variables not showing great explanatory power in this extreme value analysis did appear as strong predictors in the AID models. In formulating bundles, the lane width and the percentage of length with passing-sight restrictions were added.

The strong relationship between accident experience and the geometric characteristics of road sections reflected in the geometric bundles is revealed as follows:

Section Classification (MYER)	Geometric Bundle 1	2-3	4	5	Total
Best (0)	3	5	5	0	13
Worst (4-7)	1	1	3	6	11

The 13 best sections with no accident and the 11 worst sections with more than twice the average accident experience were classified into bundles. It is noted that all the best sections belonged to the best or near-best geometric bundles, whereas more than half of the worst sections belonged to the worst geometric bundle and very few were associated with good geometric characteristics. The results are statistically significant ($\alpha = 0.05$).

Table 5 presents a geometric-bundle-based cross-classification model of MYER accident occurrence. The results of the AID analyses were used to define the divisions for the categories of ADT and access-point density used in this model. The simple categorical model thus obtained explained 52.2 percent of the variance in this data set.

An illustrative mathematical model for Geo-1, the bundle with the best geometric features, was developed in which the MYER rate was expressed approximately as shown in Figure 5. Actual data are shown along with the model contours.

TABLE 5 Cross-Classification Model for Michigan-FHWA Skid Accident File

ADT (vpd)	Access-Point Density (points/mi)	No. of Accidents per Mile-Year by Geometric Bundle[a] Geo-1	Geo-2	Geo-3	Geo-4	Geo-5
700-1,500	0-12	0.57	0.57	–[b]	0.57	2.20
	>12	1.00	1.00	–[b]	1.00	2.20
1,500-2,000	All	1.46	0.90	–[b]	2.25	2.25
2,000-2,500	0-8	1.40	0.60	1.80	0.60	1.80
	>8	1.40	1.20	2.80	1.20	2.80
2,500-3,000	All	2.27	2.27	4.08	4.08	4.08

[a]See Table 4 for geometric elements appearing in bundle.
[b]Dash indicates no data.

Analysis of Michigan State Route Off-Road Accidents

The analysis conducted on this data set involved the total number of off-road accidents. Table 6 presents the salient results for the eight AID runs. The first off-road accident model (Model 13), which used only ADT as the explanatory variable, explained 30.2 percent of the variance. When the number of intersections was added (Model 14), the variance explanation increased to 42.8 percent.

Figure 6 presents the AID that used all 17 variables (Model 15) listed in Table 6. The variance explanation was increased to 66.0 percent, and 13 of the 17 variables appearing in a total of 27 classes were significant. Roadside obstacle clearance variables appeared in 6 of the 12 important splits shown in Figure 6. Curvature-related variables were found in two of the splits and ADT in 3 of the most important splits. The first split was at an ADT of 1,300. Above that, roadside obstacle effects dominated. At lower ADT values alignment was the most meaningful descriptor, although another ADT split appeared at 600 to 700 vpd. Two roadside hazard variables were important for the better alignment sections at the low ADT levels.

Five variables that were not significant in Model 15 or were viewed as redundant were eliminated, and Model 16 was run with 12 candidate variables. Ten of these were significant and the variance explanation was reduced by only 1.5 percent. Three more variables were eliminated in Model 17, leaving nine candidates. Eight of them were significant, and again 64.5 percent of the variation was explained.

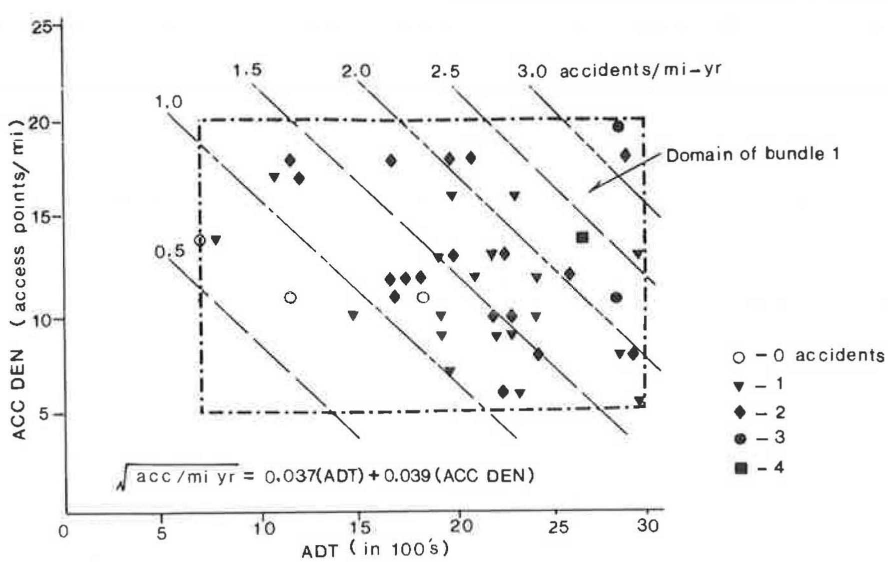

FIGURE 5 Accident prediction model for Michigan-FHWA skid file bundle Geo-1.

TABLE 6 AID Summary: Michigan State Route Data

Variable Name	Symbol	Model 13	14	15[a]	16	17	18	19[a]	20[a]	21
Dependent variable (total no. of accidents/8 mi-yr)	–	•	•	•	•	•	•	•	•	•
Flow and intersections										
ADT	ADT	•	•	•	•	•	•	•	•	•
No. of intersections on curves	Int Cu			o						
No. of intersections on tangents	Int Ta			o						
Total no. of intersections	Int Tot		•	•	•	•				•
Cross section										
Pavement width (ft)	Pv Wid			•	•	o				
Shoulder width (ft)	Sh Wid			•	•	•				
Shoulder treatment	Sh Tr			o						
Ditch offset (ft)	Dit Off			•	•	•	•			
Ditch condition	Dit Con			•	o	•				
Alignment										
Percentage of passing-sight distance restriction	PSR			•	•	•	•			
No. of curves in segment	NC			•	•					
Percentage of segment curved	PCL			•	•	•	•			
Roadside hazard (cumulative % of exposure length)										
With obstacles within 6 ft of surface	OBJ6			o						
With obstacles within 10 ft of surface	OBJ10			•	•					
With obstacles within 14 ft of surface	OBJ14			•	o	•	•			
With obstacles within 20 ft of surface	OBJ20			•						
With obstacles within 30 ft of surface	OBJ30			•	•					
Bundles										
Roadside	Rs Bun							•		
Alignment	Al Bun							•		
Overall geometric	Geo Bun								•	•
Variation explained (%)	–	30.2	42.8	66.0	64.5	64.5	63.9	63.7	62.4	62.3
No. of final groups	–	11	18	27	16	16	24	27	21	23

Note: o = variable did not appear in significant split; • = variable appeared in significant split.
[a]AID branch diagrams shown in Figures 6-8.

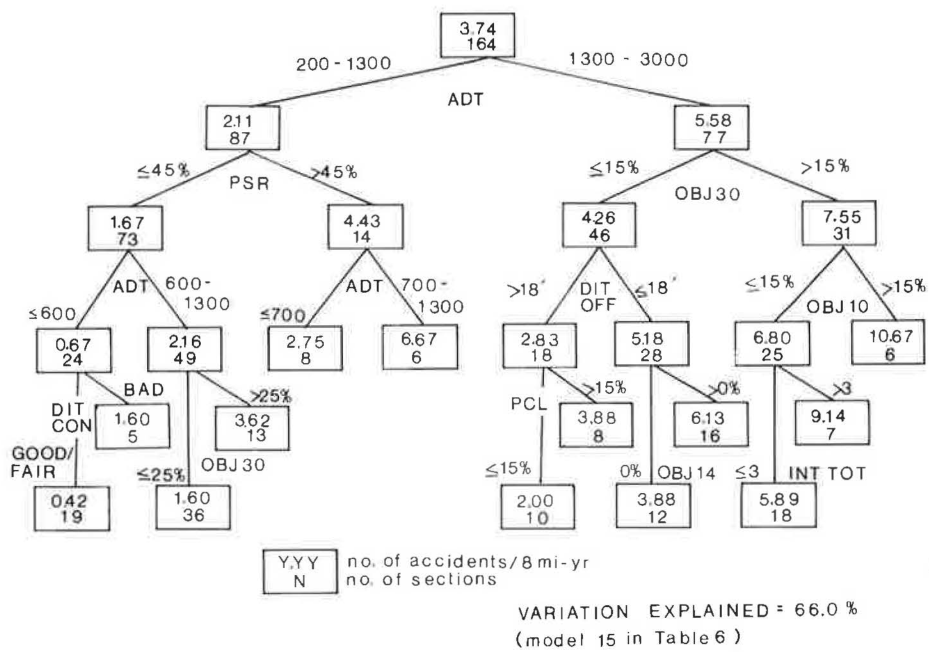

FIGURE 6 Michigan State route total off-road accident AID branch diagram using all variables.

Six factors were retained in the factor analysis. The first and second factors contained only roadside features. The third factor was an access-point density factor. Horizontal alignment was captured in the fourth factor. ADT was the only significant contributor to the fifth factor. The sixth factor was a cross-sectional design factor. The analysis showed large communalities that reflect the high power of the factors.

Tables 7 and 8 present the levels of the four variables selected for the bundles. Model 18, which used these four variables, explained 63.9 percent of the variance in 24 classes. This was a decrease of only 0.6 percent resulting from the elimination of four variables. Eight alignment bundles and eight roadside-element bundles were then formed, as shown in Tables 7 and 8. Again the bundles were numbered by generally decreasing geometric quality. Figure 7 presents the AID analysis using only ADT and the alignment and roadside-element bundles as variables (Model 19). It resulted in an explained variance of 63.7 percent. At volumes less than 1,500 vpd the alignment bundle was more important, whereas the

TABLE 7 Definition of Roadside Bundles for Michigan State Route Off-Road Accidents

Bundle Designation	OBJ14 (%)	Ditch Offset (ft)	No. of Sections
Rs-1	0	>18	19
Rs-2	0	16-18	20
Rs-3	0	8-15	15
Rs-4	>0	>18	21
Rs-5	1-5	16-18	22
Rs-6	1-5	8-15	26
Rs-7	>5	16-18	14
Rs-8	>5	8-15	27

TABLE 8 Definition of Alignment Bundles for Michigan State Route Off-Road Accidents

Bundle Designation	Percentage of Segment Curved	Percentage of Sight Restriction	No. of Sections
Al-1	0	0	19
Al-2	1-30	0	12
Al-3	0	1-30	21
Al-4	1-30	1-30	37
Al-5	0-30	>30	22
Al-6	>30	0	8
Al-7	>30	1-30	26
Al-8	>30	>30	19

roadside-element bundle was more important at the higher volumes. However, both bundles appeared in all 13 final groupings formed by this model. In the first split for each type of bundle, accident experience generally increased with decreasing bundle quality.

A single geometric bundle variable was then defined. Table 9 presents the nine categories of this bundle. Only Geo-9 is clearly made up of the worst levels of the four geometric elements. The AID analysis (Model 20) explained 62.4 percent of the variance. Figure 8 presents the AID using the nine geometric bundles and ADT. Again, larger bundle numbers were rank associated with worse accident experience. Including the number of intersections in a further analysis (Model 21) did not improve the variance explanation.

The 23 sections with from 8 to 18 accidents, more than twice the average of 3.74, were compared with the 25 segments with no accidents. The worst accident sections had the following significant (α = 0.05) relative characteristics:

- Higher ADT (2,010 versus 800 vpd),
- More intersections (2.5 versus 1.7),
- Narrower pavement (21 versus 22 ft),
- More length with passing restrictions (34 versus 15 percent),
- Worst ditch cross-section design,
- More length with roadside hazards within 14 ft of the road (15 versus 8 percent), and
- More length with roadside hazards within 20 ft of the road (30 versus 20 percent).

Of these variables, pavement width was not significant in the main AID variable analysis (Model 17) and was not included in the bundles because factor analysis showed that it was highly correlated with shoulder width, which was included in the bundling.

The classification of the extreme accident sections by the geometric bundles formulated previously is shown as follows:

Section Classification (no. of accidents/section)	Geometric Bundle 1-3	4-6	7-8	9	Total
Best (0)	12	10	3	0	25
Worst (8-18)	4	4	7	8	23

There is a clear pattern that the best sections with no accidents were highly associated with the better bundles. The worst sections with more than twice the average accident experience were more likely to be associated with worse bundles than with better bundles. This confirms the fact that, even without considering the effect of ADT or traffic conflicts, geometric characteristics have a very strong effect

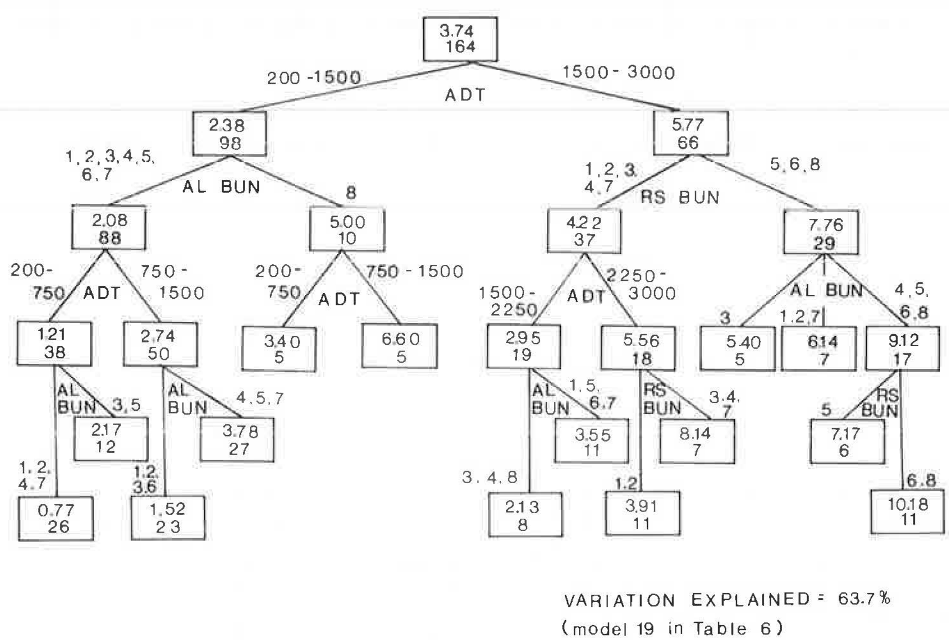

FIGURE 7 Michigan State route total off-road accident AID branch diagram using eight roadside-element and eight alignment bundles and ADT.

TABLE 9 Definition of Geometric Bundles for Michigan State Route Off-Road Accidents

Bundle Designation	OBJ14 (%)	Ditch Offset (ft)	Percentage of Segment Curved	Percentage of Sight Restriction	No. of Sections
Geo-1	0	All	0-30	0-30	43
	>0	>18	0-30	0	
Geo-2	1-5	16-18	0-30	0	15
	>0	>18	0-30	>0	
	0	All	0-30	>30	
Geo-3	0	All	>30	All	14
	0	>18	>30	>30	
Geo-4	>5	8-18	0-30	0-30	21
	1-5	8-15	0-30	0	
Geo-5	1-5	8-15	0-30	>0	14
Geo-6	>0	>18	>30	>0	10
Geo-7	1-5	16-18	>30	0-30	11
	1-5	16-18	0-30	1-30	
Geo-8	>5	8-18	0-30	>30	21
	1-5	8-15	>30	0-30	
	1-5	16-18	>30	>30	
Geo-9	>5	8-18	>30	>0	15
	1-5	8-15	>30	>30	

on off-road accident experience; more of the better-designed sections have no accidents and more of the worse-designed sections have more accidents.

A simple cross-classification model for total off-road accidents is presented in Table 10 that uses three levels of ADT and the nine geometric bundles grouped into four levels, resulting in a variance explanation of 51.4 percent. Mathematical function accident prediction models based on ADT were then developed for the nine geometric bundles grouped into the same four levels as those in the cross-classification model. These results are given in Table 11. Off-road accidents increased with ADT; the exponent increased from 0.5 to 0.9 as the bundle quality declined. These models are shown in Figure 9, which reveals that the increase in off-road accidents with ADT is more severe for sections with worse geometrics than for sections in which the geometrics were better. The off-road accident experience of the worst bundle, 9, was much worse than that of the others at lower levels of ADT.

SUMMARY AND CONCLUSIONS

In this research a national data set of total accident experience and a statewide data set of off-road accidents on two-lane rural roads were analyzed by using advanced multivariate techniques. Results of this research on different accident types support previous findings (11) that traffic volume is the most important single factor in the frequency of accidents for two-lane rural roads with flows below 3,000 vpd. In AID analysis ADT alone predicted accidents per mile-year with variance explanations of 32 and 30 percent in the two data sets, respectively.

In both data sets access-point density had an effect in predicting total accidents second only in importance to ADT. Including intersection density increased the variance explanation from 32 to 41 percent in the Michigan-FHWA skid data and from 30 to 43 percent in the Michigan off-road data. Including driveway densities increased the variance explanation to 55 percent in the Michigan-FHWA skid data. This variable was not available in the second data set. The interactive effect of ADT and access density was also important. Further, the effect of geometric characteristics was important in combination with access density.

TABLE 10 Cross-Classification Model for Michigan State Route Off-Road Accidents

ADT (vpd)	No. of Accidents per Mile-Year by Model and Geometric Bundle[a] A Geo-1-3	B Geo-4-6	C Geo-7-8	D Geo-9
≤750	0.19	0.12	0.12	0.50
750-1,500	0.22	0.30	0.84	0.84
1,500-3,000	0.51	0.66	0.95	1.32

[a]See Table 9 for geometric elements appearing in bundle.

TABLE 11 Mathematical Model for Michigan State Route Off-Road Accidents

Model	Geometric Bundle	Regression Equation (no. of accidents/mi-yr)	R^2	N
A	1-3	$0.11(ADT)^{0.49}$	0.24	72
B	4-6	$0.074(ADT)^{0.72}$	0.51	45
C	7-8	$0.061(ADT)^{0.93}$	0.62	32
D	9	$0.16^{a}(ADT)^{0.76}$	0.26	15

Note: ADT is in hundreds of vehicles (2 < ADT < 30).
[a]Not significant (alpha = 0.05).

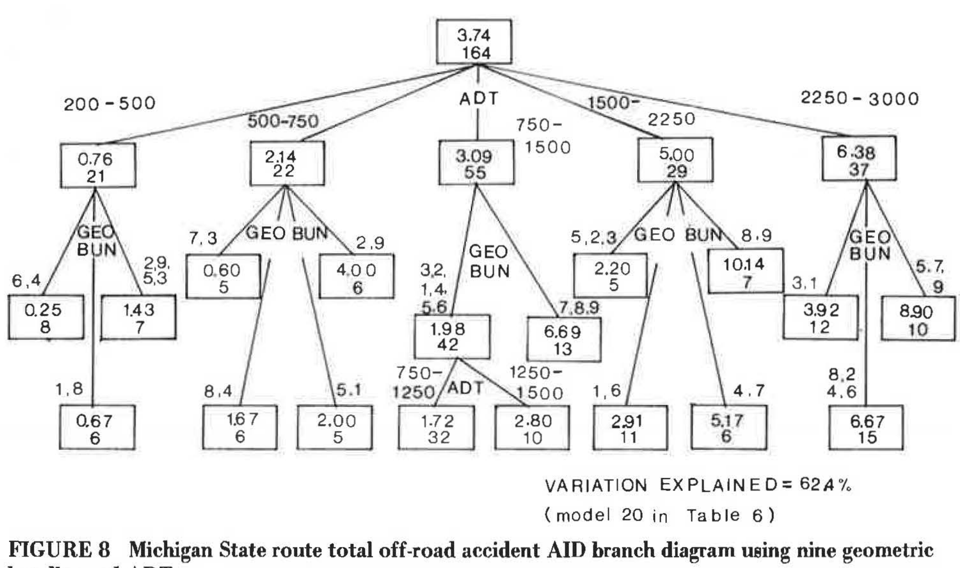

FIGURE 8 Michigan State route total off-road accident AID branch diagram using nine geometric bundles and ADT.

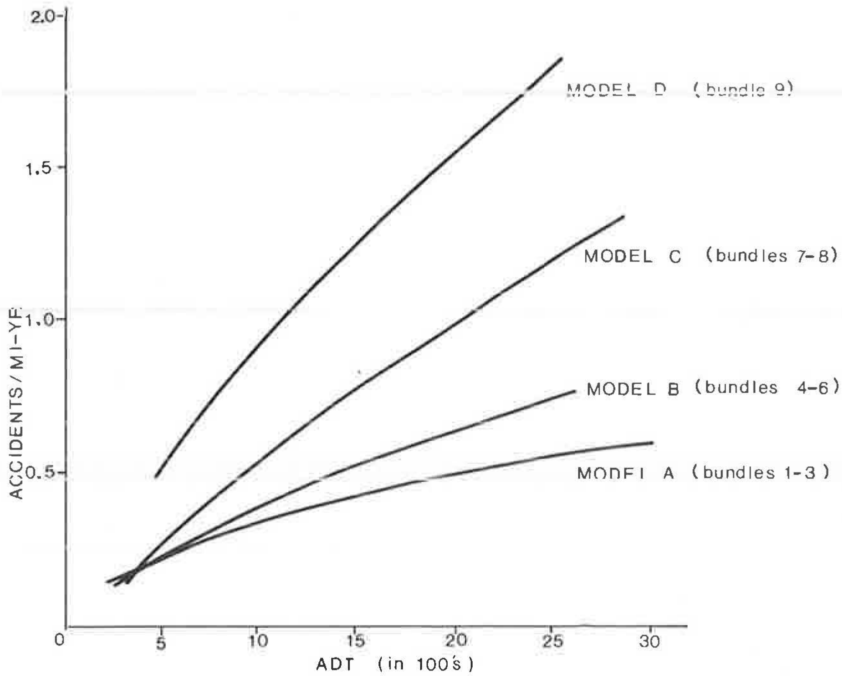

FIGURE 9 Off-road accident prediction models for Michigan State route data.

Including geometric effects by five bundles explained an additional 10 percent of the variance in the Michigan-FHWA skid data, a total of 65.4 percent. Use of all 18 variables originally explained 72.1 percent. For off-road accidents in the Michigan state route data set, nine geometric bundles increased variance explanation to 62.4 percent. Use of 17 original variables explained 66 percent of the variation.

Comparing the results of this study with those of a study that similarly examined ADT values ranging up to 13,000 vpd (3) shows that the importance of both ADT and access-point density is somewhat less for the lower ADT range. The groups of geometric elements and the variables associated with operating policies such as 85th-percentile speed contribute more to the explanation of accident variation in the moderate ADT range than they do in the higher volume range.

Simple categorical models explain 52.2 percent of the variation in total accident density in the Michigan-FHWA skid data and 51.4 percent of the off-road accident variation in the Michigan state route data set.

Illustrative continuous-function mathematical models were also developed. For the Michigan-FHWA skid data file a regression model for the bundle with the best geometric features included the effects of ADT and access density and explained 35 percent of the remaining variance in this group. Regression models developed for groups of geometric bundles from the off-road accident data file included only the effect of ADT. In these models up to 62 percent of the group variance was explained. The effects of the geometric elements were included in the stratification of the bundles. The different models for the various bundle groups show that the effects of the geometric elements are not constant over the various ranges of traffic volume but are very interactive with it.

The most important findings from this research are viewed as follows:

1. Accidents interact in such a complex way with traffic volume that using the VMER, in which ADT enters into the exposure, requires particular care in specifying a mathematical model structure. In statistical studies of most data sets, this blunts the effects of this most important variable. This study has again shown that use of an annual section length exposure measure, accidents per mile-year, permits ADT to be treated as a classification or an independent regression variable, and one can predict accident occurrence more effectively for a given data set with this measure.

2. Groups of geometric and roadside design element values, found together in clusters because of design policies and traffic use, form bundles to which accident occurrence is as responsive as it is to the individual elements of road design themselves. The safety ranking of the various bundles also gives some insight into the relative importance of individual geometric design elements.

3. Classifying sections with the best and worst accident records by their geometric bundles revealed a clear pattern. The best bundles were associated with the sections that had the lowest accident records and the worst bundles were associated with the sections that had the highest accident records in both data sets. This confirms the strong effects of geometric characteristics on accident experience even without taking ADT or traffic conflicts into consideration.

4. Comparisons of the sections with the best and worst accident records identified the variables with the greatest explanatory powers found in the models. In consideration of other findings, it is of particular interest that there is an absence of a strong independent safety effect of pavement or shoulder width for rural two-lane highways carrying fewer than 3,000 vpd.

Results of this research indicate that treating geometric and roadside elements in clusters rather than individually is worthy of consideration in the planning of safety improvements on two-lane rural roads with moderate traffic volumes.

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The authors take sole responsibility for the analyses and interpretations of these data sets provided through the courtesy of FHWA and the Michigan Department of Transportation.

Publication of this paper sponsored by Committee on Geometric Design.

Abridgment

Offtracking of the Larger Combination Commercial Vehicles

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ABSTRACT

Recent legislative enactments at all government levels have resulted in increases in trucks and tractor-trailer combinations. Section 138/415 of the Surface Transportation Assistance Act of 1982 called for a feasibility study of a national intercity truck route network for commercial vehicles up to 110 ft long. Naturally these changes in overall vehicle size will have an impact on highways and their operations, especially on roads with restrictive geometrics. To assess the ability of a vehicle to operate, one characteristic, its offtracking, must be evaluated. Several studies involving offtracking are reviewed and it is noted that there is a need for additional study of the problems associated with very long combination vehicles. The offtracking characteristics of very long vehicles, as well as those of less extreme vehicles, are described. Two methods of measuring offtracking are used, mathematical formulation and an adjustable scale model. Curves that illustrate vehicle offtracking paths were produced with the model. The formula was used to compute and compare the maximum offtracking of the vehicles studied. The results are tabulated and sample templates provided.

Because of the interest in large trucks, mostly generated by federal legislation and some state legislation that calls for the elimination or reduction of size restrictions, there is a need for some documentation of the turning characteristics of these vehicles. Some work has already been done by agencies such as the Western Highway Institute (WHI), the California Department of Transportation, and the Society of Automotive Engineers on offtracking characteristics; however, little effort has been directed at the offtracking characteristics of the new larger combination truck units with very great overall lengths (approximately 118 ft).

Insight is provided, through offtracking curves and computed maximum offtracking, as to the characteristics of these supertrucks in comparison with shorter, more conventional vehicles. A review of previous offtracking work is presented along with methods of drawing the curves and calculating maximum offtracking.

OFFTRACKING REVIEW

Offtracking is defined in many ways, but all mean the same thing; that is, offtracking is the difference in paths of the frontmost inside wheel and rearmost inside wheel of a vehicle as it negotiates a turn. Actually, whether the distance is measured between the front and rear inside wheels, outside wheels, or the center of the axles is of no consequence; it will be the same. A similar term is "trackwidth," which is the total width of the path a vehicle makes as it traverses a corner and which is measured from the frontmost outside tire path to the rearmost inside tire path. This gives an indication of the minimum pavement width necessary to accommodate the vehicle around a corner.

It will be shown that the most important factors in offtracking are the radius and degree of turn and the length and configuration of the vehicle. Of course, many factors affect the offtracking of vehicles and they cannot all be accounted for in any predictable manner. Although vehicle length and configuration and turn radius are the main determining factors, speed and superelevation of the turn can have significant effects. Indeed, if a truck combination is going fast enough, centripetal effects may reduce offtracking to zero and may produce an overall negative offtrack effect. Likewise, a slow-moving trailer on a highly superelevated curve will experience more severe offtracking than expected. Still other factors include driver expertise, condition of the truck and its loads, wind and weather, and the condition of tires and road surface. Only by recording the paths of actual vehicles can all factors be taken into account. The modeling and mathematical methods of simulating offtracking cannot account for any of these extraneous yet real influences. All this should be taken into consideration when the results presented here are viewed.

Several methods may be used to determine the amount of offtracking for a given vehicle at a given turning radius. They are

1. Observation of actual vehicles,
2. Mathematical formulation, and
3. Simulation with models.

Observing real truck combinations would be the most accurate method and would include all the minor factors affecting offtracking. Unfortunately, few agencies can afford the time and expense of acquiring all the needed vehicles and driving them through countless possible turn situations.

Finding a vehicle's maximum offtracking for a given turn radius is most easily accomplished by using a mathematical formulation. Although the exact equation is awkward to work with, nearly perfect approximations can be used with great ease and are well suited for making comparisons of different vehicles or turns. Unfortunately, the formulation gives no indication of the shape of the curves or where along the curve the maximum offtracking will occur. Also, in cases where the vehicle's rear axle passes to the inside of the turn radius center or where the vehicle does not maintain a given turn radius long enough to achieve maximum offtracking, the equations cannot be used.

Simulation with models requires considerably more (but not excessive) work than the equations and produces a much more complete representation of a vehicle offtracking pattern. It can be used for any vehicle at any turn radius (or even combination of turns), and offtracking can be measured anywhere along the curve. More detailed discussion of mathematical formulas and models will be presented after a discussion of some previous offtracking studies.

PREVIOUS WORK

Studies that use the methods just mentioned have been made on this topic for some time. Several studies that were used as a basis for this report are discussed.

On the basis of observations of actual vehicles in simulated turning situations, Leisch (1) produced a set of offtracking templates that have been used by many state highway departments. The origin and use of the templates, which consist of offtracking curves of five vehicles at various turning radii, were documented in an HRB paper (2).

The Society of Automotive Engineers (SAE) has, in its publications, provided mathematical formulations to describe offtracking (3). The general formula for a single-unit vehicle is

$$OT = \{WB^2 + [(TR^2 - WB^2)^{1/2} - HT]^2\}^{1/2} - (TR^2 - WB^2)^{1/2} + HT$$

where

- OT = offtracking [maximum for given turning radius (TR)],
- WB = wheelbase, and
- HT = front wheel trackwidth divided by 2.

Similar formulas for articulated vehicles are even more complex and unwieldy. Fortunately, WHI developed an equation that accurately approximates the SAE equation and that is uniform regardless of vehicle configuration (4). The much simpler formula, which is also discussed in an AASHTO report entitled Offtracking Considerations for Truck Tractor-Trailer Combinations (5), is

$$MOT = R - (R^2 - \sum L^2)^{1/2}$$

where

- MOT = maximum offtracking,
- R = turn radius, and
- $\sum L^2$ = sum of the squares of axle spacings.

WHI also describes and compares other methods of measuring offtracking, including the use of this formula, models, actual equipment, and graphics. Two methods, the mathematical formula and models, are used to address some turn problems, including urban street intersections.

Unfortunately, WHI did not address larger vehicles such as large doubles with two 48-ft trailers. Generally, though, the WHI publication remains very informative and was useful throughout this study.

VEHICLE OFFTRACKING STUDY

Proposals fostered by the Surface Transportation Assistance Act (STAA) of 1982 spurred several legislative initiatives in many states throughout the United States, which resulted in allowing a range of trucks, tractor-semitrailer combinations, and other combinations to operate at increased lengths and widths. In Texas, House Bill 1601 removed overall length restrictions on vehicles using semitrailer combinations while placing limits on individual trailers and semitrailers of 57 ft. However, because of other state and federal limitations, the more common trailer lengths will be 28 (and 28.5) ft and 48 ft. House Bill 1602 increased allowable widths from 96 in. to 102 in.

The work presented in this study was done at two different times. Initially templates were constructed for the use of the Texas State Department of Highways and Public Transportation (SDHPT) following a specific request. Later this work was expanded to include additional vehicles and different template forms.

As a result there are two sets of vehicles and two types of curves. The first set, as stipulated by the SDHPT, involves several tractor-semitrailer combinations with 48- or 57-ft semitrailers and double combinations with 28.5- or 48-ft trailers. These vehicles were used in making templates that follow the pattern established by Leisch and include turn radii of 45, 60, and 75 ft.

Given the new laws governing vehicle size limitations, the second set of vehicles consists of those believed to be a good representation of probable combinations. It includes tractor-semitrailers, doubles, triples, a truck-trailer combination, and buses, with wheelbases ranging from 30 ft for a bus to 108 ft for a double with 48-ft trailers. Only right-angle turns with a radius of 25 or 30 ft, such as might be encountered in an urban intersection, are included in the templates.

METHODS

Two methods are used in this paper to evaluate offtracking--mathematical formulas and models. An equation was used to estimate the maximum offtracking of the vehicles and the models were used to produce curves that show the shape of the vehicle's path.

Mathematical Formulation

As a means of comparing the effects on offtracking of different vehicle configurations and different turn radii, the mathematical formula developed by WHI is utilized. Its accuracy and ease of use make this method of evaluation a worthy tool for anyone.

Tractrix Integrator

A main element of this paper is to provide curves that reveal the effects of longer truck combinations. The instrument used to produce these curves is called a Tractrix Integrator, which is a model used to simulate actual vehicle offtracking characteristics.

It is a device that has a scaled bar supported at one end by a pointer and steadying frame. Attached at the other end below the bar is an inked wheel that makes a trail of ink as the bar is moved.

To use the Tractrix Integrator, the distance between the pointer and wheel is adjusted to a desired scale. The pointer is moved carefully over some fixed path representing the path followed by the front axle of a vehicle. As the Tractrix Integrator is moved, the rear wheel is marking the path of the rear axle of the vehicle. To model combination vehicles, the path of the tractor is modeled; then the

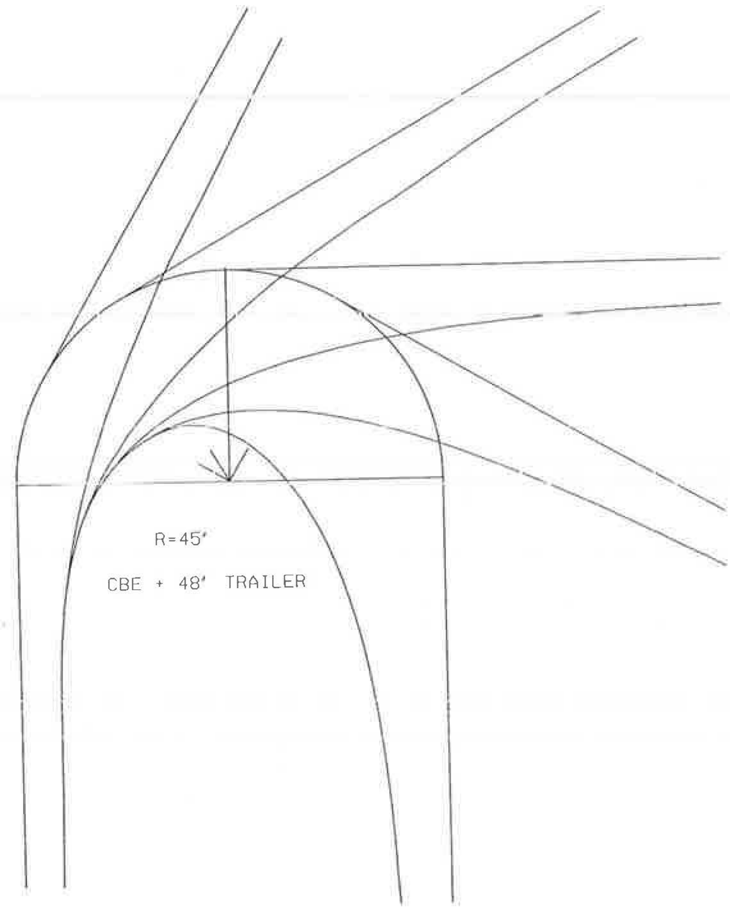

FIGURE 1 Sample offtracking template.

Tractrix Integrator is adjusted to the scale of the trailer. The pointer is pulled along the path of the rear tractor axle so that the trailer rear axle path is produced. Successive passes of the Tractrix Integrator may be made to represent any vehicle configuration.

The resulting curves closely replicate the expected paths and track widths of the outside front wheel and inside rear wheel. The advantage of this type of representation is that the maximum offtracking can be measured for any degree of turn and turn radius, as well as the amount of offtracking anywhere along the curve. Also, these curves can be used in a case where the path of the rear axle tracks inside the center of radius of curvature, a case where mathematical formulas are unusable. It should be noted that these curves are only approximations of actual vehicle paths. Some simplification is done concerning vehicle configuration; for instance, kingpin placement is always assumed to be directly over the rear axle set of the tractor. Effects of such simplification are small, however, and are not of concern in this paper.

RESULTS

The primary objective of this project was the production of offtracking templates that could be used to aid in the design or evaluation of roadway geometrics. The result is a set of 18 templates covering 14 vehicles with combinations of vehicle type and turn type that total 74. These templates (Figure 1) and descriptions of the vehicles are available from the Center for Transportation Research, University of Texas at Austin. Presented in Tables 1 and 2 is a summary of some offtracking measurements taken from those templates.

Also performed was an evaluation of maximum offtracking by using the WHI formula. This computation is easy to perform for any specific vehicle; consequently, tabulated results are not presented here. Figure 2 is included, however, to illustrate track-width characteristics of several relevant vehicles.

The trend toward excessive, and in some cases unacceptable, offtracking for large twin-trailer vehicles is evident. Poor offtracking characteristics will detract from whatever benefits are offered by those vehicles. Alternatively, triple-trailer vehicles, although offering many of the same advantages as large doubles of similar overall length, do so without the detrimental excessive offtracking.

TABLE 1 Measured Offtracking: Set 1

		Offtracking (ft) by Turn Radius (ft) and Degree					
	Vehicle Configu-	45		60		75	
No.	ration	90°	180°	90°	180°	90°	180°
1	3-S2	18.6	27.9	14.8	20.7	13.1	14.7
2	3-S2	21.7	35.3	18.5	25.7	15.7	19.7
3	3-S2	23.3	36.5+	19.9	28.5	17.1	21.7
4	3-S1-2	10.9	14.1	9.1	10.3	7.5	7.6
5	3-S2-4	-	-	-	35.5	20.5	27.3

TABLE 2 Measured Offtracking: Set 2

No.	Vehicle Configuration	Offtracking (ft) by Turn Radius (ft) and Degree 25, 90°	30, 90°
6	SU-30	–	12.2
7	SU-35	–	15.2
8	3-2	–	14.5
9	2-S1	12.7	–
10	2-S1-2	19.0	–
11	2-S1-2-2	24.1	–
12	3-S2	21.5	–
13	3-S2-2	27.0	–
14	2-S2-4	33.4	–

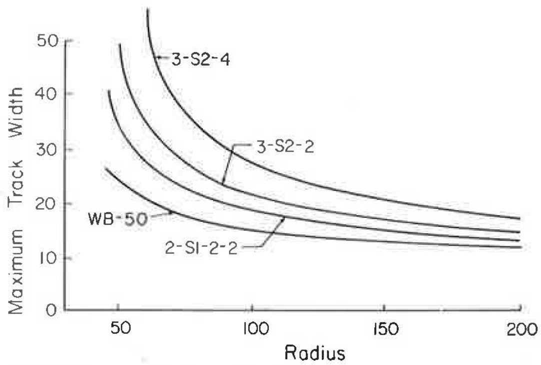

FIGURE 2 Vehicle trackwidth characteristics.

CONCLUSION

The information developed and presented in this paper is intended to further the discussion and appreciation of selected characterizations of the longer "super" combination commercial vehicles introduced in Section 138/415 of the STAA of 1982. These units and their inherent features must be assessed in order for the highway engineer to consider appropriate modifications to currently accepted highway geometric design policies and procedures. The highway engineering profession must remain abreast of emerging trends such as these vehicle units and their effects in order to provide effective guidance to elected or administrative officials as well as dialogue with the motor carrier industry. In this manner, they are better able to provide constructive judgments surrounding the benefits and costs to a national asset--the collective national highway infrastructure.

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Publication of this paper sponsored by Committee on Operational Effects of Geometrics.

A Quick-Response Technique for Impact Assessment of Highway Improvement Projects

JASON C. YU and LESLIE M. G. PANG

ABSTRACT

To help master the complexity involved in the evaluation of highway improvement projects, this study was intended to develop a procedural framework for identifying a general set of significant impact measures and to further suggest a quick-response technique for assessing potential impacts of project alternatives. The impact identification procedure is in three stages: search, screening, and consolidation. The project impacts are assessed by consideration of the level, scope, and number of potential impacts and are rated on a numerical scale employing a linear utility function. Although many highway impact evaluation methods have been developed in the past, this study has suggested a quick and inexpensive tool for preliminary project evaluation that requires little detailed data and reasonable staff time yet should provide adequate estimates of whether further planning and development of a project or projects are worthwhile.

Effective evaluation of urban highway projects demands that the impacts of improvement projects on the entire community, as well as on the system users, be fully considered and analyzed. However, this is not a simple task because of the multiplicity of impacts and the difficulty in measuring their levels. In order to cope with the complexity, effective tools for the systematic identification and assessment of significant impact measures are needed to set priorities for alternative highway projects.

Impact measures serve to help indicate to what degree given highway projects satisfy community and areawide goals and objectives. The selection criteria for impact measures are many; directness, adequacy, simplicity, sensitivity, and so on, all have a significant bearing on their choice for project evaluation. Thus, the complexity of the problem stems not only from its magnitude, but also from the diversity of its parts. The assessment of impact measures is also a difficult task because it can take a considerable amount of time and manpower. Because of the financial limitations placed on local highway agencies today, it has become necessary for many of them to look closely at quick-response techniques. Many past studies have dealt with highway planning methods for cost-benefit analysis and for alternatives analysis. However, these methods have usually been too complex and data-intensive to be used for actual application. In the sketch planning, only rough estimates are required to determine the relative project feasibility. Therefore, research is needed to develop a technical, yet practical, tool for use in the preliminary evaluation and priority ranking of highway projects.

The main purpose of this study was to develop a quick, efficient, and inexpensive approach for highway project evaluation and decision making in a typical urban environment. The approach was developed with the following specific objectives in mind:

1. To formulate a systematic procedure by which impact measures can be identified and
2. To develop a useful concept to easily define, assess, and rate the potential effects of alternative projects.

The quick-response technique developed is expected to be useful to highway agencies that are asked to do "quick and dirty" project feasibility studies that consider the interests of all parties involved.

FRAMEWORK FOR IMPACT IDENTIFICATION

The selection of impact measures has been the object of numerous past studies, but the processes suggested are arbitrary. A step-by-step procedure is needed to formalize the selection process.

Figure 1 shows the conceptual framework suggested for the development of the impact measures for highway project evaluation. As shown, there are basically three stages in the procedure: search, screening, and consolidation. In brief, the process involves establishing a broad preliminary list of impact measures through the quantification of goals and objectives and the evaluation of typical system effects by projects. Next, unnecessary impact measures are eliminated from the list through the use of a set of objective screening criteria. The remaining impact measures are consolidated so that there is a manageable number of measures for evaluating alternative projects.

Stage One: Search

As indicated earlier, impact measures can be defined as explicit terms used in determining to what degree goals and objectives are satisfied. They serve as the basis for comparing projects for obtaining maximum benefits from the highway investment. With these definitions, prospective impacts are searched through a systematic process involving two elements. The first element involves a so-called "top-down" process. The broad, implicit goals are identified, then more explicit objectives are stated, and finally impact measures are selected to reflect the extent to which goals are achieved. These goals can be achieved by a wide variety of highway improvement options. For each goal a number of objectives can be

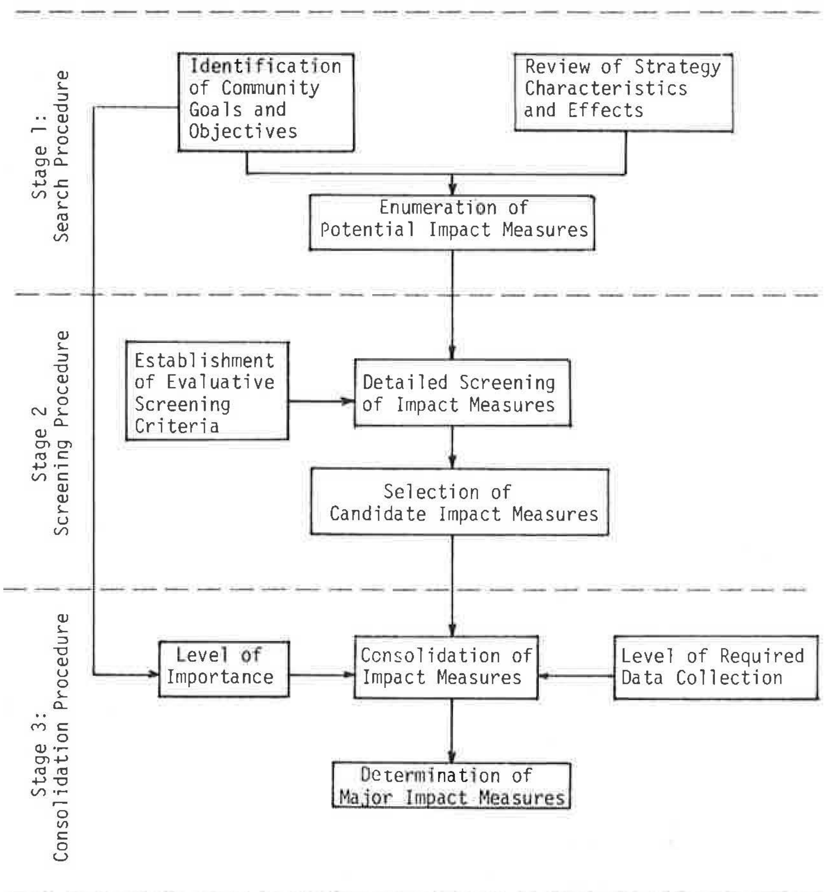

FIGURE 1 Conceptual framework for determining major impact measures.

developed that provide a more quantifiable measure of goal attainment. From these objectives, candidate impact measures are established by developing working or operational definitions in order to test or indicate the degree to which the objectives are attained. The second element is to examine the potential projects and attempt to assess what impacts would occur if the projects were implemented. These impacts are then translated into impact measures by establishing guidelines for the measurement of the magnitude of the project impact.

Stage Two: Screening

The screening procedure involves eliminating any potential impact measures that are not appropriate for evaluative purposes. The general criteria used to screen the measures include the following:

1. The measures should have a clear and specific relationship to community goals and objectives,
2. Measuring an effect that is adequately measured by other impacts should be avoided,
3. The measures should be sensitive to real changes in objective attainment,
4. The measures should be formulated at the appropriate level of detail for the analysis, and
5. The measures should be understandable and simple in interpretation from the view of citizens and nontechnical decision makers.

If any potential impact measure does not meet the preceding requirements, it is eliminated from consideration. The result of this stage is a set of selected impact measures that are feasible and desirable for the evaluation of alternative projects.

Stage Three: Consolidation

Stage 3 is devised to consolidate numerous candidate impact measures identified in Stage 2, so that a manageable number of measures can be obtained. The approach is to consider the level of importance combined with the level of data collection effort required for analyzing individual impacts. The level of importance of an impact measure indicates its degree of usefulness or even necessity in the analysis procedure. Some measures are absolutely necessary because they alone measure the attainment of a particular objective. Others are not so important because other more descriptive measures are available. The level of data collection effort required in the assessment of an impact defines the potential time and manpower involved in field measurement or estimation. Even if an impact is considered important, there may be situations in which the prohibitive cost of obtaining the impact data will prevent their use.

By addressing these two considerations, the set of previously selected impact measures are reevaluated and recombined to form a final set of representative measures. Only measures considered highly important and requiring minimal data-collection effort are desirable for project evaluation. These considerations are necessary because many transportation agencies are often restricted by finances, manpower, and time constraints.

DEFINITIONS OF IMPACT MEASURES

The major impact measures for the evaluation of highway projects generally include benefit/cost ratio, quality of highway service, energy conservation, and public safety, economic, environmental, social, financial, and land use impacts. These measures are selected through the use of the three-stage process discussed and reflect those considered significant for use in typical urban areas.

The selection of highway improvement projects is a process involving objective as well as subjective factors in an effort to provide the community with a plan having the greatest overall net benefit. For the trade-off analysis it is necessary to categorize project impact measures as objective and subjective.

Objective Impact Measures

The objective impact measures include project costs and benefits that are measurable in dollar terms. The costs should be considered over the life span of the entire project, whereas the benefits are those measurable savings received directly by the user. Typical user benefits are savings in travel time and vehicle operating and maintenance costs. The reason for considering only user's benefits as the objective impacts is that highway projects are usually initiated for the purpose of improving travel service.

Highway project costs typically consist of capital, maintenance, operating, and administrative costs. The capital costs of a project include land acquisition, engineering, construction, traffic control devices, grading and drainage, aesthetic treatments, temporary measures for traffic rerouting, and special requirements unique to the project. Costs will vary with the type of facility required, the scope of the project, and the specific characteristics of the local area through which the facility is constructed. Maintenance costs include those associated with routine periodic upkeep and repairs to the physical project facilities. Operating costs include those that are necessary for the operation of the system facility. Administrative costs are those associated with the management of the affairs connected with the project.

In most highway benefit-cost analyses, the costs and benefits are expressed on an annual basis. Therefore, the benefit/cost ratio is the equivalent uniform annual benefits divided by the equivalent uniform annual costs. The method of benefit-cost analysis is well documented in the literature and has been widely used by highway engineers and planners.

Subjective Impact Measures

Many project impacts must be evaluated primarily in a subjective manner and in terms of the overall community values and goals. Although some subjective impacts can be quantified, they cannot be expressed directly in monetary terms. They are the general class of indirect effects and consequences induced by or resulting from a highway improvement project. However, they cannot be called secondary consequences because they are a primary result of the project implementation.

Subjective impacts affect the whole community where a highway project is being considered. The following are typical subjective impacts used in evaluating alternative highway projects:

1. Quality of highway service: impacts to the highway user, such as travel comfort and convenience, service to the disadvantaged, and responsiveness to transportation needs;
2. Energy conservation: the effects of conserving fuel resources as part of the community desire to satisfy the local, regional, or national energy goals and policies;
3. Economic impacts: direct impacts on the economy of the area affected by the project implementation, such as the number of business relocations, the number of employment opportunities, the impact on employee productivity, and changes to the tax base;
4. Environmental impacts: effects on air quality, noise levels, aesthetics, and on environmentally sensitive areas;
5. Social impacts: displacement of residences and community facilities, changes in neighborhood stability and cohesion, and impacts on recreational, educational, and other facilities;
6. Financial impacts: impacts on budget and equity of financial burden; and
7. Land use impacts: effects on land accessibility, intensity of land use, and changes in land use patterns.

A TECHNIQUE FOR IMPACT ASSESSMENT

In providing a quick-response technique, a rating scheme is suggested to reflect the magnitude of various impacts resulting from alternative highway projects under a given situation. A scale of -3, -2, -1, 0, +1, +2, and +3 may be used to indicate the estimated magnitude of alternative effects. The negative and positive values of the scale reflect the unfavorable and favorable impacts, respectively. The absolute values of 0, 1, 2, and 3 indicate the level of impact produced, that is, negligible, small, medium, and large. In this case, a linear utility function is assumed to simplify the problem of impact assessment because the actual impact patterns are difficult and expensive to determine. The rating relies on the ability of technical staff in estimating the potential effects produced by one project compared with other projects under consideration. The scaled value only represents the relative magnitude of impacts rather than their absolute value and thus can readily be derived from professional judgment and experience without extensive data collection and analysis effort. Therefore, a substantial amount of time and manpower can be saved.

For practical purposes, it is necessary to define the magnitudes of individual impacts to better guide the highway agency in its evaluation effort and to provide some degree of consistency in impact assessment. The definitions of impacts broken down to the subimpact level are derived and are given in Table 1. It should be pointed out that these definitions are only guidelines and not a strict mandatory set of measures. The reason for this is obvious; impacts that are characteristically difficult to measure and assess objectively are involved. However, the suggested guidelines represent a framework to establish some concrete appraisal of impact for highway project evaluation.

The approach utilized in the rating scheme involves considering three critical aspects of the project impact: first, the scope of the impact, that is, the percentage of community population affected by the project; second, the level of the individual subimpacts that constitute the entire impact class; and third, the total number of significantly favorable and/or unfavorable subimpacts for a given impact class.

TABLE 1 Definitions of Significant Subimpacts

Impact	Subimpact	Definition: Significantly Favorable	Definition: Significantly Unfavorable
Objective: benefit/cost ratio	None	Not applicable	Not applicable
Subjective			
Quality of service	Passenger comfort and convenience	Outstanding level of comfort and convenience offered	Comfort and convenience totally lacking
	Reliability	Project characteristics indicate a high potential for greater service dependability than that for existing systems	Project exhibits relatively poor service dependability characteristics
	Service to disadvantaged	Outstanding level of service provided	No service to disadvantaged
	Service to disadvantaged	Major increase in number of disadvantaged served (at least 5 percent)	Any decrease in number of disadvantaged due to the project
	Responsiveness to transportation needs	High flexibility for meeting future requirements	Rigid project plan
Energy conservation	Energy reduction areawide	Substantial reduction in energy consumption (at least 5 percent reduction areawide)	Minimal reduction in energy consumption areawide
	Community desire to conserve energy	Significant contribution toward attainment of such goals	Counter to achievement of such goals
Public safety	Accident behavior	Any considerable decrease in accident frequency, rate, and/or severity (50 percent or more)	Any increase in accident frequency rate and/or severity
	Personal security	Any significant decrease in crime rate (50 percent or more)	Any increase in crime rate
Economic	Business relocation	Some business relocation into the area	Any number of business relocations out of the area
	Dollar sales	More than 15 percent greater than the last 3-yr average	Less than 5 percent smaller than the last 3-yr average
	Employment	Employment increase rate more than population growth rate or unemployment rate decrease	Employment increase rate less than population growth rate or unemployment rate increase
	Employment productivity	Sale/cost ratio 10 percent greater than last 3-yr average	Less than 5 percent less than last 3-yr average
	Tourism revenue	More than 10 percent greater than last 3-yr average	Less than 5 percent less than last 3-yr average
	Changes in tax base	Significant increase in tax revenues	Any tax loss related to project
Environmental	Aesthetics	Results in enhancement of natural surroundings	Project aesthetics considered detrimental to community
	Vehicle emissions	Any reduction in pollutant level	Exceeding allowable vehicle emission standards
	Noise levels	Any reduction in noise level	Exceeding allowable noise standards
	Impacts on natural historic and/or archaeological site	Project enhances site attractiveness	Any detrimental effect on these sites
Social	Residences displaced	No residences displaced	Greater than 10 percent of residences displaced in proximity of project
	Effect on neighborhood stability and cohesion	Project provides significant help in reinforcing community	Project is major disruptive force in community
	Responsiveness to community needs (such as impacts on religion, health, education, recreational, emergency, and other services)	Project promotes use of these services	Displacement of any service facilities or discouragement in use of these facilities
Financial	Impact on budget	Minimal effect on budget allocation	Causes an unbalanced budget allocation
	Equity of financial burden	Fair and balanced distribution of project benefits and costs	Uneven distribution of benefits and costs resulting in an unfair burden for one sector of population
Land Use	Accessibility	Project provides high level of accessibility to desired land areas	Project minimizes or hampers accessibility to desired land areas
	Intensity of land use	Project promotes intensification of selected land areas	Project obstructs intensification of selected land areas
	Changes in land use pattern	Project actively encourages desired changes in land use patterns	Project prevents desired changes in land use patterns

Suggested standards for impact rating are presented in Table 2. An impact assigned a high (+3) rating would (a) affect a large proportion of the total given population, (b) encompass individual subimpacts categorized as significantly favorable, and (c) include a preponderance of favorable subimpacts. On the opposite end of the spectrum, an impact assigned a -3 rating would (a) affect a relatively large percentage of the population, (b) encompass some individual subimpacts of significantly unfavorable magnitude, and (c) include a majority of unfavorable subimpacts. In the middle of the spectrum, that is, a rating of 0, a very small portion of the population would be affected, the magnitude of the subimpacts would be generally insignificant, and/or either most of the subimpacts would be insignificant or there would be a balance between significantly favorable and unfavorable subimpacts. These three concepts are shown in Figure 2.

The procedure suggested for use in the rating assignment is as follows:

1. For each impact class, by using Table 1, determine whether each of its subimpacts falls in the categories defined as either a significantly favorable or unfavorable subimpact. Total the number of such significant subimpacts for the impact class.
2. Estimate the scope of the impact, that is, the proportion of the population (given as a percentage) affected by the project impacts.
3. Given the number of significantly favorable or unfavorable subimpacts and the scope of the impact, use Table 2 to determine the corresponding impact rating.

TABLE 2 Impact Rating Levels

Impact	Impact Rating	Description of Rating	Scope of Subimpact
Objective: benefit/cost ratio	+3	Ratio value in high range of all projects considered, ratio greater than 1	A major portion of population (at least 25 percent)
	+2	Ratio value in middle range of all projects considered, ratio greater than 1	A moderate percentage of population (at least 15 percent)
	+1	Ratio value in low range of all projects considered; ratio greater than 1	A minor portion of population (at least 5 percent)
	0	Ratio value equal to 1	Negligible portion of population (less than 5 percent)
	−1, −2, −3	Not applicable because project is not economically feasible	Not applicable
Subjective[a]			
Energy conservation Public safety Financial (two subimpacts)	+3	All of the subimpacts are favorable	A major portion of population (at least 25 percent)
	+2	Same as above	A moderate percentage of population (at least 15 percent)
	+1	Most of subimpacts are favorable with none unfavorable (at least 1 favorable, 0 unfavorable)	A minor portion of population (at least 5 percent)
	0	No significant subimpacts or a balanced number of favorable and unfavorable subimpacts	Negligible proportion of population (less than 5 percent)
	−1	Most of subimpacts are unfavorable (at least 1 unfavorable, 0 favorable)	A minor portion of population (at least 5 percent)
	−2	All of subimpacts are unfavorable with none favorable (2 unfavorable, 0 favorable)	A moderate percentage of population (at least 15 percent)
	−3	Same as above	A major portion of population (at least 25 percent)
Quality of service Economic (five subimpacts)	+3	Majority of subimpacts are favorable with none unfavorable (at least 4 favorable, 0 unfavorable)	A major portion of population (at least 25 percent)
	+2	Many of subimpacts are favorable with none unfavorable (at least 2 favorable, 0 unfavorable)	A moderate percentage of population (at least 15 percent)
	+1	Favorable subimpacts outnumber unfavorable ones	A minor portion of population (at least 5 percent)
	0	No significant subimpacts or a balanced number of favorable and unfavorable subimpacts	Negligible portion of population (less than 5 percent)
	−1	Unfavorable subimpacts outnumber favorable ones	A minor portion of population (at least 5 percent)
	−2	Many of subimpacts are unfavorable with none favorable (at least 2 unfavorable, 0 favorable)	A moderate percentage of population (at least 15 percent)
	−3	Majority of subimpacts are unfavorable with none favorable (at least 4 unfavorable, 0 favorable)	A major portion of population (at least 25 percent)
Environmental (four subimpacts)	+3	Majority of the subimpacts are favorable with none unfavorable (at least 3 favorable, 0 unfavorable)	A major portion of population (at least 25 percent)
	+2	At least half of subimpacts are favorable with none unfavorable (at least 2 favorable, 0 unfavorable)	A moderate percentage of population (at least 15 percent)
	+1	Favorable subimpacts outnumber unfavorable subimpacts	A minor portion of population (at least 5 percent)
	0	No significant subimpacts or a balanced number of favorable and unfavorable subimpacts	Negligible proportion of population (less than 5 percent)
	−1	Unfavorable subimpacts outnumber favorable subimpacts	A minor portion of population (at least 5 percent)
	−2	At least half of subimpacts are unfavorable with none favorable (at least 2 unfavorable, 0 favorable)	A moderate percentage of population (at least 15 percent)
	−3	Majority of subimpacts are unfavorable with none favorable (at least 3 unfavorable, 0 favorable)	A major portion of population (at least 25 percent)
Social Land use (three subimpacts)	+3	All subimpacts are favorable with none unfavorable (3 favorable, 0 unfavorable)	A major portion of population (at least 25 percent)
	+2	Majority of subimpacts are favorable with none unfavorable (at least 2 favorable, 0 unfavorable)	A moderate percentage of population (at least 15 percent)
	+1	Favorable subimpacts outnumber unfavorable subimpacts	A minor portion of population (at least 5 percent)
	0	No significant subimpacts or a balanced number of favorable and unfavorable subimpacts	Negligible portion of population (less than 5 percent)
	−1	Unfavorable subimpacts outnumber favorable subimpacts	A minor portion of population (at least 5 percent)
	−2	Majority of subimpacts are unfavorable with none favorable (at least 2 unfavorable, 0 favorable)	A moderate percentage of population (at least 15 percent)
	−3	All subimpacts are unfavorable with none favorable (3 unfavorable, 0 favorable)	A major portion of population (at least 25 percent)

[a]No. of significant subimpacts.

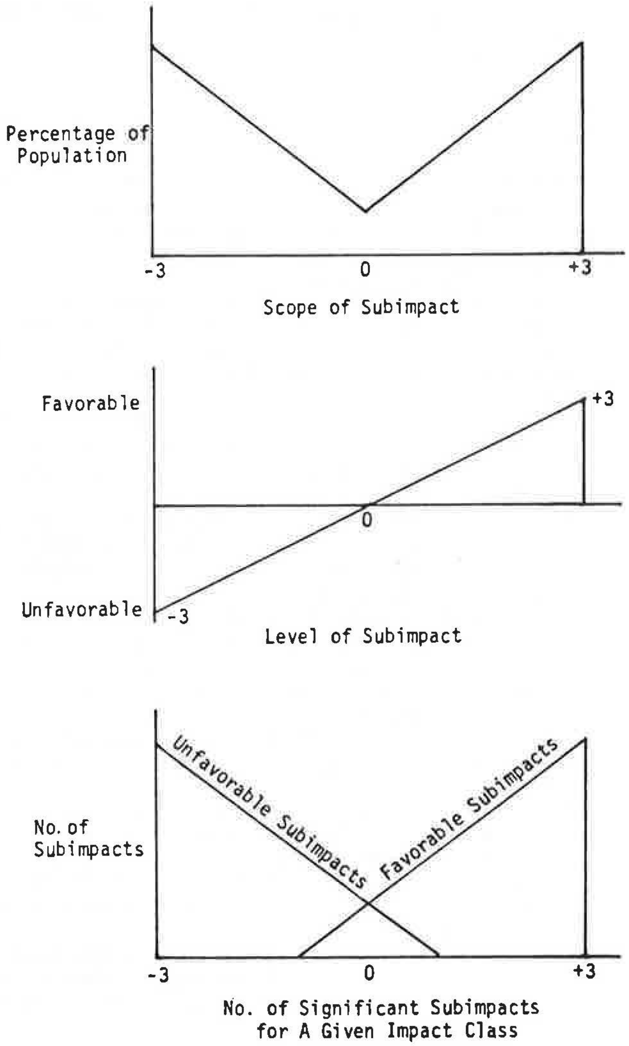

FIGURE 2 Concepts of the rating scheme for subimpacts.

For the situation in which the impact rating corresponds to the scope of impact, the rating with the lowest absolute value would be assigned to the impact. For example, if the impact rating is +3 based on the number of subimpacts and the rating is +1 based on the scope, an overall impact rating of +1 would be assigned. Conversely, if the two ratings were -2 and -1, the assigned impact rating would be that of the lowest absolute value, or -1. The reason for this rule is that the minimum of rating elements, number of significant subimpacts, and scope takes on the extent of the entire impact. For example, if an impact is significantly unfavorable yet affects only a very small percentage of the population, it is logical that the rating is relatively small (-1 or 0) because the scope of the negative impact is so insignificant. Also, if an impact is balanced in terms of favorable and unfavorable impacts and affects a large segment of the population, again the overall rating for the impact would be low (zero). A neutral impact affecting a significant number of people would still be considered neutral in its overall effect.

If a given community elected to add a greater number of subimpacts than those shown in Table 2, Table 3 may be applied in which the number of significant subimpacts is in terms of percentage of the number of subimpacts rather than in absolute numbers.

TABLE 3 Generalized Impact Rating Levels

Impact Rating	No. of Significant Subimpacts	Scope of Subimpact
+3	Majority of subimpacts are favorable (80 percent of subimpacts)	A major portion of population (e.g., at least 25 percent)
+2	Many of subimpacts are favorable (70 percent of subimpacts)	A moderate percentage of population (at least 15 percent)
+1	Some subimpacts are favorable (60 percent of subimpacts)	A minor portion of population (at least 5 percent)
0	No significant subimpacts or a balanced number of favorable and unfavorable subimpacts	Negligible portion of population (less than 5 percent)
−1	Some subimpacts are unfavorable (60 percent subimpacts)	A minor portion of population (at least 5 percent)
−2	Many subimpacts are unfavorable (70 percent of subimpacts)	A moderate percentage of population (at least 15 percent)
−3	Majority of subimpacts are unfavorable (80 percent of subimpacts)	A major portion of population (e.g., at least 25 percent)

Note: This table is to be used for six or more subimpacts per impact class.

A DECISION-MAKING MECHANISM

One of the main difficulties of highway project planning is that planners cannot usually make simultaneous gains on all goals and objectives. Different interests often compete directly with one another--improvements on one objective often require reductions on others. In order to decide what constitutes the best mix of benefits and losses to the various goals and objectives, the trade-off analysis is needed to arrive at a judgment of the overall worth of alternative plans. Therefore, a systematic evaluation procedure is necessary to help planners think through and document these types of trade-offs.

After the magnitude of all project impacts, objective and subjective, has been rated based on the standard rating scheme discussed previously, it is then imperative to determine the relative importance of these impacts in order to reach an overall determination of a project's worth. The weighing technique employed to establish a priori preference has been widely adopted in the public decision process. The technique is a method of attaching weights to the different impacts considered. It is expected that the value judgment of a decision-making body should accurately reflect the priority of citizens' concerns. A simple yet satisfactory approach for preliminary ranking of alternative projects is to calculate for each the sum of the values corresponding to ratings multiplied by the weights for the respective impacts. All projects can then be ranked in accordance with their total scores, with higher numbers indicating more attractive alternatives (1,2).

In order to aid highway agencies in ranking alternative improvement projects, a further step is necessary to have an effective and practical mechanism for their decision making. The scoring technique for project evaluation and decision making has been the subject of numerous past studies. For example, Thomas and Schofer (3) suggested a cost-effectiveness approach where the effectiveness is some measure of objective attainment. Hill (4) proposed a goal-achievement analysis where the alternative plans are examined in terms of the entire set of objectives. Goals are defined operationally and goal achievement is measured in units that are relevant to the particular objectives. The relative effectiveness of alternative plans in advancing the set of desired objectives is determined by applying a weighting system to the objectives and to the subgroups, sectors, locations, and activities affected. Schimpeler and Grecco (5) devised an effectiveness matrix in which the evaluation of alternative systems is based on a comprehensive, weighted hierarchy of community development criteria. The basic decision model relates to the evaluation of alternative design concepts by a single group of professional planners on the basis of a single set of weighted community decision criteria. Schlager (6) developed a rank-based expected value method of plan evaluation. Yu and Pang (7,8) have recently developed a computer-based cost-effectiveness model capable of incorporating both tangible and intangible impacts into a single mathematical function to produce the composite measure of effectiveness value for priority ranking of plans.

CONCLUSIONS

This study has suggested a procedural framework that identifies a general set of impact measures for the evaluation of highway improvement projects. Further, a simplified approach was developed to make "quick and dirty" impact assessment of project alternatives. Highway project evaluation is a challenging task because various projects have many impact components and the problem of estimating impacts has many dimensions. The quantity, intensity, scope, and acceptance of major impacts all bear on project ranking and decisions. The complexity of the problem stems not only from its magnitude but from the diversity of its parts. Detailed impact studies often require high cost and manpower and are time consuming as well. The suggested technique for impact assessment entails only the relative extent of potential impacts of highway improvement alternatives instead of the absolute impact value. Therefore, this technique can efficiently assist the planner in highway sketch planning to determine whether further planning and development efforts are worthwhile for any given project.

ACKNOWLEDGMENTS

This study was performed as part of a larger research project supported by UMTA, U.S. Department of Transportation. The research and training grant was awarded to the Department of Civil Engineering, University of Utah. The practical input from the Transportation Planning Division of the Utah Department of Transportation is greatly appreciated.

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Publication of this paper sponsored by Committee on Methodology for Evaluating Highway Improvements.