1195

Transportation Research Record

Geometric Design and Operational Effects

Transportation Research Board
National Research Council
Washington, D.C. 1988

Transportation Research Record 1195
Price: $21.50

mode
1 highway transportation

subject areas
21 facilities design
54 operations and traffic control

Transportation Research Board publications are available by ordering directly from TRB. They may also be obtained on a regular basis through organizational or individual affiliation with TRB; affiliates or library subscribers are eligible for substantial discounts. For further information, write to the Transportation Research Board, National Research Council, 2101 Constitution Avenue, N.W., Washington, D.C. 20418.

Printed in the United States of America

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

Geometric design and operational effects.

p. cm.—(Transportation research record, 0361-1981; 1195)
ISBN 0-309-04760-9
1. Roads—Design and construction. 2. Traffic engineering.
I. National Research Council (U.S.). Transportation Research Board.
II. Series.
TE7.H5 no. 1195
[TE175]
388 s—dc20
[625.7'25] 89-13128
CIP

Sponsorship of Transportation Research Record 1195

GROUP 2—DESIGN AND CONSTRUCTION OF TRANSPORTATION FACILITIES
Chairman: David S. Gedney, Harland Bartholomew & Associates

General Design Section
Chairman: Jarvis D. Michie, Dynatech Engineering, Inc.

Committee on Geometric Design
Chairman: John C. Glennon, John C. Glennon Chartered
Secretary: George B. Pilkington II, Federal Highway Administration
James K. Cable, Paul D. Cribbins, Jerry L. Graham, C. William Gray, Dennis A. Grylicki, John E. Haviland, Milton L. Johnson, Frank J. Koepke, Ruediger Lamm, Joel P. Leisch, John M. Mason, Jr., Ted K. McCourry, John R. McLean, Thomas E. Meyer, Geoffrey M. Nairn, Jr., Timothy R. Neuman, Brooks O. Nichols, Abishai Polus, James F. Roberts, Bernard H. Rottinghaus, Seppo I. Sillan, James A. Thompson, Jess E. Truby, Jr., Ross J. Walker, Keith M. Wolhuter

GROUP 3—OPERATION, SAFETY, AND MAINTENANCE OF TRANSPORTATION FACILITIES
Chairman: James I. Taylor, University of Notre Dame

Facilities and Operations Section

Committee on Operational Effects of Geometrics
Chairman: Jerome W. Hall, University of New Mexico
Secretary: Douglas W. Harwood, Midwest Research Institute
Jack W. Anderson, Myong-Soon Chang, George A. Dale, Ronald W. Eck, Daniel B. Fambro, Richard P. Kramer, Robert P. Morris, Timothy R. Neuman, Ronald C. Pfefer, Abishai Polus, Ramey O. Rogness, Rudolph Kenneth Shearin, Jr., Justin True, Daniel S. Turner, Stephen N. Van Winkle, Walter E. Witt, Charles V. Zegeer

Committee on Methodology for Evaluating Highway Improvements
Chairman: Forrest M. Council, University of North Carolina
Secretary: William T. Baker, Federal Highway Administration
William D. Berg, Brian L. Bowman, Eugene D. Brenning, Charles Philip Brinkman, Myong-Soon Chang, William D. Glauz, Alfred-Shalom Hakkert, Fred R. Hanscom, Ezra Hauer, David L. Helman, Warren E. Hughes, Ruediger Lamm, Sidney J. Louick, King K. Mark, Thomas L. Maleck, Ernst Meyer, Bhagwant N. Persaud, Jerry G. Pigman, Ramey O. Rogness, Roy W. Taylor, Harold T. Thompson, James M. Witkowski, Paul H. Wright

David K. Witheford, Transportation Research Board staff

Sponsorship is indicated by a footnote at the end of each paper. The organizational units, officers, and members are as of December 31, 1987.

Transportation Research Record 1195

Contents

Computer Simulation Study of the Operational Effects of Two-Way Left-Turn Lanes on Urban Four-Lane Roadways

JOHN L. BALLARD AND PATRICK T. MCCOY

The two-way left-turn lane (TWLTL) is commonly used to solve safety and operational problems on four-lane undivided roadways that result from conflicts between through-traffic and drivers making mid-block left turns. Numerous studies of the safety effects of TWLTL medians have been conducted, but studies of their operational effects have been limited. Consequently, attempts to develop guidelines for the use of the TWLTL have focused on safety benefits and have not adequately accounted for the operational savings it provides. In this study, computer simulation was used to determine the operational effects of TWLTL medians on urban four-lane roadways. Multiple regression analyses of the results of the simulation runs were conducted to determine the relationships between the operational effects of TWLTL medians and prevailing roadway and traffic conditions. Regression equations were developed for predicting the reduction in stops and delay provided by TWLTL medians. This paper includes a description of the simulation model as well as the procedures, findings, and conclusions of the study.

The two-way left-turn lane (TWLTL) is commonly used to solve safety and operational problems on four-lane undivided roadways that result from conflicts between through-traffic and drivers making mid-block left turns. As illustrated in Figure 1, left turns from a four-lane undivided roadway are made from through-traffic lanes, causing through vehicles in these lanes to change lanes or be delayed. But on a roadway with a TWLTL the deceleration and storage of left-turn vehicles are removed from the through-lanes as illustrated in Figure 2. Thus, conflicts between through- and left-turn traffic are eliminated, and through-vehicles can pass by left-turn vehicles without changing lanes and without delay.

Although the potential safety and operational effects of the TWLTL are recognized by highway engineers, there are no generally accepted guidelines that define the circumstances under which the costs of providing TWLTL medians are justified. Numerous before-and-after studies, which provide measures of the safety effectiveness of the TWLTL, have been conducted. However, empirical data pertinent to the assessment of the operational effectiveness of the TWLTL are limited. Therefore, previous attempts to develop guidelines for the use of the TWLTL have focused on the safety benefits and have not adequately accounted for the operational effectiveness of the TWLTL. Consequently, there are no definitive guidelines for the cost-effective use of TWLTL medians.

The overall objective of the research was to develop guidelines for the use of TWLTL medians on urban four-lane roadways that account for the operational as well as the safety effects of these medians. Specific objectives of the research were (1) to evaluate the safety and operational effectiveness of TWLTL medians on urban four-lane roadways, (2) to develop a methodology for evaluating the cost-effectiveness of the TWLTL, and (3) to apply this methodology to develop guidelines for cost-effective use of TWLTL medians on urban four-lane roadways. The methodology and guidelines were developed to enable the identification of sections of urban four-lane undivided roadway on which the costs of installing TWLTL medians would be justified.

Computer simulation was used to determine the effects on the efficiency of traffic operations that result from the installation of TWLTL medians on urban four-lane undivided roadways. The computer simulation study was conducted using the Two-Way Left-Turn Lane Computer Simulation Model (TWLTL-SIM) developed by Ballard and McCoy at the University of Nebraska-Lincoln. This model has been used in several published assessments of TWLTL operations (*1–4*). The results of the computer simulation study were incorporated into the cost-effectiveness methodology developed in this research to compute the operational benefits of TWLTL medians. Presented in this paper are a description of the simulation model and the procedures, findings, and conclusions of the study.

SIMULATION MODEL

The TWLTL-SIM model used in this study is capable of simulating traffic operations on four types of roadways: (1) two-lane undivided roadways, (2) two-lane roadways with TWLTL medians, (3) four-lane undivided roadways, and (4) four-lane roadways with TWLTL medians. The model was written in the General Purpose Simulation System Version H (GPSS/H) Language, which is a special-purpose language particularly suited to modeling discrete systems (*5, 6*). The following discussion presents the input, logic, output, and validation of the model.

J. L. Ballard, Department of Industrial and Management Systems Engineering, University of Nebraska-Lincoln, 175 Nebraska Hall, Lincoln, Neb. 68588-0518. P. T. McCoy, Department of Civil Engineering, University of Nebraska-Lincoln, W348 Nebraska Hall, Lincoln, Neb. 68588-0531.

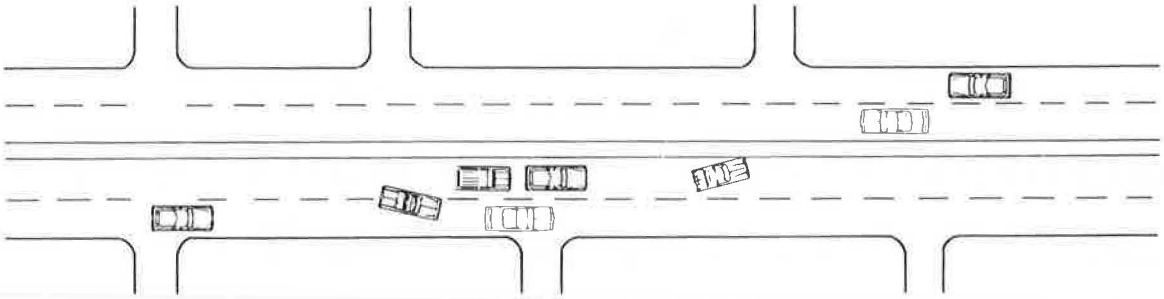


FIGURE 1 Four-lane undivided roadway.

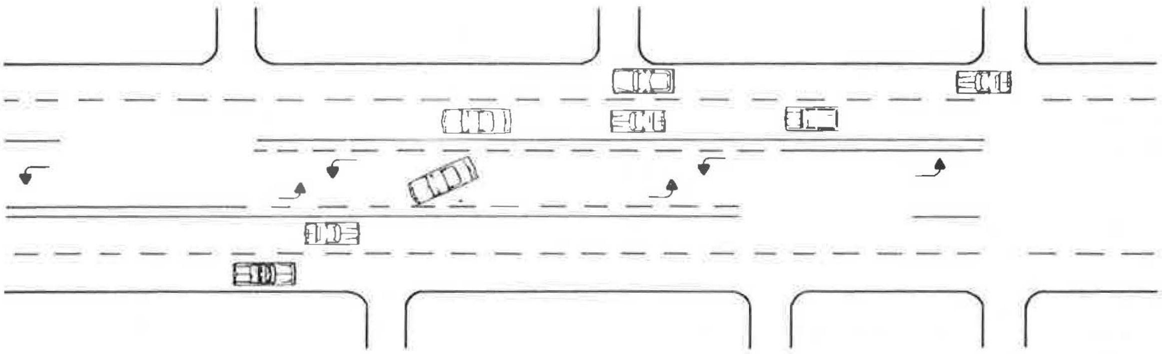


FIGURE 2 Four-lane roadway with TWLTL median.

Input

The input to the model consists of roadway geometrics and traffic characteristics data. These data include the following information:

- number of through-lanes
- presence or absence of a TWLTL
- length of roadway simulated
- locations of individual driveways
- entering traffic volume by lane in each direction, in vehicles per hour (vph)
- arrival distribution of entering traffic
- turning movement percentages at individual driveways
- travel speed in each direction, in miles per hour (mph)
- random number seeds that serve as the basis for the probabilistic generation of entering traffic headways, turning locations, and gap acceptance criteria

The model can generate nonrandom as well as random arrival patterns, so the effects of an upstream traffic signal can be simulated.

Because of the nature of the GPSS/H language, the roadway geometry is defined in terms of sections. Each lane on the roadway is divided into 20-ft. sections. Driveway locations are defined by the numbers of the sections in which they are located. Also, specified for each driveway on a roadway with a TWLTL is the section number of the farthest point upstream at which a vehicle turning left into the driveway can enter the TWLTL. Typically, left-turn vehicles enter the TWLTL at a distance of 200 feet upstream from the driveway into which they turn.

As an example, the geometry of a 1,000-ft. segment of two-lane roadway with a TWLTL is illustrated in Figure 3. Each lane is divided into fifty 2-ft. sections, which are numbered as follows:

- Lane 1: Sections 1–50
- Lane 2: Sections 51–100
- TWLTL: Sections 101–150.

The section numbers of the driveway locations and their corresponding TWLTL entry points that would be input to the model for the roadway in Figure 3 are shown in Table 1. In the case of a 1,000-ft. segment of two-lane undivided roadway, Section Nos. 101–150 would not exist. Therefore, only the section numbers of the driveway locations would be input to the model, because there would be no TWLTL and no TWLTL entry points.

Logic

In the TWLTL-SIM model, traffic enters the simulated roadway segment at either end in accordance with the traffic volumes and arrival patterns specified in the input. Three paths are possible for any vehicle entering at either end of the segment. The vehicle may (1) traverse the entire length of the segment without turning and exit at the far end, (2) traverse a portion of the segment and exit by turning left at one of the driveways, or (3) traverse a portion of the segment and exit by turning right at one of the driveways. On entering the segment, the path to be taken by each vehicle is determined probabilistically in accordance with the turning percentages specified in the input.

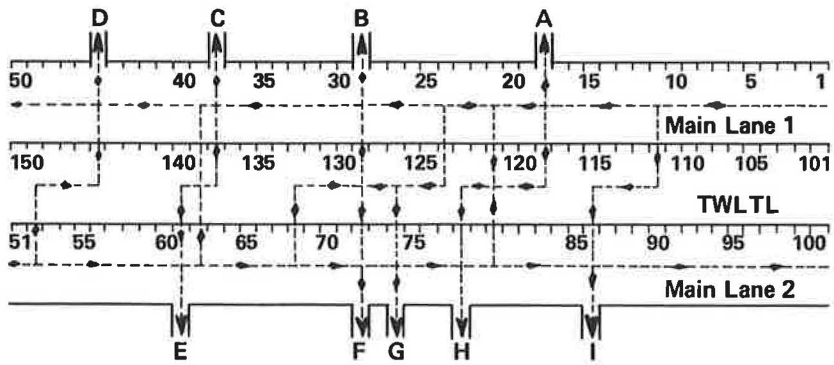


FIGURE 3 Geometry of 1,000-ft. segment of two-lane roadway with TWLTL median.

TABLE 1 SECTION NUMBERS INPUT FOR ROADWAY SEGMENT IN FIGURE 3

Driveway	Driveway Entered From	Location Section No.	TWLTL Entry Point Section No.
A	2	18	121
B	2	29	133
C	2	38	139
D	2	45	149
E	1	61	139
F	1	72	124
G	1	74	124
H	1	78	121
I	1	86	111

In addition to the vehicles entering the roadway segment at either end, some vehicles enter the segment by turning right onto the roadway from a driveway. All of these vehicles traverse the remainder of the segment and exit at the far end. The model does not include the capability to simulate left turns onto the roadway from driveways.

Left turns off the roadway may delay following vehicles or force them to stop if no TWLTL is present; such delay provides a measure of TWLTL effectiveness. The right turns onto and off the simulated roadway have no direct impact on TWLTL effectiveness. However, a right turn off the roadway could create a gap through which an opposing vehicle could turn left, while a right turn onto the roadway could fill such a gap so that it would not be available to opposing left-turn vehicles.

Vehicles move through each 20-ft. section in the main lanes at a constant speed specified in the model input and maintain at least 2-second headway. When a 2-second headway cannot be maintained behind a vehicle slowing or stopping to make a turn, following vehicles will use a uniform deceleration rate of 5 ft/sec^2. Then, when system conditions warrant, vehicles will accelerate at a uniform rate of 5 ft/sec^2 to regain their specified speed. The constant speed assumption means that the entering headway distribution is preserved until modified by the responses to turning vehicles. The assumptions of a constant speed and a constant minimum headway are an oversimplification of driver speed selection and car-following behavior on actual highways. However, these assumptions are justified in this case since the objective of the model is not to estimate the actual travel speed on arterial streets, but rather to simulate the left-turn gap acceptance process and estimate its impact on traffic operations.

In a model run with two through-lanes, one in each direction, vehicles entering in one of the through-lanes continue in that lane until they turn or exit at the other end of the roadway segment. If these vehicles encounter vehicles stopped in the through-lane ahead, they must also slow down and stop if necessary. They are not allowed to pass to the right of the stopped vehicles.

In a model run with four through-lanes, through-vehicles are assigned probabilistically to either the inside or outside lane at the entrance point of the simulated roadway. Through-vehicles that are delayed by turning vehicles on a four-lane roadway may change lanes to avoid delay. Vehicles that intend to turn right from the roadway are assigned to enter in the outside lane and continue in that lane until they reach their designated turning point. A vehicle that intends to turn left enters the segment in the inside lane and remains in the inside lane until it reaches its turning point. If there is no TWLTL, this turning point is the driveway into which it is to turn. If there is a TWLTL, this point is the TWLTL entry point of the driveway into which it is to turn. In the case of a TWLTL, the vehicle begins to decelerate at the TWLTL entry point. When it has slowed to a speed of 10 mph, it enters the TWLTL and moves ahead in the TWLTL until it reaches the driveway into which it is to turn or until it is stopped by vehicles already in the TWLTL waiting to turn left. The model continuously monitors the TWLTL and adjusts the entry point for left-turning vehicles as queues develop in the TWLTL.

If a turning vehicle reaches its entry point to the TWLTL and finds that the TWLTL section is already occupied by a left-turning vehicle from the other direction, it remains in the through lane and moves ahead until it (a) finds an unoccupied section in the TWLTL upstream from the driveway into which it is to turn, (b) reaches the driveway and stops in the through lanes, or (c) aborts the turn. In model runs at high flow rates both with and without a TWLTL, a vehicle will abort its turn and proceed ahead when stopping would cause locked or jammed flow. This capability to abort a turn was added to the model to prevent it from ceasing operation due to jamming under very high flow conditions.

In all situations, a left-turn vehicle must have a minimum acceptable gap in the opposing traffic stream before it can turn left. The required length of gap is determined probabilistically in accordance with the left-turn gap acceptance function derived by Gerlough and Wagner (*7*). The cumulative distribution function is shown in Table 2. The gap acceptance distribution based on Gerlough and Wagner was one of several candidates considered for the model. It produced the closest agreement with the field data collected for model validation.

If the left-turn vehicle is at the head of the queue and the required gap is available, the vehicle turns left. Otherwise it waits for an acceptable gap. However, if a left-turn vehicle is not at the head of the queue, it will follow the preceding left-turn vehicle across the opposing roadway as long as the available gap is longer than a minimum clearance time (1.5 seconds to cross one lane, 2.86 seconds to cross two lanes).

Output

The output from the model includes the following data:

- number of vehicles entering and exiting the segment
- number of left turns attempted and completed
- number of stops
- travel time in the segment
- stopped-time delay
- number of lane changes

The travel time, stops, and delay totals for through-vehicle, left-turning vehicles, and all vehicles are output separately.

TABLE 2 LEFT-TURN GAP ACCEPTANCE FUNCTION (7)

Gap(Seconds)	Probability of Accepting a Shorter Gap
3.0	0.00
3.5	0.15
4.0	0.32
4.5	0.52
5.0	0.69
5.5	0.82
6.0	0.90
6.5	0.95
7.0	0.97
7.5	0.986
8.0	0.993
8.5	0.997
9.0	0.998
9.5	0.999
10.0	1.000

[a] Source: Reference 7.

Validation

Time-lapse film of traffic flow on three roadway sections was taken in order to validate the model. Two sites were located in Omaha, Nebraska, and the third site was located in Lincoln, Nebraska. One of the sites in Omaha was a four-lane undivided section of roadway. The other site in Omaha and the one in Lincoln were four-lane sections with TWLTL medians. A total of 6 hours of film was obtained.

The films were analyzed to determine the volumes, left-turn percentages, travel times, delays, and percentage of vehicles stopping on the roadway sections. The model was then run using the actual traffic volumes, left-turn percentages, and roadway geometrics as input data. The results of the simulation runs were compared with those obtained from the films.

Paired *t*-tests were conducted to compare the mean stopped-time delay and mean percentage of vehicles stopping predicted by the model with those computed from the film analysis. As shown in Table 3, there was no statistically significant difference at the 0.05 level of significance between the predicted and observed means at the four-lane undivided site. Likewise, at the TWLTL sites, there was no statistically significant difference at the 0.05 level of significance between the predicted and observed values of mean stopped-time delay. However, there was a significant difference at the 0.05 level of significance between the predicted and observed values of mean percentage of vehicles stopping at the TWLTL sites, although this difference was not significant at the 0.01 level of significance. Based on a review of the film analysis, it was determined that this difference was attributable to the difficulty of the judgments required of the film analysts to distinguish vehicle stops. Therefore, it was concluded that the model was statistically valid.

In addition to statistical validation, the model has been subjected to considerable face validation. During the past 5 years, the model has been used to simulate traffic operations on several roadway segments with a wide variety of conditions (*1–4*). In each case, the model provided reasonable and consistent results.

PROCEDURE

In order to determine the operational effects of TWLTL medians on urban four-lane roadways, two sets of computer simulation runs were made with the model. One set of runs was made for four-lane undivided roadways without TWLTL medians. The second set of runs was made for four-lane roadways with TWLTL medians. Both sets of runs were made over the same ranges of traffic volumes and driveway densities. The effects of the TWLTL on stops and delay were then determined by pairwise comparisons of the model outputs for the two sets of runs for identical combinations of traffic volumes and driveway densities. Thus, for every combination of traffic volumes and driveway density, the effects

TABLE 3 COMPARISON OF MODEL AND OBSERVED RESULTS

Site	Measure of Effectiveness	Mean Difference (Model-Observed)	Std. Dev. of Difference	t	Significant[a]
Four-lane undivided	Percentage of vehicles stopping	30.33	24.58	2.14	No
	Average stopped delay (veh-min/hr)	1.13	2.17	0.65	No
Five-lane with TWLTL	Percentage of vehicles stopping	28.00	7.0	6.93	Yes[b]
	Average stopped delay (veh-min/hr)	0.17	4.05	0.07	No

[a]Statistical significance at 0.05 level of significance unless otherwise specified.

[b]Statistically significant at the 0.05 level, but not at the 0.01 level of significance.

of the TWLTL on stops and delay were computed as the differences between the respective outputs of the two runs. Multiple regression analyses of the results of the simulation runs were conducted to determine the relationship between the effects of TWLTL medians and prevailing roadway and traffic conditions. As a result of these analyses, regression equations were developed for predicting the reductions in stops and delay provided by TWLTL medians.

Simulation Runs

A series of paired runs were made to simulate traffic operations on a 1,000-ft. section of four-lane roadway while varying the traffic volume, left-turn percentage, and driveway density. Six levels of traffic volume, five levels of left-turn percentage, and three levels of driveway density were used. Simulation runs were made for a total of 54 combinations of those variable levels. The specific combinations that were simulated are shown in table 4. For each combination indicated, a pair of simulation runs were made, one run with a TWLTL and one run without a TWLTL. In each pair of runs, the same random number seeds were used, so that the identical traffic stream was used in each run. Three to five paired runs were made for each combination indicated in table 4. The combinations used in this study were selected based on the results of previous studies (*1, 2*) conducted with the TWLTL-SIM model. The combinations focus on the range of traffic operations that is of most practical interest. Lower volume levels would produce very few stops and little delay; whereas higher volume levels would produce jammed conditions because the capacity of the roadway would be exceeded.

For each driveway density simulated, the driveway locations input to the model were equally spaced driveways staggered on opposite sides of the roadway. Intuitively, the locations of the driveways within the 1,000-ft. section would have an effect on the efficiency of traffic operations. But it was beyond the scope of this study to investigate these differences within driveway density levels. Instead the primary concern of the study was to examine the differences between driveway

TABLE 4 CONDITIONS SIMULATED

Traffic Volume[a] (vph)	Driveway Density[b] (driveways/ mile)	Left-Turn Percentage[c] 2.5%	5%	7.5%	10%	12.5%
100	30			X	X	X
	60			X	X	X
	90			X	X	X
300	30			X	X	X
	60			X	X	X
	90			X	X	X
500	30			X	X	X
	60			X	X	X
	90			X	X	X
650	30			X	X	X
	60			X	X	X
	90			X	X	X
900	30		X	X	X	
	60		X	X	X	
	90		X	X	X	
1,100	30	X	X	X		
	60	X	X	X		
	90	X	X	X		

[a]Traffic volume in each direction.

[b]Total of driveways on both sides of roadway.

[c]Percentage of sum of traffic volumes in both directions.

density levels. Therefore, only the one configuration of driveway locations was used for all levels of driveway density.

The direction split used in making all simulation runs was 50/50. Preliminary runs made using 60/40 splits indicated that more stops and delay resulted using 50/50 splits. More gaps in the lower volume direction of a 60/40 split were available to accommodate the higher left-turn volume from the other direction. Therefore, it was assumed that maximum stops and delay occur with a 50/50 split.

In all simulation runs, a left-turn or right-turn vehicle was equally likely to turn into any of the driveways on the side of the roadway appropriate for the turn. Thus, all driveways had the same turning volumes. Also, all simulation runs were made using 10% right turns into driveways and 10% right turns out of driveways. As noted above, these right-turn maneuvers do not directly impact stops and delay, but they do create gaps for left turns and fill gaps that could otherwise be used for left turns.

The travel speeds used approximated the speed-volume relationships on urban arterial roadways (*8*). A travel speed of 40 mph was used for traffic volumes of 650 vph or less. A travel speed of 35 mph was used for traffic volumes greater than 650 vph.

Each simulation run was initialized by running the model for a few minutes to achieve system stability. Once stability was reached, the model was run for one hour of stimulated time. Traffic operations data were than output for this hour.

Data Analysis

The effects of TWLTL medians on stops and delay were computed for each set of conditions indicated in table 4 from the model output for each pair of simulation runs made for the particular set of conditions. The reductions in stops and delay provided by the TWLTL were computed by subtracting the stops and delay output for the run with the TWLTL from the stops and delay output for the run without the TWLTL. Once

TABLE 5 AVERAGE REDUCTIONS IN STOPS[a]

Traffic Volume[b] (vph)	Driveway Density[c] (driveways/mile)	Left-Turn Percentage[d] 2.5%	5%	7.5%	10%	12.5%
100	30			1	1	0
	60			1	1	3
	90			1	1	2
300	30			14	13	13
	60			3	8	9
	90			3	10	9
500	30			34	51	66
	60			30	41	45
	90			26	42	61
650	30			129	154	182
	60			96	137	167
	90			106	143	139
900	30		588	896	844	
	60		338	449	466	
	90		224	305	482	
1,100	30	971	1,133	933		
	60	849	1,314	1,154		
	90	1,090	1,190	1,035		

[a]Stops per hour.

[b]Traffic volume in each direction.

[c]Total of driveways on both sides of roadway.

[d]Percentage of sum of traffic volumes in both directions.

the reductions in stops and delay had been computed for all the conditions simulated, multiple regression analyses were conducted to determine the relationships between these reductions and the various levels of traffic volume, left-turn percentage, and driveway density.

The Statistical Analysis System (SAS) (*9*) was used to conduct the multiple linear regression analyses. These analyses were performed using a step-wise procedure with both forward and backward selection at the 0.05 level of significance. The dependent variables used in these analyses were reduction in stops (number per hour) and reduction in delay (seconds per hour). The natural logarithms of these variables were also used as dependent variables. The independent variables considered were traffic volume (vehicles per hour), left-turn volume (vehicles per hour), driveway density (driveways per mile), left-turn volume per driveway (vehicles per hour), and product of traffic volume (vehicles per hour) times left-turn volume (vehicles per hour).

FINDINGS

The average reductions in stops computed for the combinations of traffic volume, left-turn percentage, and driveway density for which simulation runs were made are shown in table 5. In no case did the TWLTL increase stops. Within each level of driveway density, the average reduction in stops increased as traffic volume was increased. Likewise, within each level of driveway density, the average reduction in stops usually increased as left-turn percentage was increased, except at the highest level of traffic volume. The influence of driveway density within each level of traffic volume was not consistent; however, in most cases, the average reduction in stops was highest at 30 driveways per mile. This was because the left-turn volume was apportioned equally among the driveways. Therefore, the left-turn volume per driveway at 30 driveways per mile was two and three times greater than it was at 60 and 90 driveways per mile, respectively. Conse-

TABLE 6 AVERAGE REDUCTIONS IN DELAY[a]

Traffic Volume[b] (vph)	Driveway Density[c] (driveways/ mile)	Left-Turn Percentage[d] 2.5%	5%	7.5%	10%	15%
100	30			3	4	4
	60			3	3	3
	90			2	3	3
300	30			31	44	63
	60			26	37	53
	90			22	32	45
500	30			215	333	518
	60			180	279	434
	90			155	241	374
650	30			723	1,103	1,683
	60			605	924	1,409
	90			522	796	1,215
900	30		8,607	14,037	22,893	
	60		5,277	6,739	8,607	
	90		4,570	5,431	6,455	
1,100	30	34,559	64,454	117,190		
	60	26,290	35,449	47,800		
	90	24,077	29,733	36,718		

[a]Seconds per hour.

[b]Traffic volume in each direction.

[c]Total of driveways on both sides of roadway.

[d]Percentage of sum of traffic volumes in both directions.

TABLE 7 REGRESSION EQUATIONS FOR PREDICTING REDUCTIONS IN STOPS AND DELAY

Traffic Volume[a] (vph)	Reduction	Equation[b]	R^2
<800	stops	$\ell_n\ S = 0.00579\ V_t + 0.0117\ V_\ell - 0.00678D$	0.975
	delay	$\ell_n\ D = 0.00845\ V_t + 0.0330\ V_\ell - 0.00561D - 0.0000308P$	0.978
≥800	stops	$\ell_n\ S = 0.00610\ V_t + 0.0282\ V_d$	0.996
	delay	$\ell_n\ D = 0.00898\ V_t + 0.0652\ V_d$	0.996

[a]Traffic volume in each direction.

[b] S = reduction in stops (number per hour per 1,000 ft.)

D = reduction in delay (seconds per hour per 1,000 ft.)

V_t = average traffic volume per direction (vph)

V_ℓ = sum of left-turn volumes in both directions (vph)

V_d = average left-turn volume per driveway (vph per driveway)

D = driveway density (driveways per mile)

$P = V_t \cdot V_\ell$

quently, more queuing of left-turn vehicles would tend to occur at 30 driveways per mile. At 60 and 90 driveways per mile, vehicles waiting to turn left at several driveways would be more likely to turn left through the same gap in the oncoming traffic stream.

The average reductions in delay that were computed from the simulation runs are shown in table 6. In all cases, the TWLTL provided reductions in delay. The pattern of these reductions with respect to traffic volume, left-turn percentage, and driveway density was more consistent than that of the average reductions in stops. The average reductions in delay increased with increases in traffic volume and left-turn percentage, and they decreased with increases in driveway density.

As a result of the regression analyses, two sets of regression equations were developed for predicting the reductions in stops and delay that would result from the installation of a TWLTL on a four-lane undivided roadway. One set of equations was for traffic volumes below 800 vph, and the other set was for traffic volumes of 800 vph or higher. Each set contained two equations. One equation was for predicting the reductions in stops, and the other equation was for predicting the reductions in delay.

The regression equations are shown in table 7. For traffic volumes below 800 vph, the stops and delay reduction equations were exponential functions of traffic volume, left-turn volume, and driveway density and, in cases of delay reduction, the product of traffic volume and left-turn volume. These equations accounted for more than 97% of the variations in the natural logarithms of the reductions in stops and delays.

For traffic volumes of 800 vph or higher, both the stops and delay reduction equations were exponential functions of traffic volume and left-turn volume per driveway. These equations account for more than 99% of the variations in the natural logarithms of the reductions in stops and delay.

The independent variables in all four of these regression equations were statistically significant at the 0.05 level of significance.

CONCLUSIONS

Based on the results of the computer simulation study, it was concluded that (1) as expected, the installation of TWLTL medians on urban four-lane undivided roadways provided reductions in stops and delay over a wide range of traffic volume, left-turn percentage, and driveway density variables; and (2) the magnitudes of these reductions were exponential functions of traffic volume, left-turn volume, and driveway density. It was also concluded that the stops and delay reduction equations presented in table 7 should be used in the cost-effectiveness methodology developed in this research to compute the operational benefits of the TWLTL medians. The cost-effectiveness methodology is presented elsewhere (*10*).

ACKNOWLEDGMENTS

This is the final report of project HPR 82–3, "Guidelines for Use of a Two-Way Left-Turn Lanes." The research was conducted by the Civil Engineering Department, University of Nebraska-Lincoln in cooperation with the Nebraska Department of Roads and the U.S. Department of Transportation, Federal Highway Administration. The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Nebraska Department of Roads or the Federal Highway Administration.

REFERENCES

1. P. T. McCoy, J. L. Ballard, and Y. H. Wiyaya. Operational Effects of Two-Way Left-Turn Lanes on Two-Way, Two-Lane Streets. In *Transportation Research Record 869*, TRB, National Research Council, Washington, D.C., 1982, pp. 49–54.
2. J. L. Ballard and P. T. McCoy. Operational Effects of Two-Way Left-Turn Lanes on Two-Way, Four-Lane Streets. In *Transportation Research Record 923*, TRB, National Research Council, Washington, D.C., 1983, pp. 54–57.
3. Z. A. Nemeth, P. T. McCoy, and J. L. Ballard. Impact of Two-Way Left-Turn Lanes on Fuel Consumption. In *Transportation Research Record 901*, TRB, National Research Council, Washington, D.C., 1983, pp. 29–33.
4. D. W. Harwood. *Multilane Design Alternatives for Improving Suburban Highways*. NCHRP Report 282, TRB, National Research Council, Washington, D.C., 1986.
5. T. J. Schriber. *Simulation Using GPSS*. John Wiley and Sons, Inc., New York, 1974.
6. J. O. Henriksen and R. C. Crane. *GPSS/H User's Manual*. Wolverine Software, Ann Arbor, Mich., 1983.
7. D. L. Gerlough and F. A. Wagner. *Improved Criteria for Traffic Signals at Individual Intersections*. NCHRP Report 32, HRB, National Research Council, Washington, D.C., 1967.
8. *Highway Capacity Manual, 1965*. Special Report 87, HRB, National Research Council, Washington, D.C., 1965.
9. *Statistical Analysis System*. SAS Institute, Inc., Cary, N.C., 1979.
10. P. T. McCoy and J. L. Ballard. *Cost-Effectiveness Evaluation of Two-Way Left-Turn Lanes on Urban Four-Lane Roadways*. Research Report No. TRP-02-16-86. Civil Engineering Department, University of Nebraska-Lincoln, August 1986.

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

Two-Way Left-Turn Lane Guidelines for Urban Four-Lane Roadways

PATRICK T. MCCOY, JOHN L. BALLARD, DUANE S. EITEL, AND WALTER E. WITT

Two-way left-turn lane (TWLTL) medians are commonly used to solve the safety and operational problems on four-lane undivided roadways caused by conflicts between through- and left-turning traffic. Although the potential safety and operational effects of TWLTL medians are well-recognized, there are no generally accepted guidelines that define the circumstances under which the costs of TWLTL medians are justified by the benefits they provide. The objectives of the research on which this paper was based were (1) to evaluate the safety and operational effects of TWLTL medians on urban four-lane roadways, (2) to develop a methodology for evaluating their cost-effectiveness, and (3) to use this methodology to develop guidelines for their cost-effective use. The formulation of the cost-effectiveness methodology was based on a benefit-cost analysis approach. The benefits were the accident and operational cost savings provided by TWLTL medians. The costs were the costs of installing and maintaining them. The cost-effectiveness methodology was used to develop guidelines that indicate the average daily traffic levels (ADTs), left-turn percentages, and driveway densities at which TWLTL medians on urban four-lane roadways are cost-effective. Their development was based on conditions and costs representative of those on urban four-lane roadways in Nebraska during 1986. Over the range of conditions considered, TWLTL medians were cost-effective at lower ADTs on roadways with higher left-turn percentages and fewer driveways per mile. The minimum ADT required for TWLTL medians to be cost-effective ranges from 6,200 to 6,600 vehicles per day (vpd), depending on the left-turn percentage and driveway density.

Two-way left-turn lane (TWLTL) medians are commonly used to solve the safety and operational problems on four-lane undivided roadways caused by conflicts between through- and mid-block left-turn traffic. Left turns from a four-lane undivided roadway are made from through traffic lanes causing through vehicles in these lanes to change lanes or be delayed. But on a roadway with a TWLTL, the deceleration and storage of left-turn vehicles are removed from the through lanes. Thus, conflicts between through-and left-turn vehicles are eliminated, and through-vehicles can pass left-turn vehicles without changing lanes and without delay.

Although the potential safety and operational effects of the TWLTL are recognized by highway engineers, there are no generally accepted guidelines that define the circumstances under which the costs of providing TWLTL medians are justified. Numerous before-and-after studies of the safety effectiveness of TWLTLs have been conducted. However, empirical data pertinent to the assessment of the operational effectiveness of the TWLTL are limited. Therefore, previous attempts to develop guidelines for the use of the TWLTL have focused on the safety benefits and have not adequately considered the operational effectiveness of the TWLTL.

The overall objective of the research on which this paper was based was to develop guidelines for the use of TWLTL medians that would account for the operational as well as the safety effects of these medians. Specific objectives of the research were (1) to evaluate the safety and operational effectiveness of TWLTL medians on urban four-lane roadways, (2) to develop a methodology for evaluating the cost-effectiveness of the TWLTL, and (3) to apply this methodology to develop guidelines for the cost-effective use of TWLTL medians on urban four-lane roadways. The methodology and guidelines were developed to enable the identification of sections of urban four-lane undivided roadways on which the cost of providing TWLTL medians is justified.

An analysis of accidents on the urban four-lane sections of the state highway system in Nebraska was conducted to assess the effectiveness of TWLTL medians in reducing accidents on urban four-lane roadways. Computer simulation was used to determine the operational effects of TWLTL medians. Results of the accident analysis and computer simulation study were used in the formulation of the cost-effectiveness methodology. Formulation of the cost-effectiveness methodology was based on a benefit-cost evaluation of these medians. The benefits were the accident and operational cost savings provided by TWLTL medians. The costs were those of installing and maintaining TWLTL medians. According to the methodology, if the benefits of a TWLTL exceed its costs, the TWLTL would be cost-effective.

Finally, the cost-effectiveness methodology was applied to a range of traffic volumes and driveway densities. This was done to determine the combinations of traffic volumes and driveway densities for which the construction and maintenance of TWLTL medians on urban four-lane roadways in Nebraska are cost-effective. The total annual cost savings provided by the TWLTL medians were evaluated over the range of traffic volumes and driveway densities. These savings were compared to the annual costs of constructing and maintaining TWLTL medians for the same range. Traffic volumes and driveway densities for which the total annual cost savings were greater than the annual cost of the TWLTL medians were determined to be those conditions for which TWLTL medians are cost-effective. The results of the cost-effective-

P. T. McCoy, Department of Civil Engineering, University of Nebraska-Lincoln, W348 Nebraska Hall, Lincoln, Neb. 68588-0531. J. L. Ballard, Department of Industrial and Management Systems Engineering, University of Nebraska-Lincoln, 175 Nebraska Hall, Lincoln, Neb. 68588-0518. D. S. Eitel and W. E. Witt, Roadway Design Division, Nebraska Department of Roads, P.O. Box 94759, Lincoln, Neb. 68509-4759.

ness analysis provided guidelines for the cost-effective use of TWLTL medians on urban four-lane roadways in Nebraska. The procedure, findings, and conclusions of this analysis are presented in this paper. The development of the cost-effectiveness methodology and other findings of this research are presented elsewhere (*1*).

PROCEDURE

The cost-effectiveness analysis was conducted for the addition of TWLTL medians on urban four-lane roadways. The results were intended to be representative of conditions on urban four-lane roadways in Nebraska during 1986. TWLTL medians were evaluated over the following range of traffic volumes and driveway densities:

- ADT: 5,000 to 25,000 vpd at 5,000-vpd increments
- Left-turn percentage: 2.5 to 12.5% at 2.5% increments
- Driveway density: 30 to 90 driveways/mile at 15 driveways/mile increments.

Thus, five levels of each variable were evaluated, which amounted to an evaluation of 625 combinations of ADT, left-turn percentage, and driveway density.

The same truck percentages were used to evaluate each combination. The truck percentages used were 2.1% single unit trucks and 1.3% combination trucks. These percentages were the average truck percentages reported at the continuous traffic count stations maintained by the Nebraska Department of Roads (*2*) on urban arterial streets.

A brief description of the evaluation procedure relative to the calculation of the benefits and costs of TWLTL medians follows.

Accident Cost Savings

During the 4-year period from January 1, 1980, to January 1, 1984, the accident rate on urban four-lane roadways with TWLTL medians on the state highway system in Nebraska was 8.4 accidents per million vehicle miles (*1*). Urban four-lane undivided sections on the state highway systems, which had similar prevailing roadway and traffic conditions as the TWLTL sections, had an accident rate of 12.7 accidents per million vehicle miles during the same period. Thus, the accident rate on the TWLTL sections was 34% lower than that on the four-lane undivided sections. A Poisson comparison of means test indicated that the rates were significantly different at the 5% level of significance. Also, the observed 34% difference was comparable to the TWLTL accident reduction factors of 20 to 40%, which were determined from before-and-after accident studies reported in the literature (*3–8*). Therefore, for the purpose of this cost-effectiveness analysis, it was concluded that the installation of a TWLTL median on an urban four-lane undivided roadway would reduce the accidents by 30%.

Overall, accidents on the TWLTL sections were more severe than those on the four-lane undivided sections. On the TWLTL sections, 35% of the accidents were fatal and nonfatal injury accidents. On the four-lane undivided sections, only 27% of the accidents were that severe. A chi-square test showed this difference to be significant at the 5% level of significance. However, previous before-and-after studies (*3, 5, 9*) have found that TWLTL medians reduce, rather than increase, accident severity. This suggested that perhaps the comparative study used in this analysis confounded the effects of the TWLTL medians on accident severity with those of other factors not considered. However, the limitations of the available data did not permit further examination of this contradiction of previous research findings. Therefore, for the purpose of this cost-effectiveness analysis, the accident severity for four-lane undivided roadways (i.e., 0.10% fatal, 26.5% nonfatal injury, and 73.4% property-damage-only) was used to compute the safety benefits of installing TWLTL medians on urban four-lane roadways.

The accident experience to which the 30% reduction factor was applied was the mean mid-block accident rate on urban four-lane undivided sections of the state highway system in Nebraska during the 2-year period from July 1, 1984, to July 1, 1986. Signalized intersections often have left-turn bays and left-turn phasing even on undivided roadways. In such cases, the installation of TWLTL medians would have little effect on safety at these intersections. Therefore, the accidents at signalized intersections were excluded from the calculation of the accident reduction. On the other hand, TWLTL medians would improve safety at unsignalized intersections, which usually do not have left-turn bays on undivided roadways. However, the available mean accident rate data (*10*) did not distinguish between signalized and unsignalized intersections. Therefore, the mid-block accident rate was used to avoid overstating the accident cost saving provided by TWLTL medians.

The mean mid-block accident rate was 6.17 accidents per million vehicle miles (*10*). Application of the 30% reduction factor to the mean mid-block accident rate provided an accident reduction of 1.85 accidents per million vehicle miles.

The 1986 unit accident costs used by the Nebraska Department of Roads were $220,000 per fatal accident, $9,300 per non-fatal injury accident, and $1,190 per property-damage-only accident. Applying these costs to the average severity, the average cost of an accident on a four-lane undivided roadway was computed to be $3,560. Thus, the rate of accident cost savings used in this analysis was $6,590 per million vehicle miles.

Operational Cost Savings

The operational cost savings provided by TWLTL medians are the savings in road-user stopping and travel time costs that result from the reductions in stops and delay provided by TWLTL medians. The regression equations in table 1, which were determined in the computer simulation study (*1*), are used in the methodology to predict the reductions in stops and delay provided by TWLTL medians.

Stopping Cost Savings

The savings in stopping costs were computed from the reductions in stops provided by TWLTL medians. The hourly stopping cost savings were computed as follows:

$$SCS = 0.00528\ \Delta\ S \cdot L \sum_{i=1}^{3} P_i S_i M_i \tag{1}$$

TABLE 1 REGRESSION EQUATIONS FOR PREDICTING REDUCTIONS IN STOPS AND DELAY

Traffic Volume[a] (vph)	Reduction	Equation[b]	R^2
<800	stops	$\ln S = 0.00579\ V_t + 0.0117\ V_l - 0.00678D$	0.975
	delay	$\ln D = 0.00845\ V_t + 0.0330\ V_l - 0.00561D - 0.0000308P$	0.978
≥800	stops	$\ln S = 0.00610\ V_t + 0.0282\ V_d$	0.996
	delay	$\ln D = 0.00898\ V_t + 0.0652\ V_d$	0.996

[a]Traffic volume in each direction.

[b] S = reduction in stops (number per hour per 1,000 ft.)

D = reduction in delay (seconds per hour per 1,000 ft.)

V_t = average traffic volume per direction (vph)

V_l = sum of left-turn volumes in both directions (vph)

V_d = average left-turn volume per driveway (vph per driveway)

D = driveway density (driveways per mile)

P = $V_t \cdot V_l$

where:

SCS = stopping cost savings provided by a TWLTL on an urban four-lane roadway ($/hour);
ΔS = reduction in stops from table 1 (number/hour/1,000 ft);
L = length of roadway section (miles);
P_i = proportion of vehicle type i in the traffic stream (%/100%);
S_i = stopping cost for vehicle type i from table 2 ($/1,000 stops); and
M_i = updating multiplier for vehicle type i from table 3.

The stopping costs in table 2 were those published by AASHTO (*11*) for the year 1975. Three vehicle types were included: passenger cars, single unit trucks, and 3 – S2 combination trucks. The speeds used to determine the stopping costs shown for each level of traffic volume are the same speeds used in the computer simulation study (*1*), which approximated the speed-volume relationships on urban arterial roadways (*12*). The updating multipliers, in table 3, enable the 1975 stopping costs, in table 2, to be updated to the current year. These multipliers were computed according to the AASHTO (*11*) procedures based on changes in consumer and wholesale price indices (*13*). For the vehicle mix of 96.6% passenger cars, 2.1% single-unit trucks, and 1.3% 3 – S2 combination trucks, the cost per stop was $0.03849 for directional volumes of 700 vph or less, and $0.03290 for directional volumes above 700 vph.

Travel Time Cost Savings

The savings in travel time costs were computed from the reductions in delay provided by TWLTL medians. The hourly time costs savings were computed as follows:

$$TCS = 0.00147\ \Delta D \cdot L\ \frac{CPI}{156.1} \sum_{i=1}^{3} P_i T_i \tag{2}$$

where:

TCS = travel time cost savings provided by a TWLTL on an urban four-lane roadway ($/hour);
ΔD = reduction in delay from table 1 (seconds/hour, per 1,000 feet);
L = length of roadway section (miles);
CPI = consumer price index;
P_i = proportion of vehicle type i in the traffic stream (%/100%);
T_i = value of time for vehicle type i from table 4.

The values of time in table 4 were those established by AASHTO (*11*) for the year 1975. However, these values were updated to the current year by the ratio (CPI/156.1), which is the current consumer price index divided by the 1975 consumer price index. The 1986 consumer price index was 326.3 (*13*). Thus, for the vehicle mix of 96.6% passenger cars, 2.1% single unit trucks, and 1.3% 3 – S2 combination trucks, the hourly time cost was $1.23 per hour.

TABLE 2 STOPPING COSTS ($/1,000 STOPS)[a]

Vehicle Type	Traffic Volume[b] $\leq$650 vph[c]	>650 vph[d]
Passenger Car	21.00	17.75
Single Unit Truck	48.47	43.88
3-S2 Combination Truck	163.99	151.47

[a]Source: Reference 11.

[b]Traffic volume in each direction.

[c]Speed = 40 mph.

[d]Speed = 35 mph.

TABLE 3 UPDATING MULTIPLIERS FOR STOPPING COSTS

Vehicle Type	Updating Formula
Passenger Car	$M = 0.0022\ CPI_F + 0.0001\ CPI_O + 0.0033\ CPI_T + 0.0001\ CPI_M + 0.0017\ CPI_D$
Single Unit Truck	$M = 0.0018\ WPI_F + 0.0031\ WPI_T + 0.0002\ CPI_M + 0.0008\ WPI_D$
3-S2 Combination Truck	$M = 0.0008\ WPI_{FD} + 0.0047\ WPI_T + 0.0001\ CPI_M + 0.0003\ WPI_D$

where:

CPI_F - Consumer Price Index - Private Transportation, Gasoline Regular and Premium

CPI_O - Consumer Price Index - Private Transportation, Motor Oil, Premium

CPI_T - Consumer Price Index - Private Transportation, Tires

CPI_M - Consumer Price Index - Private Transportation, Auto Repairs and Maintenance

CPI_D - Consumer Price Index - Private Transportation, Automobiles, New

WPI_F - Wholesale Price Index - Regular Gasoline to Commercial Users (Code No. 05710203.05)

WPI_{FD} - Wholesale Price Index - Diesel Fuel to Commercial Users (Code No. 05730301.06)

WPI_T - Wholesale Price Index - Truck Tires (Code No. 07120105.07)

WPI_D - Wholesale Price Index - Motor Truck (Code No. 141106)

[a]Source: Reference 11.

Annual Operational Cost Savings

The annual operational cost savings provided by TWLTL medians were computed by summing the hourly stopping and travel time costs savings from Equations 1 and 2 as follows:

$$OSC = 365 \sum_{i=1}^{24} (SCS_i + TCS_i) \qquad (3)$$

where:

OCS = annual operational cost savings provided by a TWLTL on an urban four-lane roadway ($/year);
SCS_i = stopping cost savings from Equation 1 for the ith hour of an average day ($/hour); and
TCS_i = travel time cost savings from Equation 2 for the ith hour of an average day ($/hour).

TABLE 4 VALUES OF TIME[a]

Vehicle Type	$/vehicle-hour
Passenger Car	0.35[b]
Single Unit Truck	7.00
3-S2 Combination Truck	8.00

[a]Source: Reference 11.

[b]For low time savings, average trips, and 1.56 adults per vehicle.

In Equation 3 stopping and travel time cost savings were computed for each of the 24 hours in an average day. The hourly volumes were obtained by applying the hourly distribution shown in table 5 to the ADT being considered. This distribution was the average hourly distribution of daily traffic on the urban arterial-street sections of the state highway system in Nebraska (*2*). Savings were not computed for any hours with traffic volumes outside the traffic volume range (100 to 1,100 vph in each direction) of the regression equations in table 1. The stopping and travel time cost savings were assumed to be zero for hours with volumes less than 100 vph in each direction, and cases with hourly directional volumes above 1,100 vph were not considered.

TWLTL Cost

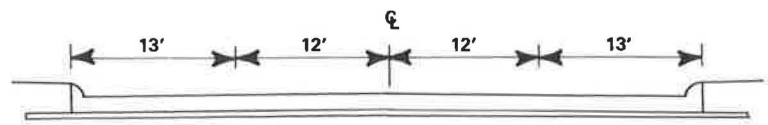
The cost of a TWLTL was computed to be the additional cost required to construct and maintain a TWLTL on a typical four-lane undivided roadway in Nebraska. The first cost of the TWLTL was computed as the difference between the first costs for a 50-ft. back-to-back section of urban four-lane undivided roadway and a 62-ft. back-to-back section of urban four-lane divided roadway with a painted median. These typical sections are shown in figure 1. The 1986 first costs of these sections were estimated by the Nebraska Department of Roads to be $1,190,000 per mile and $1,373,000 per mile, respectively. Thus, the estimated first cost of the TWLTL was $183,000 per mile. This estimate included the following cost items: right-of-way, earthwork, concrete pavement, drainage, utilities, and engineering. The first cost was annualized using a 6% interest rate, 20-year project life, and zero salvage value. Thus, the annualized first cost was $15,950 per mile.

The 1986 annual maintenance cost of the TWLTL was estimated by the Nebraska Department of Roads to be $800 per mile. This estimate included the maintenance cost items of pavement repair, pavement markings, and snow removal. Therefore, the total annual cost of the TWLTL was $16,750.

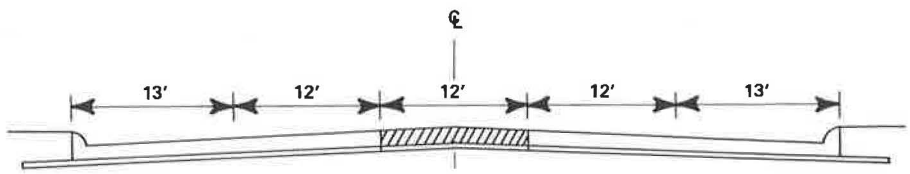
In each case evaluated the total annual cost savings (accident plus operational cost savings) was compared to the annual TWLTL cost to determine whether or not the TWLTL was cost-effective. The combinations of ADT, left-turn percentage, and driveway density, for which the savings were greater than the cost, were identified as those for which TWLTL medians on urban four-lane roadways are cost-effective.

TABLE 5 AVERAGE HOURLY DISTRIBUTION OF ADT

Hour	%ADT	Hour	%ADT
12:00 a.m. - 1:00 a.m.	1.45	12:00 p.m. - 1:00 p.m.	6.95
1:00 a.m. - 2:00 a.m.	0.98	1:00 p.m. - 2:00 p.m.	6.65
2:00 a.m. - 3:00 a.m.	0.49	2:00 p.m. - 3:00 p.m.	6.58
3:00 a.m. - 4:00 a.m.	0.33	3:00 p.m. - 4:00 p.m.	7.16
4:00 a.m. - 5:00 a.m.	0.31	4:00 p.m. - 5:00 p.m.	8.13
5:00 a.m. - 6:00 a.m.	0.86	5:00 p.m. - 6:00 p.m.	7.84
6:00 a.m. - 7:00 a.m.	2.53	6:00 p.m. - 7:00 p.m.	5.89
7:00 a.m. - 8:00 a.m.	5.65	7:00 p.m. - 8:00 p.m.	4.88
8:00 a.m. - 9:00 a.m.	4.80	8:00 p.m. - 9:00 p.m.	4.02
9:00 a.m. - 10:00 a.m.	4.58	9:00 p.m. - 10:00 p.m.	3.71
10:00 a.m. - 11:00 a.m.	5.15	10:00 p.m. - 11:00 p.m.	2.82
11:00 a.m. - 12:00 p.m.	6.09	11:00 p.m. - 12:00 p.m.	2.15



a. Four-lane undivided.



b. Four-lane divided with painted median.

FIGURE 1 Typical urban sections.

FINDINGS

The results of the cost-effectiveness analysis are presented in figure 2. Shown in this figure are the combinations of ADT, left-turn percentage, and driveway density for which the use of TWLTL medians on urban four-lane roadways in Nebraska is cost-effective. For a given driveway density, the combinations of ADT and left-turn percentage for which TWLTL medians are cost-effective are located to the right of the curve that corresponds to the particular driveway density. The combinations for which TWLTL medians are not cost-effective are located to the left of the driveway-density curve. For example, on an urban four-lane roadway with a driveway density of 30 driveways per mile, a TWLTL would be cost effective over the range of left-turn percentages if the ADT is above 6,600 vpd. If the ADT is below 6,200 vpd a TWLTL would not be cost-effective in any case.

Thus, figure 2 provides guidelines for the cost-effective use of TWLTL medians on urban four-lane roadways. It should be noted that the left-turn percentage used in figure 2 is the combined percentage of the ADT that turns left from both directions. Also, in using figure 2 it must be remembered that

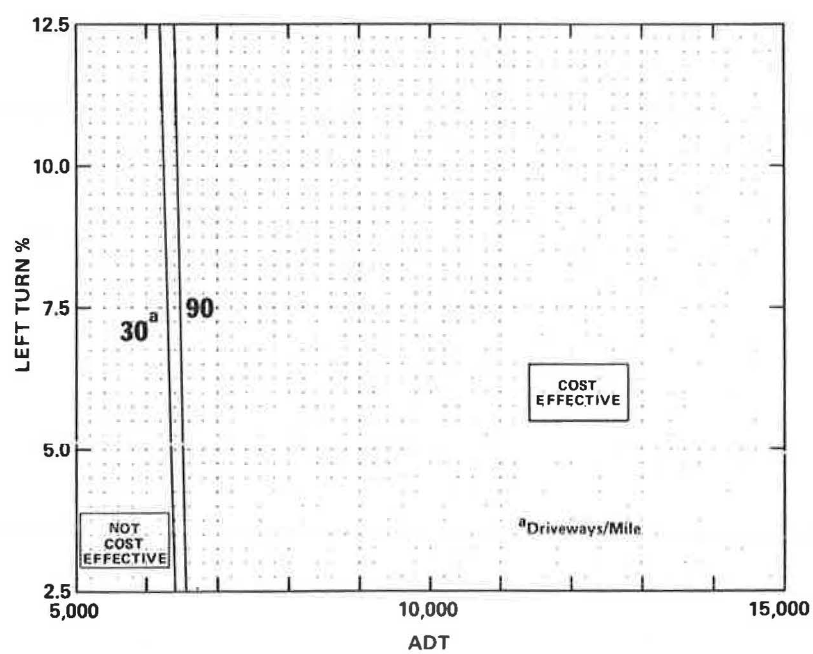


FIGURE 2 Cost-effectiveness of TWLTL based on total cost savings.

its development was based on conditions and costs that were intended to be representative of those on urban four-lane roadways in Nebraska during 1986. This figure is not applicable to cases in which the conditions and costs are substantially different. In such cases, the cost-effectiveness methodology presented elsewhere (*1*) should be used instead of figure 2 to determine the cost-effectiveness of TWLTL medians.

Because of the effects of driveway density found in the computer simulation study reported elsewhere (*1*), TWLTL medians are shown in figure 2 to be cost-effective at lower ADTs on roadways with lower driveway densities than they are on roadways with higher driveway densities. The reductions in stops and delays provided by a TWLTL were all found to be lower as driveway density increased. This was because in the computer simulation the left-turn volume was apportioned equally among the driveways. Therefore, the left-turn volume per driveway at 30 driveways per mile was two and three times greater than it was at 60 and 90 driveways per mile, respectively. Consequently, more queuing of left-turn vehicles would tend to occur at 30 driveways per mile; and, at 60 and 90 driveways per mile, vehicles waiting to turn left at several driveways would be more likely to turn left through the same gap in the oncoming traffic stream.

Assuming equal left-turn volume per driveway may result in an understatement of the benefits provided by TWLTL medians. If the left-turn volume had not been apportioned equally among the driveways, multiple use of gaps would have occurred less frequently. Less-frequent multiple use of gaps would have increased the stops and delays experienced by traffic on the four-lane undivided roadways, which, in turn, would have increased the operational cost savings provided by TWLTL medians.

Operational Cost Savings

The conditions for which TWLTL medians are cost-effective based solely on operational cost savings are shown in figure 3. Over the range of left-turn percentages considered, TWLTL medians are not cost-effective under any conditions on urban four-lane roadways with ADTs below 10,500. Conversely, TWLTL medians are cost-effective solely on the basis of operational cost savings on urban four-lane roadways with ADTs above 16,200.

TWLTL medians provide greater operational cost savings on roadways with higher left-turn volumes. Therefore, as shown in figure 3, TWLTL medians are cost effective at lower ADTs on roadways with higher left-turn percentages at a given driveway density. For example, at 30 driveways per mile, the minimum ADT at which TWLTL medians are cost-effective ranges from 10,800 on roadways with 12.5% left turns to 14,400 on roadways with only 2.5% left turns. Also, as explained earlier, TWLTL medians provide greater operational cost savings on roadways with lower driveway densities. Therefore, as shown in figure 3, TWLTL medians are cost-effective at lower ADTs on roadways with lower driveway densities, at a given left-turn percentage. For example, at 7.5% left turns, the minimum ADT at which TWLTL medians are cost-effective ranges from 12,200 on roadways with 30 driveways per mile to 13,700 on roadways with 90 driveways per mile.

Accident Cost Savings

The combinations of ADT and left-turn percentage for which TWLTL medians on urban four-lane roadways are cost-effective solely on the basis of the accident cost savings are shown

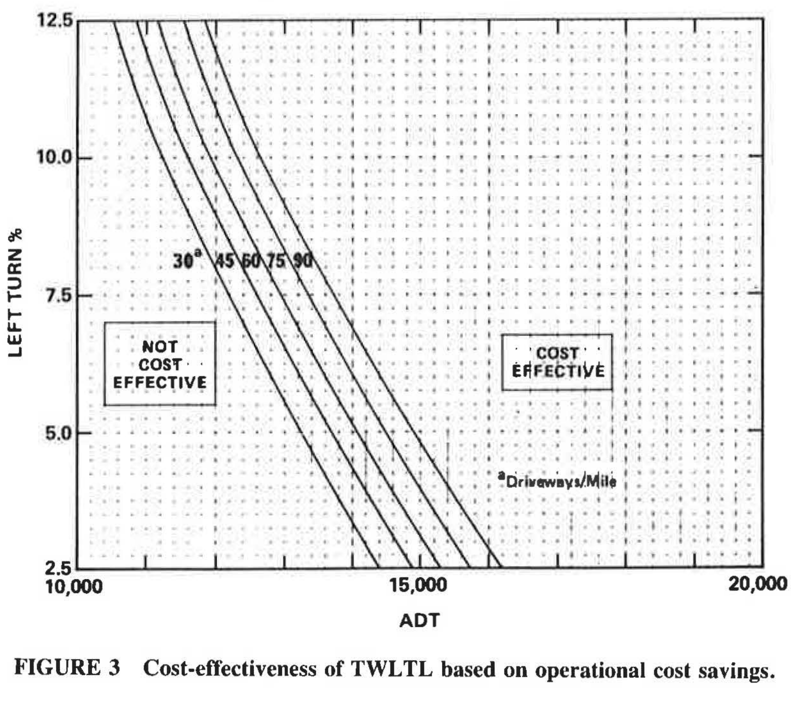


FIGURE 3 Cost-effectiveness of TWLTL based on operational cost savings.

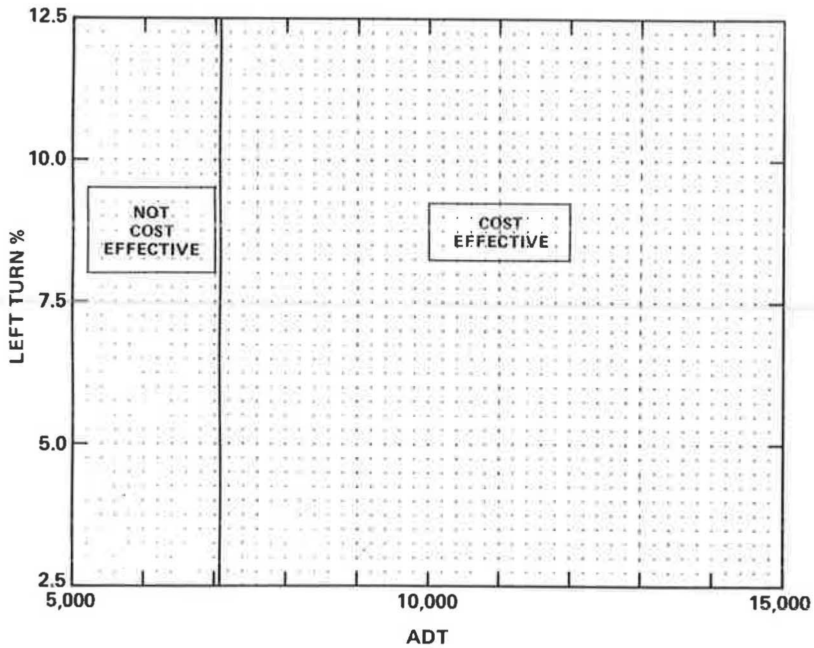


FIGURE 4 Cost-effectiveness of TWLTL based on accident cost savings.

in figure 4. Based on accident cost savings alone, TWLTL medians are cost-effective at ADTs above 7,100 vpd, regardless of left-turn percentage or driveway density.

CONCLUSIONS

Based on the results of the cost-effectiveness analysis, the following conclusions were reached with respect to the provision of TWLTL medians on urban four-lane roadways in Nebraska:

- The ADTs at which TWLTL medians are cost-effective depend on the left-turn percentage and driveway density on the roadway. TWLTL medians are cost-effective at lower ADTs on roadways with higher left-turn percentages and fewer driveways per mile. The minimum required ADT ranges from 6,200 to 6,600 vpd, depending on the left-turn percentage and driveway density.
- On the basis of operational cost savings alone, the minimum ADT required for TWLTL medians to be cost-effective ranges from 10,500 to 16,200 vpd, depending on the left-turn percentage and driveway density.
- On the basis of accident cost savings alone, TWLTL medians are cost-effective at ADTs above 7,100 vpd, regardless of left-turn percentage or driveway density.

However, in using the guidelines presented in this paper, it must be remembered that they were developed based on accident experience, traffic conditions, road-user costs, and TWLTL costs that were considered representative of urban four-lane roadways in Nebraska during 1986. Thus, on urban four-lane roadways with higher than average accident rates, truck percentages, and/or peak-hour volumes, TWLTL medians would be cost-effective at ADTs lower than those indicated by these guidelines. Conversely, on urban four-lane roadways with lower than average accident rates, truck percentages, and/or peak-hour volumes, TWLTL medians would be cost-effective only at higher ADTs than indicated by these guidelines. In addition, the use of different road-user costs and TWLTL costs would also change the results of this analysis. Higher road-user costs and lower TWLTL costs would reduce the minimum ADTs at which TWLTL medians are cost-effective. On the other hand, lower road-user costs and higher TWLTL costs would increase these ADTs. Therefore, in cases where the conditions and/or costs differ substantially from those used in developing these guidelines, the cost-effectiveness methodology presented elsewhere (*1*) should be used instead of these guidelines to determine the cost-effectiveness of TWLTL medians.

Finally, it must be remembered that factors other than cost-effectiveness must also be considered before making the final decision on the installation of TWLTL. Even though a TWLTL may be evaluated as being cost-effective, other factors may indicate that it is not appropriate in a particular situation. Previous research, experience, and opinions of others (*7, 8, 14, 15, 16, 17*) have indicated that TWLTL medians are not appropriate on streets with the following characteristics: (a) little conflict between left-turn and through movements (b) major-arterial street classification, (c) low driveway density, (d) short intersection spacing, (e) potential for interlocking left-turn movements between access points, (f) inadequate sight distance, (g) high pedestrian volumes, (h) few accidents associated with left-turn maneuvers, and (i) adequate indirect left-turn access. Thus, application of the guidelines must be tempered with engineering judgment.

ACKNOWLEDGMENTS

This is the final report of project HPR 82–3, "Guidelines for Use of Two-Way Left-Turn Lanes." The research was conducted by the Civil Engineering Department, University of Nebraska-Lincoln in cooperation with the Nebraska Department of Roads and the U.S. Department of Transportation, Federal Highway Administration. The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Nebraska Department of Roads or the Federal Highway Administration.

REFERENCES

1. P. T. McCoy and J. L. Ballard. *Cost-Effectiveness Evaluation of Two-Way Left-Turn Lanes on Urban Four-Lane Roadways*. Research Report No. TRP – 02-16-86. University of Nebraska-Lincoln.
2. *1985 Continuous Traffic Count Data and Traffic Characteristics on Nebraska Streets and Highways*. Nebraska Department of Roads, Lincoln, Neb., May 1986.
3. M. R. Hoffman. Two-Way, Left-Turn Lanes Work. *Traffic Engineering*, Vol. 44, No. 11, August 1974, pp. 24–27.
4. J. C. Glennon et al. *Technical Guidelines for the Control of Direct Access to Arterial Highways*; *Volume I: General Framework for Implementing Access Control Techniques*, Report No. FHWA-RD-76-86; *Volume II: Detailed Description of Access Control Techniques*, Report No. FHWA-RD-76-87. Federal Highway Administration, August 1975.
5. Continuous Left-Turn Lanes Reduce Accidents. *AASHTO Quarterly*, Vol. 58, No. 1, January 1979, p. 26.
6. J. S. Thakkar. Study of the Effect of Two-Way Left-Turn Lanes on Traffic Accidents. In *Transportation Research Record 960*, TRB, National Research Council, Washington, D.C., 1983, pp. 27–33.
7. ITE Technical Committee 5B – 4. Effectiveness of Median Storage and Acceleration Lanes for Left-Turning Vehicles. *ITE Journal*, Vol. 55, No. 3, March 1985, pp. 20–25.
8. D. W. Harwood. *NCHRP Report 282: Multilane Design Alternatives for Improving Suburban Highways*. TRB, National Research Council, Washington, D.C., 1986.
9. R. B. Sawhill and D. R. Neuzil. Accidents and Operating Characteristics on Arterial Streets with Two-Way Median Left-Turn Lanes. In *Highway Research Record 31*, HRB, National Research Council, Washington, D.C., 1963, pp. 20–56.
10. *Mean Accident Rates for State Highway Sections*. Nebraska Department of Roads, Lincoln, Neb., July 1986.
11. *A Manual on User Benefit Analysis of Highway and Bus-Transit Improvements*. American Association of State Highway and Transportation Officials, Washington, D.C., 1977.
12. *Highway Capacity Manual, 1965*. HRB, Special Report 87, 1965.
13. *Consumer Price Index Report*. U.S. Department of Labor, Bureau of Labor Statistics, Washington, D.C., 1975–present.
14. Z. A. Nemeth. Two-Way Left-Turn Lanes: State-of-the-Art Overview and Implementation Guide. In *Transportation Research Record 681*, TRB, National Research Council, Washington, D.C., 1978, pp. 62–69.
15. C. M. Walton and R. B. Machemehl. Accident and Operational Guidelines for Continuous Two-Way Left-Turn Median Lanes. In *Transportation Research Record 737*, TRB, National Research Council, Washington, D.C., 1979, pp. 43–54.
16. M. R. Parker, Jr. *Methodology for Selecting Urban Median Treatments: A User's Manual*. VHTRC 82-R3, Virginia Highway and Transportation Research Council, Charlottesville, July 1981, Revised March, 1982.
17. ITE Technical Committee 4A – 2. Design and Use of Two-Way Left-Turn Lanes. *ITE Journal*, Vol. 51, No. 2, February 1981, pp. 30–37.

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

Safety Effects of Cross-Section Design for Two-Lane Roads

CHARLES V. ZEGEER, DONALD W. REINFURT, JOSEPH HUMMER, LYNNE HERF, AND WILLIAM HUNTER

The purpose of this study was to determine the effect on accidents of lane widening, shoulder widening, and shoulder surfacing. Detailed traffic, accident, roadway, and roadside data were collected on 4,951 miles of two-lane roadway in seven states. Statistical testing was used along with an accident prediction model to determine the expected accident reductions related to various geometric improvements. Accident types found to be most related to cross-section features included run-off-road, head-on, and sideswipe (same direction and opposite direction) accidents. The roadway variables found to be associated with a reduced incidence of these related accident types (and included in the predictive model) are wider lanes, wider shoulders (paved slightly safer than unpaved), better roadside condition, flatter terrain, and lower traffic volume. Lane widening was shown to reduce related accidents by 12 percent for 1 foot of widening (for example, 10-foot lanes to 11-foot lanes), 23 percent for 2 feet of widening, 32 percent for 3 feet of widening, and 40 percent for 4 feet of widening. The effects of shoulder widening on related accidents was determined for paved and unpaved shoulders. For shoulder widths between zero and 12 feet, the percent reduction in related accidents due to adding paved shoulders is 16 percent for 2 feet of widening, 29 percent for 4 feet of widening, and 40 percent for 6 feet of widening. Accident reductions due to adding unpaved shoulders were slightly less than for paved shoulders.

In the U.S. today, there are an estimated 3.1 million miles of rural two-lane highways, which represent 97 percent of the rural mileage and 80 percent of all highway miles. Approximately 80 percent of rural two-lane roads have an average daily traffic (ADT) of less than 400, while 38 percent have an ADT of less than 50. Rolling terrain accounts for 58.9 percent of rural two-lane roads, with 31.5 percent on flat terrain, and 9.6 percent in mountainous areas.

For the two-lane rural highway system, 32.5 percent has 10-foot lane widths, 40.5 percent has 11- to 14-foot lane widths, and 27 percent has lane widths of 9 feet or less. Only 16.2 percent of rural two-lane highways have shoulder widths of 7 feet or more, with 47.8 percent having shoulder widths of 3 to 6 feet, and 36.1 percent with shoulders of 2 feet or less. Only 12.4 percent of rural two-lane roads have paved shoulders (*1*).

In recent years, there has been increased concern by highway officials and the public regarding the deterioration of the U.S. highway network, particularly on two-lane rural roads. Efforts have continued by highway agencies to maintain the structural integrity of highways through various improvement programs such as 3R (resurfacing, restoration, and rehabilitation). Considerable controversy has resulted regarding the effects of such pavement maintenance activities on highway safety and the most appropriate designs for improved roadways.

C. V. Zegeer, D. W. Reinfurt, and W. Hunter, Highway Safety Research Center, University of North Carolina, CB 3430 Craige Trailer Park, Chapel Hill, N.C. 27514. J. Hummer, Department of Civil Engineering, Purdue University, West Lafayette, Ind. 47907. L. Herf, Traffic Engineering Department, City of Southfield, Mich. 48076.

Faced with upgrading the existing two-lane rural highway system, highway officials need accurate information on the relationships between accidents and various geometric and roadside designs. Previous research studies have reported widely differing results, and little is known about the combined effects of both geometric and roadside features on accident frequency and severity. Thus, there is a need to better quantify the effects on accidents of alternative geometric and roadside designs. In addition, there is a need to develop a method for estimating accident-related benefits that would result from various roadway improvements on two-lane rural roads.

The major objective of this study was to determine the effects of lane width, shoulder width, and shoulder type on accidents. Then, based on these effects, the safety benefits of 3R improvements should be quantified relative to improvements to lanes and shoulders. This paper was based on a research study performed jointly for the Federal Highway Administration and the Transportation Research Board (*2*) and uses data collected in Alabama, Michigan, Montana, North Carolina, Utah, Washington, and West Virginia. More details of the safety effects of roadside features from that report are presented in a companion paper in this Record (*3*).

BACKGROUND

More than thirty articles and reports were critically reviewed relative to the safety effects of lane and shoulder width and shoulder type. Specific criteria were used to determine the major strengths and weaknesses of each source, including data sample size, adequate data detail, possible data errors, data biases, use of adequate control variables, proper analysis assumptions, accident types used (for example, run-off-road, head-on), appropriate analysis techniques, and proper interpretation of results. Basic principles outlined in the Federal Highway Administration's "Accident Research Manual" (*4*) and a User's Manual on "Highway Safety Evaluation" (*5*) were also considered in the critical review.

Initial review of the literature found major flaws in many of the accident studies, and only nine of them survived preliminary screening. Of these nine, a study by Rinde (*6*) dealt with shoulder widening, while studies by Dart and Mann (*7*), Shannon and Stanley (*8*), and Zegeer, Mayes, and Deen (*9*) involved analyses of both lane and shoulder widths. Studies by Heimbach, Hunter, and Chao (*10*), Turner et al. (*11*), and Rogness, et al. (*12*) involved an analysis of shoulder type, while studies by Foody and Long (*13*) and Jorgensen (*14*) analyzed lane width, shoulder width, and shoulder type.

The studies by Rinde (*6*) and Rogness, et al. (*12*) were before-and-after studies of completed shoulder widening projects in which the authors controlled for external factors. The remaining seven studies were comparative analyses that developed accident relationships with one or more geometric variables. Of these seven, three used regression analysis to develop predictive accident models.

To select the most reliable and complete information available, data and information from the nine studies were carefully analyzed. Data that covered a wide range of lane- and shoulder-width and shoulder-type combinations were desired. Also, data showing accident experience for the specific accident types most related to lane and shoulder deficiencies were considered most useful.

Although no satisfactory quantitative model was found within the published literature relating accident rate to various lane and shoulder conditions, prior research has established the general effects of these elements on highway accidents. Qualitatively, these effects can be summarized as follows:

- Lane and shoulder conditions directly affect run-off-road (ROR) and opposite direction (OD) accidents. Other accident types, such as rear-end and angle accidents, are not directly affected by these elements.
- Rates of ROR and OD accidents decrease with increasing lane width. However, the marginal effect of lane-width increments is diminished as either the base lane width or base shoulder width increases.
- Rates of ROR and OD accidents decrease with increasing shoulder width. However, the marginal effects of shoulder-width increments are diminished as either the base lane width or base shoulder width increases.
- For lane widths of 12 feet or less, each foot of lane widening has a greater effect on accident rates than an equivalent amount of shoulder widening.
- Non-stabilized shoulders, including loose gravel, crushed stone, raw earth, and turf, exhibit greater accident rates than stabilized (such as tar with gravel) or paved (such as bituminous or concrete) shoulders.

These qualitative relationships served as the basis for developing a quantitative accident model from previous literature, as given in detail in a publication by Zegeer and Deacon (*15*).

PLANNING AND COLLECTION OF DATA

Analysis Issues

Prior to deciding the types and amount of data to be collected, a clear understanding was needed of the specific analysis issue of concern. The key issue addressed in this study was determining the relationships between accidents and various combinations of lane width, shoulder width and shoulder surface types on two-lane roads. In addressing this analysis issue, there was a need to first determine what traffic, roadway, and roadside variables have a significant influence on accidents. Then, appropriate mathematical models could be developed to predict accident experience as a function of related traffic and roadway variables. Such models would enable estimation of the expected accident reduction for improvements on two-lane roads such as lane widening, shoulder widening, and shoulder surfacing for various traffic and roadway conditions. For this analysis, there was also a need to develop measures, ratings, or hazard scales that could be used to quantify roadside characteristics for purposes of data collection, analysis, and improvement considerations.

Study Design

As discussed previously, the key issue of this study is aimed at determining the effects of various combinations of lane width, shoulder width, and shoulder surface type on accident experience. Two basic analysis approaches were considered for addressing this issue:

1. A before-and-after study with control sites.
2. Modeling the relationships between accidents and various combinations of geometric and roadside conditions (to control for numerous factors that may affect the results).

While the before-and-after-with-control-site analysis may be used for determining countermeasure effectiveness in some cases, numerous problems prevented its use in this study. First, sites with each of the cross-sections of interest in this study would have to be found for numerous traffic and highway conditions in each of several states. Furthermore, projects would have to be found for which no other improvements were made. This would have been unlikely, since many widening projects, for example, are done in conjunction with pavement resurfacing along with such improvements as drainage, resurfacing, delineation, and/or bridge improvements. Also, control sites (in other words, sites similar to the project sites for which no improvements were made) are needed to minimize data biases. Since suitable control sites and project sites would have been difficult to find, the use of the before-and-after-with-control-site analysis was considered to be impractical for use in this study.

The use of mathematical accident predictive models does not utilize accident data before and after projects were implemented. Instead, they can be used to develop relationships between accidents and the traffic and roadway features of concern. This type of analysis does not rely on locating suitable project and control sites but is based instead on a large sample of randomly selected roadway sections. However, care must be exercised to collect and control for the variables that have important effects on accident experience in addition to the variables of interest. It should also be mentioned that nearly all of the major accident research studies on roadway geometrics use some form of accident modelling instead of before-and-after-with-control-site experimental designs. For this study, accident predictive models were used to determine the effects of various geometric and roadside improvements on accidents.

Selection of Data Variables

Accident experience on rural highways is a complex function of many factors including those associated with physical aspects of the roadway, as well as a multitude of other factors related to driver, vehicle, traffic, and environmental conditions. One 1978 study estimated that at least 50 roadway-related features could have an effect on accidents (*14*). However, in typical accident analyses, there are often relatively few important traffic and roadway variables that individually show significant relationships with accidents.

The selection of variables for use in this study was based on a literature search of past research to determine the ones that have been shown to be most important on two-lane roads in rural, suburban, or urban areas. The collection of every possible roadway, traffic, and accident variable would have been both unnecessary and impractical.

For each of the selected roadway sections, the following traffic and roadway variables were collected:

- Section information (section identification, length, pavement type, terrain, ditch type, area type, type of development, speed limit)
- Average annual daily traffic (AADT)
- Speed limit
- Horizontal curvature (seven different data variables indicating percent of the section within curvature groups of >2.5 degrees, 2.5 degrees, >5.5 degrees, 7.0 degrees, >14.0 degrees, 19.0 degrees, and >28.0 degrees). Horizontal curve data were not available for some sections.
- Vertical grade (four different data variables indicating the percent of the section with percent grade of >2.5 percent, >2.5 percent, >4.5 percent, and >6.5 percent). Vertical grade data were not available for some sections.
- Sideslope ratio (two to one or steeper, three to one, four to one, five to one, six to one, or seven to one or flatter).
- Width of lanes and shoulders and shoulder type (such as paved, stabilized, gravel, earth, or grass).
- Number of bridges, intersections (by type of sign or signal control), overpasses, railroad crossings, driveways (by type residential, commercial, recreational, or industrial setting).
- Type of delineation and on-street parking.

Since the roadside condition is known to be an important factor related to accidents, a roadside hazard scale was developed based on the literature review and the results of a workshop involving thirteen highway and roadside safety professionals. The roadside hazard rating developed for this study was a subjective measure of the hazard associated with the roadside environment. The rating values indicated the accident likelihood and damage expected to be sustained by errant vehicles on a scale from one (low likelihood of an off-roadway collision or overturn) to seven (high likelihood of an accident resulting in a fatality or severe injury). The ratings were determined from a seven-point pictorial scale, as illustrated in figure 1 for rural highways. The data collectors chose the rating value (one through seven) that most closely matched the roadside hazard level for the roadway section in question. In many cases, the roadside hazard along a section varied considerably, so the roadside hazard rating should represent a "middle"

ROADSIDE HAZARD RATING OF 1

ROADSIDE HAZARD RATING OF 3

ROADSIDE HAZARD RATING OF 5

ROADSIDE HAZARD RATING OF 7

FIGURE 1 Examples of pictorial ratings from the roadside hazard rating scale.

value (for example, if ratings generally range from four to six along a section, a rating of five would best represent the roadside hazard rating of the section). In addition to the subjective roadside hazard rating, a measure termed "roadside recovery distance" was also developed and collected for each section along with detailed data on roadside obstacles by type and distance from the roadway. Details of these measures and the resulting analysis are given elsewhere (*2, 3*).

Accident Variables

For most of the selected roadway sections, accident data were collected from the state computer records for a 5-year period. For approximately 5 percent of the roadway sections, accident data for 2 to 3 years were used, to exclude time periods when roadway characteristics changed or when accident data were not readily available. Non-uniform variables and definitions among the seven states had to be considered in redefining the accident variables for the analysis. While dozens of accident variables could have been chosen, only those necessary for the analysis were selected.

For each roadway section, the accident information collected included number of years of accident data (5 years in most cases); total number of accidents on the section; number of accidents by severity category (property damage only, A-injury, B-injury, C-injury, and fatal); number of people injured (by injury level) and killed; number of accidents by light condition and pavement condition; number of accidents by type (fixed object, rollover, other run-off-road, head-on, opposite direction sideswipe, same direction sideswipe, rear end, backing or parking, pedestrian or bike or moped, angle or turning, train related, animal related, other or unknown); and number of accidents by type of fixed obstacle struck.

Site Selection

To fulfill the study objectives, sites were desired in states that covered a variety of geographic characteristics, climatic conditions, roadway designs, terrain conditions, traffic conditions, and other factors. Also, states were desired that had reasonably low accident reporting thresholds (for example, $500 or less per accident) to minimize inconsistencies among states in reporting property damage accidents. States were also desired that had accurate computer accident data for five or more years with accident data items of interest (such as accident type, severity, accurate locational information, etc.). States must also have accurate and current traffic volume (ADT) data, roadway inventory information, and photolog film (for collecting roadside and other information). The seven states chosen for data collection were Alabama, Michigan, Montana, North Carolina, Utah, Washington, and West Virginia.

A sample of 4,951 miles of two-lane roads was selected from the seven states, which was considered to be more than adequate for meaningful analysis and for accident modeling purposes. Only two-lane roadway sites were selected, and section lengths ranged from 1 to 10 miles in rural areas and from 0.5 to 5 miles in urban areas. Sections were selected that were relatively homogeneous throughout the section regarding basic geometric and operational features. For example, a section ended when ADT changed moderately, lane width changed by 1 foot or more, shoulder width changed by more than 3 or 4 feet, or a noticeable change occurred in the roadside condition.

Selecting from these categories also produced a variety of roadside conditions for analysis. Samples were selected only on state numbered or U.S. numbered routes, since accident data was found to be more accurate and complete on those systems than on local road systems.

Data Collection

Data Sources

The data sources for the accident analysis included field data collection, photologs, state agency records (such as maps, ADT listings, computerized roadway inventories), police accident records (either computer accident tapes or computer accident summaries), and the Highway Performance Monitoring System (HPMS) computer database. Much of the roadside information was extracted from photologs including roadside data for individual obstacles, roadside hazard ratings, and measures of roadside recovery distance.

State records were used as a primary source for ADT data and vertical and horizontal curvature data for many of the sections (for example, non-HPMS sections). The HPMS database was used for initial site selection and also as a secondary source for ADT data and horizontal and vertical curvature data for much of the rural sample. Police accident records were the sources of all accident data in the seven states.

For many of the most important data elements, two or three sources were used for verification. For example, independent field measurements and photolog measurements were taken of sideslopes, lane width, shoulder widths and types, and cross-section design for much of the sample. For many data variables, the photolog measurements were the primary data source, but verification was carried out using state inventory data and/or HPMS data. Inconsistencies in measurements of key data variables were resolved and corrected.

Data Collection Techniques

Homogeneous roadway sections were identified from the HPMS data tape and from computerized state roadway inventories. Samples of approximately 500 to 1,000 miles were desired from each state. Sections were selected independently of accident data to avoid any accident bias of the database. Therefore, some zero-accident sections resulted. Stratified random sampling was used to select an adequate sample of sections within certain needed categories of ADT, lane width, and shoulder width and type. This was necessary since a database of nearly all 11- and 12-foot lanes, for example, would not allow for determining the effects of various lane widths (for example, 9 to 12 feet) on accidents.

Detailed roadside data and roadway information were recorded from state photologs. The photologs were 35mm photographs taken from a moving vehicle in equal distances of 100 frames per mile (52.8 feet between frames). Location information was given at the bottom of each file frame and typically included route number, milepost, county, direction of travel, and date of filming. Teams of technicians viewed frames consecutively for preselected sections and recorded information directly onto data forms. Three data forms used with photolog film included those of basic roadway data, cross-section data, and detailed roadside obstacle data. For data involving lane and shoulder widths and lateral placement of roadside obstacles, a calibrated grid was placed over the photolog viewing screens for each photolog frame. This process allowed for coding of roadside recovery distance for each 0.1 mile for each roadway section (both sides of the road).

Creation of the Database

Close data quality control was practiced throughout the data collection process. All data were double-keyed into a computer file. A series of programs was written, which read data for each section and checked—

- Each data variable against allowable lower and upper limits;
- The logic of accident totals (for example, total accidents had to equal PDO + injury + fatal);
- The computed accident rates by accident type; and
- The match of lane width, shoulder width, speed limit, area type, and other variables to ensure agreement for all data sources (HPMS, photolog, state records, and field measurements).

Data "outliers" were printed and corrected or deleted as necessary. The final data file contained 325 data variables for each roadway section. With 1,944 records (roadway sections) and 868 characters per record, the database consisted of 1.69 million data characters.

RESULTS OF DATA ANALYSIS

Database Characteristics

The database contained data for 4,785.14 miles of rural roadway (1,801 sections) and 166.14 miles of urban streets (143 sections), for a total of 4,951.28 miles (1,944 total sections). The average section length was 2.66 miles in rural areas and 1.16 miles in urban areas, or 2.55 miles overall. Data were collected on approximately 1,033 miles of roadway from Alabama, 699 miles from Michigan, 547 miles from Montana, 746 miles from North Carolina, 525 miles from Utah, 737 miles from Washington, and 665 miles from West Virginia.

Data were collected entirely on two-lane roads but covered a wide range of traffic and geometric conditions. Shoulder widths ranged from zero to 12 feet and lane widths varied from 8 to 14 feet. In terms of traffic volume, approximately half of the mileage (2,392 miles) had an ADT between 1,000 and 4,000, while only 387.7 miles (7.8 percent) had an ADT above 7,500, and 938.4 miles (19 percent) had ADTs of 750 or less.

It is clear that this data sample has higher traffic volume levels than those of the nationwide two-lane rural highway system. This was expected, since our sample was purposely taken on state-maintained (in other words, U.S. and state numbered routes), whose accident data accuracy was thought to be much better than that on local roads. However, as discussed below, this sampling procedure resulted in a data sample with accident rates very close to national samples as reported by Smith (*1*). Also, the effect of higher ADTs was accounted for in all of the accident predictive models, along with the other roadway variables of concern, such as lane width, shoulder width and type, and roadside condition. It should also be mentioned that the ideal data sample for this type of modelling analysis was not one that was truly representative of national distributions by ADT only, but instead covered the full range of traffic and roadway conditions in the United States, to the extent practical.

Of the 4,785 miles of rural highway, 4,119 miles (or 86 percent) had speed limits of 55 mph; 544.5 miles (11.4 percent) had speed limits of between 40 and 50 mph; and 121.6 miles (2.5 percent) were in built-up rural areas with speed limits of 25 to 35 mph. The predominance of 55 mph speed limits for sections in the rural databases prevented an in-depth analysis of the effects of speed limits on accident experience. Data were included from 1,946.7 miles in flat terrain, 2,134.0 miles in rolling terrain, and 870.5 miles in mountainous areas. The database also included sections with wide ranges of roadside conditions, sideslopes, curvature, and other factors.

General Accident Characteristics

There were 62,676 total reported accidents on sections in the database including 38,857 property-damage-only accidents (62.0 percent), 22,944 injury accidents (36.6 percent), and 875 fatal accidents (1.4 percent). A review of the accident data by type revealed that the most frequently reported accidents were angle and turning (23.5 percent), followed by rear end (19.8 percent), run-off-road fixed object (19.3 percent), animal (8.3 percent) and rollover (6.8 percent). The average accident rate was found to be 266.35 accidents per 100 million vehicle miles (mvm), or 3.69 accidents per mile of roadway per year.

Of the 1,944 sample sections in the database, 1,468 were from rural areas and the remaining 476 were from urbanized areas (areas with populations of 5,000 or more). Of those 476 sections, 143 were classified as having an urban appearance (designated as urban sections) by the data collectors and 333 appeared rural to the data collectors (designated as U/R sections). For purposes of the predictive model, only the "pure" rural sections were used (in other words, U/R and urban sections were excluded). Detailed analyses of urban sections and roadside characteristics are given elsewhere (*2*).

A summary of various accident statistics is given for the 1,801 rural sections and 143 urban sections in table 1. The average rate of total accidents was 603.18 per 100 mvm for urban sections, and 239.61 per 100 mvm for rural sections. There were 13.51 accidents per mile per year in urban areas, compared to 2.91 in rural areas. In both cases, the urban rate was greater than the rural rate. Higher traffic volumes, more frequent intersections, and denser roadside development are a few of the possible factors that may cause higher accident rates in urban areas than rural areas.

In terms of accident severity, injury accidents constituted 37.5 percent (20,008 of 53,358) of total accidents in rural areas, compared to 31.5 percent (2,936 of 9,318) in urban areas. Fatal accidents accounted for 1.57 percent of the accidents in rural areas and 0.41 percent in urban areas. Accident

TABLE 1 SUMMARY OF ACCIDENT STATISTICS FOR RURAL AND URBAN ROADWAY SECTIONS

Variable Name	No. of Accidents		Accs/100 MVM		Accs/Mi/Yr	
	Rural	Urban	Rural	Urban	Rural	Urban
Total Accs.	53,358	9,318	239.61	603.18	2.91	13.51
PDO Accs.	32,513	6,344	146.41	432.85	1.81	9.46
Injury Accs.	20,008	2,936	88.75	168.31	1.06	3.99
Fatal Accs.	837	38	4.45	2.02	0.04	0.05
People Injured	32,756*	4,565*	141.74*	262.53*	1.74*	6.62*
People Killed	1,016*	52*	5.26*	3.97*	0.05*	0.07*
Daylight Accs.	31,108	6,294	135.94	430.61	1.75	9.39
Dawn or Dusk Accs.	2,535	353	11.31	19.83	0.13	0.47
Dk. with Lights	1,863	907	6.78	62.49	0.12	1.34
Dk. w/o Lights	17,764	1,732	84.97	86.06	0.90	2.20
Unkn. Light Cond.	88	32	0.61	4.18	0.01	0.10
Dry Accs.	35,783	6,174	162.79	408.12	1.96	9.10
Wet Accs.	11,294	2,193	47.02	146.96	0.64	3.27
Snow/Ice Accs.	5,802	855	27.17	40.52	0.29	0.97
Unkn. Pvt. Accs.	479	96	2.63	7.58	0.03	0.16
ROR - Fixed Object	10,937	1,154	54.71	60.54	0.54	1.44
ROR - Rollover	4,122	123	25.91	6.72	0.18	0.14
ROR - Other	2,621	219	15.36	12.28	0.15	0.29
Head-On	1,858	255	8.01	13.41	0.10	0.32
Sideswipe - Opp. Dir.	2,628	369	12.55	19.89	0.15	0.50
Sideswipe - Same Dir.	1,925	363	8.74	29.82	0.12	0.59
Rear End	9,593	2,827	30.12	162.95	0.58	3.89
Parking	922	233	4.51	20.16	0.06	0.39
Ped./Bike/Moped	516	139	2.12	7.23	0.03	0.18
Angle & Turning	11,415	3,315	41.39	244.44	0.68	5.25
Train	43	4	0.32	0.18	0.002	0.004
Animal	5,068	144	26.80	7.34	0.22	0.19
Other or Unknown	1,710	173	9.08	18.22	0.10	0.33

*These variables represent the number of people injured or killed, and not the number of accidents.

rates were higher in rural areas than in urban areas for rollover, train, and animal accidents. Urban rates were higher for the remaining accident types, and particularly for angle and turning, parking, rear-end, and same-direction sideswipe accidents.

A detailed review of the distribution of the variables in the database was made to examine the quality of the data. The minimum, maximum, mean, and standard deviation were computed for selected variables. Lane widths ranged from 8 to 14 feet and shoulder widths varied from zero to 12 feet (11 feet for earth shoulders). There were an average of 2.35 intersections per mile (maximum of 11 on one section), 0.20 bridges per mile, and 0.21 other structures (such as overpasses) per mile. There were 13.77 driveways per mile on the average (total of both sides of the road) with a maximum of 81 per mile on one section. The number of total accidents per mile per year ranged from zero on some low-volume sections to 71.14 on one particularly high-volume section. There were an average of 0.94 single vehicle accidents per mile per year with a range from zero to 11.38. Extensive data checking was conducted, particularly to confirm the accuracy of the extremes. A comparison was made of accident rates between the seven-state database and previous accident studies. The FHWA study by Smith et al. of rural roads throughout the United States included accident rates and the percent of injury and fatal accidents for rural roads in many states by ADT group as shown in table 2 (*1*). Corresponding rates from the seven-state database revealed close similarities. For example, rates of total accidents (per 100 mvm) were similar for each ADT group, except for ADTs greater than 10,000, where the rate of 244 from the seven-state database was lower than the rate of 300 from the Smith study. This may be due to the low sample size (only 80 sections) in that ADT group in the seven-state database. Percentages of injury and fatal accidents also compared quite favorably for each ADT group.

Another comparison was made with the results of the 1979 Kentucky study on lane and shoulder widths by Zegeer, as shown in table 3 (*9*). Accident rates are given for total and single-vehicle accidents for lane widths of 7 to 13 feet. Total and single-vehicle accident rates were similar between the studies for the 10-, 11-, and 12-foot lane widths. For less than 10-foot lanes, the rates were slightly lower for the seven-state database for both total accidents and single-vehicle accidents. The differences are probably the result of wider shoulders in the seven-state database compared to the Kentucky sites for sections with 9-foot lanes. For 13-foot lane widths, the Kentucky database had a lower rate of single-vehicle accidents and a higher rate of total accidents than the seven-state database. This may be the result of smaller sample sizes or other site differences.

TABLE 2 COMPARISON OF RURAL ACCIDENT EXPERIENCE BY ADT GROUP FOR RURAL SEVEN-STATE DATABASE AND SMITH STUDY

Accident Measure	ADT Group					
	1-400	401-1,000	1,100-2,000	2,501-5,000	5,001-10,000	>10,000
Avg. Total Acc. Rate (Acc/100 MVM)	288(300)*	246(250)	228(230)	225(220)	257(250)	244(300)
Percent Fatal Accs.	2.4(2.5)	3.1(3.0)	1.9(3.0)	1.8(2.5)	1.2(2.0)	0.9(2.0)
Percent Injury Accs.	38.9(36)	39.3(37)	38.8(37)	35.8(36)	37.7(35)	39.4(35)
Percent PDO Accs.	58.7(61.5)	57.6(60)	59.2(60)	62.4(61.5)	61.1(63)	59.8(63)

*Values in parenthesis are from Smith study.

While the accident rates agree closely between the seven-state database and other studies, differences did exist in average accident frequencies. For example, an average of 2.91 total accidents per mile per year was found on rural roads in the seven-state database, compared to approximately one accident per mile per year reported for rural Kentucky roads (*9*). This difference was the result of considerably higher traffic volumes on the seven-state sample compared to the Kentucky data. Thus, in the model-building process, ADT was used as a control variable and the effects of the other important variables were determined as accurately as possible.

Determination of Important Variables

The next series of analyses was intended to provide input into the selection of variables for use in the model-building process. The final selection of variables for inclusion in the model was based on (1) which variables were logically related to accidents (lane width, shoulder width, shoulder type, and roadside conditions), (2) the Chi-square analysis, (3) stepwise linear regression, and (4) analysis of variance and covariance.

Accident Variables

A series of Chi-square analyses were conducted to determine the specific accident types that were most highly correlated with lane width, shoulder width, shoulder type (paved, gravel, or earth) sideslope, and roadside rating. The significance levels were 0.05 or less (95 percent confidence or higher) for many of the tests, due primarily to large sample sizes but not necessarily to strong correlations. Thus, the

TABLE 3 COMPARISON OF ACCIDENT RATES BETWEEN KENTUCKY STUDY AND RURAL SEVEN-STATE DATABASE

Lane Width (feet)	Rate of Single Vehicle Accidents (Acc/100 MVM)		Rate of Total Accidents (Acc/100 MVM)	
	Kentucky Study	Seven-State Study	Kentucky Study	Seven-State Study
7	196 (396)	-	416 (396)	-
8	185 (2,808)	174 (28)	366 (2,808)	369 (28)
9	155 (8,249)	130 (711)	303 (8,249)	283 (711)
10	127 (2,537)	130 (907)	287 (2,537)	300 (907)
11	74 (788)	75 (1,438)	206 (788)	218 (1,438)
12	63 (610)	76 (1,406)	197 (610)	211 (1,406)
13	51 (38)	95 (294)	217 (38)	174 (294)

Numbers in parentheses represent mileage of samples in each cell.

contingency coefficient (which takes sample sizes into consideration) was used as the primary measure of association between the geometric elements of concern and the specific accident types. A matrix of contingency coefficients was produced during the series of Chi-square tests for various accident types and roadway features. Contingency coefficients of 0.220 were found to differentiate the upper third of the contingency coefficients in this analysis. The accident types that consistently appeared to be highly correlated with the roadway features of concern were single vehicle (fixed object, rollover, and other run-off-road accidents), head-on, and sideswipe (opposite-direction and same-direction) accidents. Single-vehicle, total, and some types of multi-vehicle accidents were found to be strongly associated with one or more of the roadway variables. On the other hand, animal, parking, angle and turning, and other or unknown accidents were not highly correlated with the roadway variables of concern. Insufficient samples of pedestrian and train accidents were available for these analyses.

Based on the results discussed above and a review of accident rates and trends for various accident types, the accident types thus considered to be most appropriate and logical for use in a predictive model were—

- Single-vehicle (fixed-object, rollover, and other run-off-road) accidents and
- Related multi-vehicle (head-on, opposite-direction sideswipe, and same-direction sideswipe) plus single-vehicle accidents.

Total accidents were found to be a reasonably strong measure of the overall effects of traffic roadway variables.

Traffic and Roadway Variables

The most important traffic and roadway variables for use in an accident predictive model were determined. Since many of the geometric variables in the database were interrelated or were derivations of the same variable, only one form of each variable was considered for use in the predictive model. Relationships were determined between accidents and individual traffic and roadway variables, as given in the full report (*2*). Accident relationships with individual variables were somewhat misleading, due to strong interactions between some roadway variables in terms of their combined effect on accidents. For example, narrow lanes and high roadside hazards were associated with higher accident experience than wide lanes or safe roadsides. However, roads with narrow lanes were found to often have a high roadside hazard rating as well. Also, roads with wide lanes are more likely to have reasonably safe roadsides than roads with narrow lanes. Thus, a review of the simple relationship between lane width and accidents gives a distorted picture, since roadside condition and other factors are also interacting to affect accidents unless they are controlled for in the analysis.

Chi-square analysis, stepwise linear regression, and analysis of variance and covariance were also conducted to infer accident relationships. The following traffic and roadway variables were found to be most highly related to accident experience and thus were used as candidate independent variables for modeling purposes:

- Average daily traffic (ADT)
- Lane width (W) in feet
- Average paved shoulder width (PA) in feet
- Average unpaved (gravel/stabilized/earth/grass) shoulder width (UP), in feet
- Median roadside (or hazard) rating (H)
- Median sideslope rating (SS)
- Terrain (TER)
- Percent of sections with >2.5 degree curves ($CURV$)
- Percent of sections with >2.5 percent grade ($GRAD$)
- Number of driveways per mile ($NDRI$)
- Number of intersections per mile ($NINT$)
- Certain derived variables (for example, $W + PA$)
- Selected interactions

DEVELOPMENT AND TESTING OF PREDICTIVE MODEL

Guided both by the previous literature (*15*) and an examination of the relationships of the important independent variables with various accident types, models were fit to the following (*15*):

- Single-vehicle accidents (AS) including fixed object, run-off-road rollover, and other run-off-road.
- Single vehicle plus opposite direction head-on, opposite direction sideswipe, and same direction sideswipe (AO).
- Total accidents (AT).

Of the 32,417 accidents on the 1,362 rural sections, 13,105 or 40.4 percent were AS (single vehicle) while 17,155 or 52.9 percent were related (AO) accidents.

Again guided by past work (*15*), several general model forms were investigated, including

$$A/M/Y = C_0(\mathrm{ADT})^{C_1}(C_2)^{W}(C_3)^{PA}(C_4)^{UP}(C_5)^{H} \quad \text{(Model 1)}$$

$$= C_0(C_1)^{\mathrm{ADT}}(C_2)^{W}(C_3)^{PA}(C_4)^{UP}(C_5)^{H} \quad \text{(Model 2)}$$

$$= C_0 + C_1\mathrm{ADT} + C_2W + C_3PA + C_4UP + C_5H \quad \text{(Model 3)}$$

$$= C_0(\mathrm{ADT})^{C_1}(W)^{C_2}(PA)^{C_3}(UP)^{C_4}(H)^{C_5} \quad \text{(Model 4)}$$

where:

$$A/M/Y = \text{accidents per-mile-per-year} = \frac{A}{L \times T}$$

with:

A = number of accidents on highway section

L = section length (miles)

T = number of years of accident data

C_0, C_1, C_2, C_3, C_4, and C_5 are constants, and ADT, W, PA,

UP, and H are as given previously. Model forms consist of the basic model equation without numerical coefficients (in other words, only C_i), whereas the equations will include numerical coefficients. The above models were tested using AS, AO, and AT accidents per mile per year. In addition to models with only main effects, several models with interaction terms were also evaluated. These interaction terms included (lane width × paved shoulder width) and [(lane width + paved shoulder width) × unpaved shoulder width]. In no case did the interaction terms noticeably improve upon the main effects models, and most often the interaction term coefficients were insignificant. Thus, the final models contain only main effects variables.

In all cases tested, Models 1 and 2 fit the data better than Models 3 or 4, and, also, coefficients of Models 1 and 2 were more reasonable. Although Model 2 seemed to fit the data slightly better than Model 1 on the basis of the R^2 values (which indicate the proportion of the total variation explained by the model), in some cases the relative effects of W, PA, and UP were not as reasonable. For example, for single-vehicle accidents using Model 2, the effects of PA and UP (paved and unpaved shoulders) are more important than W (lane width). This finding, in addition to the fact that the R^2 values were not much different between Models 1 and 2, led to the selection of Model 1 as the recommended model form.

All models utilized ADT as an independent variable because it was highly correlated with accidents per mile per year. ADT is a measure of exposure that has been shown in the literature to have a relatively high correlation with accident frequency in most situations. Basic cross-section elements lane width (W), paved shoulder width (PA), and unpaved shoulder width (UP) were also included in every model, and their individual effects were significant. Other primary variables examined were median roadside rating (H), median sideslope rating (SS), and other measures of roadside condition. In addition, certain likely confounding variables were studied including terrain (TER), percent of section with ≥2.5 degree curves ($CURV$), percent of section with ≥2.5 percent grades ($GRAD$), number of driveways per mile ($NDRI$), and number of intersections per mile ($NINT$).

It should be noted that a variety of models were examined that used alternative definitions of the cross-section variables (for example, one model using ADT, W, $(W + PA)$, $(W + PA + UP)$, and $(W + PA + RECC)$ and another using W^2). In no case did models with these various alternatives fit the data as well as the original model form or provide coefficients as intuitively acceptable as those derived for models with simpler variables.

Final Models

A series of models was produced that best fit various accident types (AS, AO, and AT), using lane width, width of paved shoulder, width of unpaved shoulder, ADT, and roadside hazard rating. Values of R^2 ranged between 0.39 and 0.46. In examining the effects of other potentially confounding variables, models incorporating terrain were found to be useful in further enhancing the models.

Thus, although several models were found to be acceptable, the final selected model is as follows:

$$\begin{aligned} AO/M/Y = {} & 0.0019\,(\mathrm{ADT})^{0.8824}\,(0.8786)^{W}\,(0.9192)^{PA} \\ & \times (0.9316)^{UP}\,(1.2365)^{H}\,(0.8822)^{TER1} \\ & \times (1.3221)^{TER2} \end{aligned}$$

where TER1 = 1 if flat, 0 otherwise; $TER2$ = 1 if mountainous, 0 otherwise and ADT, W, UP, PA, and H are as given previously. The R^2 value for this model was 0.456, or 45.6 percent of the variation in accidents was explained by the traffic and roadway variables. The relative contribution of each variable to this explained variation was 70.2 percent by ADT, 8.6 percent by W (lane width), 1.7 percent by PA (paved shoulder width), 10.5 percent by UP (unpaved shoulder width), 7.2 percent by H (roadside hazard rating), and 1.8 percent by TER (terrain).

This model was selected because (1) it included the accident types found to be most related to cross-sectional features (head-on, sideswipe, and single-vehicle accidents), (2) the coefficients appear to be reasonable and consistent with the literature, (3) it had a relatively high R^2 value, and (4) terrain effects (flat, rolling, or hilly) are incorporated into the model. Models using accident rates (such as Accidents/100 mvm or single-vehicle accidents per 100 mvm) were calibrated in parallel to those for accidents per mile, per year. In general, the R^2 values were considerably lower for models using Accidents/100 mvm. Details of these and other models are given elsewhere (*2*).

Model validation was performed on single-vehicle accidents using 75 percent of the data, which were randomly selected. The average deviation between the observed and predicted accidents per mile, per year was 0.36. Since the average single-vehicle accident rate for all 1,362 rural sections was 0.73 accidents per mile, per year, the average deviation was just slightly less than half the average rate. Considerable efforts were also made to examine other confounding variables and multicollinearity between two or more independent variables, as discussed elsewhere (*2*). In short, the final model given above was considered the best available for expressing relationships between accidents and related traffic and roadway features.

To illustrate the use of the predictive model, consider a two-lane rural roadway section 3.4 miles long on a rolling terrain, 2,500 ADT, lane width (W) of 10 feet, paved and gravel shoulders (PA and UP) of zero feet, and a roadside hazard rating (H) of five. An estimate of the number of related (AO) accidents per mile, per year would then be

$$\begin{aligned} AO = {} & .0019\,(\mathrm{ADT})^{.8824}\,(.8786)^{W}\,(.9192)^{PA}\,(.9316)^{UP} \\ & \times (1.2365)^{H}\,(.8822)^{TER1}\,(1.3221)^{TER2} \\ = {} & .0019\,(2{,}500)^{.8824}\,(.8786)^{10}\,(.9192)^{0}\,(.9316)^{0} \\ & \times (1.2365)^{5}\,(.8822)^{0}\,(1.3221)^{0} \\ = {} & 1.5 \text{ related accidents per mile, per year.} \end{aligned}$$

For a 3.4-mile section, the expected accidents would be (1.5 related accidents per mile, per year) × 3.4 miles = 5.1 related accidents per year.

Accident Predictive Nomograph

A nomograph was developed, which represents the relationships between selected accidents and the six variables of con-

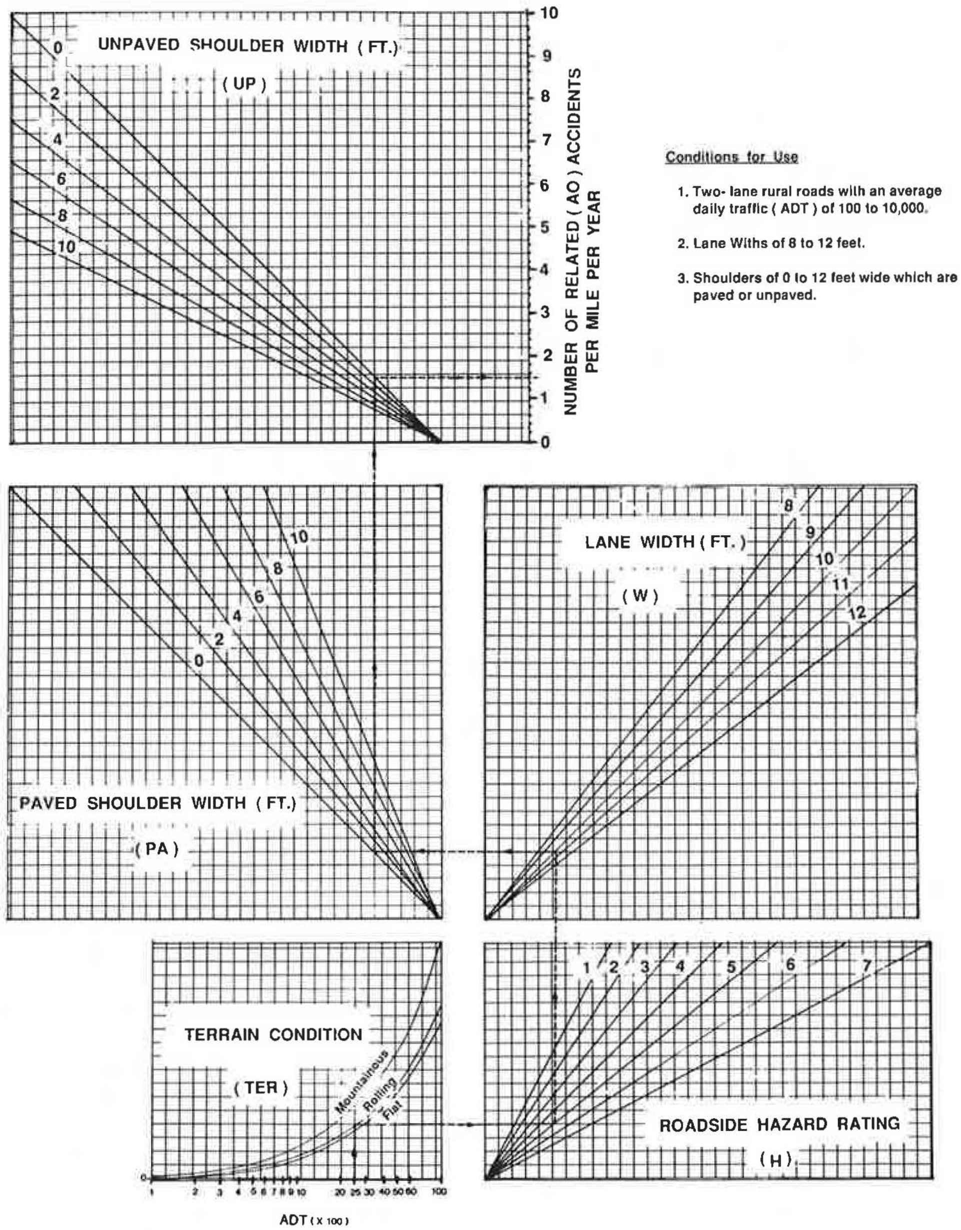


FIGURE 2 Accident prediction nomograph.

cern as illustrated in figure 2. Thus, by knowing the lane width, ADT, terrain, roadside hazard rating, and width of paved and unpaved shoulder, the expected number of related (*AO*) accidents may be determined for a two-lane highway section.

For example, assume the sample section given previously. Enter the nomograph with an ADT of 2,500 and proceed up to the terrain curve (rolling, in this case). From that point, draw a horizontal line to the roadside hazard rating line (*5*). Draw a line up to the lane width (10 feet) line; and then proceed horizontally to the line of the paved shoulder width (zero feet). Next, draw a line up to the unpaved shoulder width line (zero feet) and then over to the accident scale. Read the value of the predicted number of related (*AO*) accidents per mile per year, which is 1.5 in this case (as found using the accident predictive model. Multiplying the section length (3.4 miles) by the number of related accidents per mile per year (1.5) yields 5.1 related accidents per year as calculated in the previous section.

In order to determine the percentage of accident reduction that would result from lane or shoulder widening projects, accident reduction (AR) factors were developed using the model. Values for the factors were determined by computing the predicted difference in related accidents between the before and after conditions (from the model) and dividing that value by the predicted accidents in the before condition. Accident reduction factors for lane widening only are shown in table 4. Table 4 reveals that as the amount of lane widening increases, the percent reduction in related accidents also increases. For example, widening a road with 10-foot lanes to 12-foot lanes (in other words, 2 feet of widening per lane) would be expected to result in a 23 percent reduction in related accidents, all other factors being equal. Accident reduction factors for shoulder widening are shown in table 5. This table reveals

TABLE 4 PERCENT ACCIDENT REDUCTION OF RELATED ACCIDENT TYPES FOR LANE WIDENING ONLY

Amount of Lane Widening (ft.)	Percent Reduction in Related Accident Types
1	12
2	23
3	32
4	40

TABLE 5 PERCENT ACCIDENT REDUCTION OF RELATED ACCIDENT TYPES FOR SHOULDER WIDENING ONLY

Amount of Shoulder Widening (ft.) per Side	Percent Reduction in Related Accident Types	
	Paved	Unpaved
2	16	13
4	29	25
6	40	35
8	49	43

that wider shoulders are associated with a reduction in related (*AO*) accidents. Widening paved shoulders by 4 feet, for example, will be expected to reduce related accidents by 29 percent.

AR factors for various combinations of lane and shoulder widening and paving are shown in table 6. For example, assume a roadway with a 9-foot lane width and a 2-foot gravel shoulder (before condition) is being considered for widening to 11-foot lanes with 4-foot paved shoulders. The expected percent reduction in related accidents can be obtained from table 6, by finding the amount of lane widening (2 feet in this example) and the existing shoulder condition at the left of the table (2-foot unpaved). Looking to the right, find the cell that corresponds to a 4-foot paved shoulder. In this example, a 37 percent reduction in related accidents would be expected to result from the proposed improvements. To determine the number of related accidents per mile, per year that would be avoided by lane and/or shoulder improvements, multiply the AR factor by the number of related accidents per mile, per year from the nomograph or predictive model.

To illustrate the use of these tables, assume an existing 3-mile section of rolling terrain with an ADT of 1,000, a 10-foot lane width, no shoulder, and a roadside hazard rating of five. This would correspond to 0.68 related accidents per mile, per year × three miles = 2.04 related accidents per year in the untreated condition. Widening to 12-foot lanes and 6-foot gravel (unpaved) shoulders would result in a 50 percent reduction in related accidents, according to table 6. This translates to (0.50 × 2.04) = 1.02, or approximately two related accidents reduced per year on the 2-mile section.

Based on the AR factors developed from the model, the same percentage of accidents will be reduced for a specific amount of lane or shoulder widening, regardless of the lane width or shoulder width in the before condition. For example, adding a 4-foot paved shoulder to a 10-foot lane with no shoulder would result in the same accident reduction percentage as adding 4 feet of shoulder to a 12-foot lane with an existing 6-foot paved shoulder. However, the actual number of related accidents eliminated per mile, per year will be greater for adding the 4-foot paved shoulder to the 10-foot lane, since the model would also predict a greater number of accidents for the section with the 10-foot lane. Greater overall benefits would result, then, from adding the 4-foot shoulder to the 10-foot lane.

It is also important to mention that the predictive model and nomograph only apply to two-lane, rural roadways with lane widths of 8 to 12 feet, shoulder widths of zero to 12 feet (paved or unpaved) and traffic volumes of 100 to 10,000. One must not assume that these accident reductions apply, for example, to lane widths of ≤7 feet or ≥13 feet.

AR factors were also developed to determine the percentage of related (*AO*) accidents that would be reduced due to sideslope flattening and lowering the roadside hazard ratings, and details of such effects of roadside improvements are discussed elsewhere (*2, 3*).

SUMMARY AND CONCLUSIONS

This study was intended to quantify the benefits expected from lane widening, shoulder widening, shoulder surfacing, and general roadside improvements. Detailed accident, traffic,

TABLE 6 ACCIDENT REDUCTION FACTORS FOR RELATED ACCIDENT TYPES FOR VARIOUS COMBINATIONS OF LANE AND SHOULDER WIDENING

Amount of Lane Widening (ft)	Existing Shoulder Condition (Before Period)		Percent Related Accidents Reduced							
			Shoulder Condition in After Period							
			2 ft. Shoulder		4 ft. Shoulder		6 ft. Shoulder		8 ft. Shoulder	
	Shoulder Width	Surface Type	Paved	Unpaved	Paved	Unpaved	Paved	Unpaved	Paved	Unpaved
3	0	N/A	43	41	52	49	59	56	65	62
	2	Paved	32	--	43	--	52	--	59	--
	2	Unpaved	34	32	44	41	53	49	60	56
	4	Paved	--	--	32	--	43	--	52	--
	4	Unpaved	--	--	36	32	46	41	54	49
	6	Paved	--	--	--	--	32	--	43	--
	6	Unpaved	--	--	--	--	37	32	47	41
	8	Paved	--	--	--	--	--	--	32	--
	8	Unpaved	--	--	--	--	--	--	39	32
2	0	N/A	35	33	45	42	53	50	61	56
	2	Paved	23	--	35	--	45	--	53	--
	2	Unpaved	25	23	37	33	46	42	55	50
	4	Paved	--	--	23	--	35	--	45	--
	4	Unpaved	--	--	27	23	38	33	48	42
	6	Paved	--	--	--	--	23	--	35	--
	6	Unpaved	--	--	--	--	29	23	40	33
	8	Paved	--	--	--	--	--	--	23	--
	8	Unpaved	--	--	--	--	--	--	31	23
1	0	N/A	26	24	37	34	47	43	55	50
	2	Paved	12	--	26	--	37	--	47	--
	2	Unpaved	14	12	28	24	39	34	48	43
	4	Paved	--	--	12	--	26	--	37	--
	4	Unpaved	--	--	17	12	30	24	41	34
	6	Paved	--	--	--	--	12	--	26	--
	6	Unpaved	--	--	--	--	19	12	31	24
	8	Paved	--	--	--	--	--	--	12	--
	8	Unpaved	--	--	--	--	--	--	21	12

-- These cells are left blank, since they would correspond to projects which would decrease shoulder width and/or change paved shoulders to unpaved shoulders.

roadway, and roadside data from 4,951 miles of two-lane roads in seven states were collected and analyzed. An accident predictive model and detailed statistical procedures were used to determine expected accident reductions related to various geometric improvements. The following are the key study results:

1. The types of accidents found to be most related to cross-section features (lane width, shoulder width, shoulder type, and roadside characteristics) include
   - Single-vehicle (fixed-object, rollover, or run-off-road other)
   - Related multi-vehicle (head-on, opposite-direction sideswipe, or same-direction sideswipe)
   - The combination of the accident types listed above were termed related accidents (or *AO* accidents)
2. The traffic and roadway variables found to be associated with a reduced rate of single-vehicle accidents were wider lanes, wider shoulders, greater recovery distance, lower roadside hazard rating, and flatter terrain. This effect and the accident reductions discussed below are based on the detailed analyses and accident predictive model developed for two-lane rural roads having ADTs between 50 and 10,000; lane widths of 8 to 12 feet, and shoulder widths of zero to 12 feet (paved or unpaved).
3. The effects of lane width on related accidents were quantified. The first foot of lane widening (2 feet of pavement widening) corresponds to a 12 percent reduction in related (*AO*) accidents, 2 feet of widening (widening lanes from 9 to 11 feet, for example) results in a 23 percent reduction, 3 feet results in a 32 percent reduction, and 4 feet of widening results in a 40 percent reduction. These reductions apply only for lane widths between 8 and 12 feet.
4. The effects of shoulder widening on related (*AO*) accidents was determined for paved and unpaved shoulders. For shoulder widths between zero and 12 feet, the percent reduction in related accidents due to adding paved shoulders is 16 percent for 2 feet of widening (each side of the road), 29 percent for 4 feet of widening, and 40 percent for 6 feet of widening. Adding unpaved shoulders would result in 13 percent, 25 percent, and 35 percent reductions in related accidents for 2, 4, and 6 feet of widening, respectively. Thus, paved shoulders are slightly more effective than unpaved shoulders in reducing accidents.

RECOMMENDATIONS

This study provides a set of accident reduction factors to enable computation of estimated accident benefits for a variety of cross-sectional improvements. It is recommended that consideration be made for such improvements on all roadway sections being considered for 3R-type projects. In fact, an informational guide, *Two-Lane Road Cross-Section Design*, has been developed that enables estimation of the safety benefits of various roadway and roadside improvements on specific sections of two-lane roads (*16*). The guide also includes a project-cost model, which is based on cost information from numerous U.S. states.

As discussed earlier, many of the rural, two-lane roads in the United States are restrictive and substandard in terms of lanes, shoulders, roadsides, and other roadway features. Unfortunately, budgetary and practical constraints prevent widening and other needed roadway improvements on all substandard highways at once. One rational approach is to establish priorities for cross-sectional improvements based on where the needs are the greatest. The use of the accident reduction factors given in this paper along with the step-by-step procedures in the informational guide provide a way of computing expected benefits and costs of improvements. This rational decision-making process can help identify the types of projects that are most desirable and cost-effective in various roadway situations.

ACKNOWLEDGMENT

This paper was based on a joint study for the Federal Highway Administration and Transportation Research Board by Goodell-Grivas, Inc. The Highway Safety Research Center of the University of North Carolina served as a Subcontractor for the project. The authors wish to express their appreciation to Justin True, FHWA Contract Manager, and John Deacon and Robert Skinner, who served as the TRB contract managers. Special thanks are also offered to the numerous state representatives in Alabama, Michigan, Montana, North Carolina, Utah, Washington, and West Virginia who cooperated with the project team and provided necessary data and information. The authors assume responsibility for the accuracy of information contained in this paper.

REFERENCES

1. S. A. Smith, J. Purdy, H. W. McGee, D. W. Harwood, A. D. St. John, and J. C. Glennon. *Identification, Quantification, and Structuring of Two-Lane Rural Highway Safety Problems and Solutions*. Report Nos. FHWA/RD/83/021 and 83/022, Federal Highway Administration, Washington, D.C., 1983.
2. C. V. Zegeer, J. Hummer, L. Herf, D. Reinfurt, and W. Hunter. *Safety Effects of Cross-Section Design for Two-Lane Roads*. Report No. FHWA/RD/87/008, Federal Highway Administration, Washington, D.C., December 1986.
3. C. V. Zegeer, J. Hummer, D. Reinfurt, W. Hunter, and R. Stewart. Accident Effects of Sideslope and Other Roadside Features on Two-Lane Roads. In *Transportation Research Record 1195*, TRB, National Research Council, Washington, D.C., 1988.
4. F. Council, et al. *Accident Research Manual*. Report No. FHWA/RD/80/016, Federal Highway Administration, University of North Carolina, 1980.
5. D. D. Perkins. *Highway Safety Evaluation—Procedural Guide*. Federal Highway Administration, Washington, D.C., 1981.
6. E. A. Rinde. *Accident Rates vs. Shoulder Width*. Report No. CA-DOT-TR-3147-1-77-01, Federal Highway Administration, California DOT, 1977.
7. O. K. Dart and L. Mann. Relationships of Rural Highway Geometry to Accident Rates in Louisiana. In *Highway Research Record 312*, HRB, National Research Council, Washington, D.C., 1970. pp. 1–16.
8. P. Shannon and A. Stanley. *Pavement Width Standards for Rural, Two-Lane Highways*. The Pacific Northwest Regional Commission, Idaho Transportation Department, 1976.
9. C. V. Zegeer, J. G. Mayes, and R. C. Deen. Cost-Effectiveness of Lane and Shoulder Widening of Rural, Two-Lane Roads in Kentucky. In *Transportation Research Record 806*, TRB, National Research Council, Washington, D.C., 1981.
10. C. L. Heimbach, W. W. Hunter, and G. C. Chao. Paved Highway Shoulder and Accident Experience. *Transportation Engineering Journal Proceedings*, ASCE, Volume 100, No. TE4, 1974, pp. 889–908.
11. D. S. Turner, D. B. Fambro, and R. O. Rogness. Effects of Paved Shoulders on Accident Rates for Rural Texas Highways. In *Transportation Research Record 819*, TRB, National Research Council, Washington, D.C., 1981, pp. 30–37.
12. R. O. Rogness, D. B. Fambro, and D. S. Turner. Before-After Accident Analysis for Two Shoulder Upgrading Alternatives. In *Transportation Research Record 855*, TRB, National Research Council, Washington, D.C., 1982.
13. T. J. Foody, and M. D. Long. *The Specification of Relationships Between Safety and Roadway Obstructions*. Report No. OHIO-DOT-06-74, Ohio Dept. of Transportation, 1974.
14. Roy Jorgensen & Associates. *Cost and Safety Effectiveness of Highway Design Elements*. NCHRP Report 197, TRB, National Research Council, Washington, D.C., 1978.
15. C. Zegeer and J. Deacon. *Effect of Lane Width, Shoulder Width, and Shoulder Type on Highway Safety: A Synthesis of Prior Literature*. TRB, National Research Council, Washington, D.C., 1986.
16. C. V. Zegeer, J. Hummer, L. Herf, D. Reinfurt, and W. W. Hunter. *Two-Lane Road Cross-Section Design: Informational Guide*. FHWA/RD/87/094, Federal Highway Administration, Washington, D.C., 1986.

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

Accident Effects of Sideslope and Other Roadside Features on Two-Lane Roads

CHARLES V. ZEGEER, DONALD W. REINFURT, WILLIAM W. HUNTER, JOSEPH HUMMER, RICHARD STEWART, AND LYNNE HERF

The purposes of this study were to (1) develop one or more methods for quantifying roadside hazard, (2) define factors that influence run-off-road accidents, and (3) estimate the accident benefits of various roadside improvements. Detailed traffic, accident, roadway, and roadside data were collected on 4,951 miles of two-lane rural roads in seven states. Roadside data included development and use of a pictorial seven-point roadside hazard scale, a measure of roadside recovery (clear zone) distance, field sideslope measurements, and detailed types of and lateral distances to roadside obstacles. Statistical testing was used along with log-linear modeling to determine the interactive effects of roadside and roadway features on accidents. Flatter sideslopes of 3:1 to 7:1 were found to be related to lower rates of single-vehicle accidents. However, only a 2 percent reduction in single-vehicle accidents was found for a 3:1 sideslope compared to a 2:1 sideslope. Reductions in related accidents due to general roadside improvements were found to range from 19 percent to 52 percent, depending on the amount of roadside improvement. Trees and utility poles are the roadside objects most often struck. Obstacles associated with the highest percent of severe accidents include culverts, trees, utility and light poles, bridges, rocks, and earth embankments.

Single-vehicle accidents represent a major safety problem and account for a majority of accidents on rural two-lane roadways with up to 2,000 vehicles per day (*1*, *2*). Many of these accidents occur on roadways with steep roadside slopes and numerous rigid, fixed objects (such as trees and utility poles) close to the roadway edge. Therefore, the need exists for cost-effective improvements to roadside features that will eliminate or reduce these unnecessary hazards.

Numerous types of roadside improvements can reduce the frequency and/or severity of run-off-road accidents. These include

- Flatten slopes where accident experience shows a need.
- Increase horizontal clearance (offset) to obstructions, including utility poles.
- Provide impact attenuators and breakaway devices.
- Extend culverts.
- Remove trees and other obstacles near the roadway edge.

C. V. Zegeer, D. W. Reinfurt, and W. Hunter, Highway Safety Research Center, University of North Carolina, CB 3430 Craige Trailer Park, Chapel Hill, N.C. 27514. J. Hummer, Department of Civil Engineering, Purdue University, West Lafayette, Ind. 47907. L. Herf, Traffic Engineering Department, City of Southfield, Michigan 48076.

Although clearing roadsides of obstructions will provide more recovery area for run-off-road motorists, little information is available about the specific effects on accident frequency and severity of such improvements under various roadway conditions. Thus, highway officials may have difficulty justifying roadside improvements, even where there is a real need. The benefits of roadside improvements should, therefore, be determined so the cost-effectiveness of such improvements can be weighed against other types of improvements (for example, lane widening, shoulder widening, shoulder surfacing, and improved roadway delineation). To accomplish this, it is important to first quantify roadsides in terms of clear, concise definitions for highway safety purposes.

The purposes of this study were to (1) develop one or more methods for better quantifying roadside hazards, (2) define factors that influence the frequency and severity of run-off-road accidents, and (3) estimate the accident benefits of various improvements to the roadside environment. This study included the development of accident relationships and mathematical models based on numerous traffic, roadway, and roadside variables from 4,951 miles of two-lane rural roads in seven states. The study resulted in expected accident reductions resulting from sideslope flattening and various other roadside improvements. This paper was based on a research study performed jointly for the Federal Highway Administration and the Transportation Research Board. More details of the safety effects of roadside and roadway features may be found in that research report (*3*).

BACKGROUND

Roadside Features and Accident Frequency

Studies that have investigated fixed-object accidents by the type of object struck include studies by Newcomb and Negri (*4*), Rinde (*5*), Foody and Long (*6*), and Hall, Burton Coppage, and Dickinson (*7*). These studies indicate that utility poles are among the fixed objects most frequently involved in roadside accidents. Other frequently struck roadside objects include trees, sign posts, guardrails, ditch embankments, and bridge structures.

Other studies indicate that the number of fixed objects and their offset influences roadside accident frequency. For example, Zegeer and Parker found that utility pole accidents increased significantly with a decrease in pole offset or an increase in ADT or pole density (*8*). Mak and Mason also found that pole density and pole offset had an effect on the frequency of pole accidents (*9*). Jones and Baum found that the number of poles and pole spacing was highly related to the probability of a utility pole accident (*10*).

Hall et al. reported that most of the utility pole accidents they examined involved poles that were either within 11.5 feet of the roadway or on the outside of horizontal curves (*7*). Foody and Long reported that 37 percent of all single-vehicle, fixed-object accidents involved objects 6 to 12 feet from the roadway. Also, approximately 81 percent of the accidents involving roadside features occurred within 20 feet of the roadway (*6*).

Relationships have also been reported between the degree of sideslope and roadside accident frequency. Graham and Harwood examined the effect of clear recovery zones and different sideslopes and found that steeper sideslopes caused an increase in single-vehicle, run-off-road accidents for all ADT levels and roadway types (*11*). Weaver and Marquis simulated various roadside slope designs and discovered that vehicles leaving the roadway were less likely to roll if the slope was fairly flat (*12*). Perchonok, Ranney, Baum, Morrison and Eppick found that higher landfills and deeper ditches caused more vehicle rollovers (*13*). Roadway geometrics, such as horizontal curvature and grade, have been reported by Jones and Baum (*10*), Hall et al. (*7*), and Perchonok et al. (*13*) to affect the number of run-off-road and fixed-object accidents.

Roadside Features and Accident Severity

Jones and Baum found that the types of fixed objects associated with the most severe accidents include utility poles (49.7 percent injury) and trees (41.8 percent) (*10*). Rollover accidents resulted in 51.4 percent injury and 1.1 percent fatal accidents. Other accidents associated with high severity included accidents involving bridges, culverts, ditches, and embankments. Graf, Boos, and Wentworth found that 47 percent of utility pole accidents resulted in a fatality or injury and that ditch embankments and utility poles were also among the most hazardous obstacles in terms of accident severity (*14*).

Higher vehicle speeds have been found to be associated with greater accident severity. Mak and Mason investigated the relationship between the severity of pole accidents and vehicle impact speed (*9*). The authors reported that there is a 50 percent chance of injury in a pole accident at impact speeds as low as 6 mph. The severity of these injuries increased dramatically for impact speeds above 30 mph. On the other hand, Jones and Baum, using the speed limit to approximate impact speed of utility pole accidents, estimated that a 50 percent chance of injury exists in a utility pole accident when the impact speed is approximately 34 mph (*10*). The sizable discrepancy in results between these two studies could be partly due to difficulties in estimating impact speed and/or inappropriateness in using speed limit to approximate impact speeds.

Roadside Accident Prediction Models

Several models have been developed to predict the frequency and/or severity of single-vehicle or roadside accidents. Edwards et al. developed what is probably the most widely known model for determining hazardous roadside obstacles (*15*). This probabilistic hazard index model was developed to predict the annual number of fatal and nonfatal injury accidents associated with roadside objects on freeway sections. Glennon and Wilton modified the model to include other roadway types, including urban arterial streets, rural two-lane highways, and rural multi-lane highways (*16*). However, the model relies on accurate estimates of vehicle encroachments, which have been a topic of controversy in recent years.

Cleveland and Kitamura developed a group of multiplicative regression equations to predict the frequency of run-off-road accidents on rural two-lane highways in Michigan (*17*). Models were developed based on the following factors: (1) traffic volume, (2) percentage of road length with passing sight restrictions, (3) percentage of road length curved, (4) number of curves, and (5) percentage of road length with roadside objects within 20 feet. A severity model was also developed, and key factors included traffic volume, percentage of road length curved, percentage of road length having roadside objects within 10 feet, and object stiffness. Reported R^2 values for the models ranged from 0.26 to 0.49, but model validation revealed less than desired results due to data outliers.

Zegeer and Parker tested ten different models to predict annual utility pole accidents per mile (*8*). The models were based on 2,500 miles of highway in four states, involving 9,500 utility pole accidents. The multiplicative exponential model was selected as optimal because it not only had the highest R^2 value (0.63), but it also made intuitive sense. Model validation proved quite successful for an independent data set. A nomograph was developed that allows easy estimation of frequency of utility pole accidents based on known levels of traffic volume, the number of poles per mile, and the lateral offset of poles from the travel lane. The model, however, applies only to utility pole accidents and not to other roadside accident types, such as trees, rollover accidents, etc.

In summary, considerable research has been conducted on the frequency and severity of run-off-road accidents and related factors. However, review of the literature also indicated a definite need to better quantify roadside hazards and to develop a means to accurately predict run-off-road accidents for a variety of traffic, roadway, and roadside conditions. This study was initiated as a result of those needs.

PLANNING AND COLLECTION OF DATA

Analysis Issues

The data collection was guided by the need to address the following types of issues:

1. What methods or scales can be used to define and quantify roadside hazards?
2. What is the effect of various traffic, roadway, and roadside factors on run-off-road accidents?
3. What is the expected accident reduction that will result from improvements to the roadside?
4. What is the effect of sideslope on the rate of single-vehicle and rollover accidents?
5. What types of roadside obstacles are most often struck, and what accident severities are associated with each obstacle type?

The first issue above was addressed at a workshop of 13 safety professionals. Hundreds of roadside photographs were reviewed at the workshop, which led to the development of a "Roadside Hazard Scale" and "Roadside Recovery Distance" as measures of roadside condition. Issues 2 and 3 above were addressed after collection and analysis of detailed accident, traffic, and roadway data from 4,951 miles of two-lane rural roadways in seven U.S. states and the development of accident predictive models.

Issue 4 (effects of sideslopes) was investigated based on field-measured sideslopes on 1,776 miles of roadway in three states along with other accident, traffic, and roadway data. An accident predictive model was developed for sideslopes of 2:1 or steeper, 3:1, 4:1, 5:1, 6:1, and 7:1 or flatter. Issue 5 was addressed using the 4,951-mile database and other detailed accident severity information from three states to better define the types of obstacles associated with high accident frequencies and severities.

Selection of Data Variables

The data variables needed for this study included traffic and roadway variables (traffic volume, lane width, shoulder width, shoulder type, sideslope), roadside obstacle variables, accident variables, (for example, by type and severity), and other traffic and roadway features that have a proven or logical relationship with accidents. Variables were selected for use in this study based on a literature search of past research to determine which ones are important on two-lane roads in rural, suburban, or urban areas. The collection of every possible roadway, traffic, and accident variable would have been both unnecessary and impossible.

Traffic and Roadway Variables

For each of the selected roadway sections, the following traffic and roadway variables were collected:

- Section information (identification number, length, pavement type, terrain, ditch type, area type, type of development, speed limit)
- Traffic volume (ADT)
- Horizontal and vertical curvature of the section (expressed in terms of percent of section with various degrees of curvature or percent grade)
- Sideslope length and sideslope ratio (2:1 or steeper, 3:1, 4:1, 5:1, 6:1, or 7:1 or flatter, which is expressed as the ratio of the lateral distance to the vertical drop of the sideslope). For each section, sideslope measures were recorded up to four points per mile for each side of the road from field measurements and/or photolog techniques and expressed as minimum sideslope, maximum sideslope, average sideslope, 20 percent sideslope value, median sideslope value, and 80 percent sideslope value.
- Width of lanes and shoulders and shoulder type (paved, stabilized, gravel, earth, or grass)
- Number of bridges, intersections (by type of sign or signal control), overpasses, railroad crossings, driveways (by type—residential, commercial, recreational or industrial)
- Type of delineation and on-street parking

Roadside Obstacle Variables

Individual Obstacle Data Data were collected for specific types of roadside obstacles and also in terms of overall roadside hazard, as described later. For each roadway section, an inventory was taken of every point obstacle within 30 feet of the road, as measured from the edge line or the outside edge of the travel lane. The inventory involved classifying each point object into the appropriate category of distance from the travel lane: zero to 1 foot, 2 to 3 feet, 4 to 6 feet, 7 to 10 feet, 11 to 15 feet, 16 to 20 feet, 21 to 25 feet, and 26 to 30 feet. In addition to point objects, an inventory was also taken of 18 different types of continuous objects, such as guardrails and bridge rails for each roadway section in terms of their length and offset from the roadway (zero to 1 foot, 2 to 3 feet, etc.). The inventory of point and continuous objects in this manner allowed for matching the frequency, length, and placement of specific obstacle types for a section with the corresponding types of obstacles struck on each section. A detailed discussion of the results of the analyses by obstacle type is given in the full research report (*3*).

Roadside Hazard Ratings While a detailed inventory was conducted of roadside obstacles on each section, there was also a need to develop one or more measures of roadside hazard that would be representative of the overall roadside hazards for the section. However, very little such research has been performed to characterize roadside condition.

A roadside hazard scale was developed based on the literature review and the results of a workshop involving thirteen highway and roadside safety professionals. At the workshop, hundreds of photographs of roadside situations in both rural and urban areas from more than fifteen states were organized and shown to workshop participants. The participants rated situations in each photograph in terms of potential frequency and potential severity of accidents and also in terms of overall hazard, for a combined frequency and severity scale. A two-dimensional rating scale, involving a three by three matrix, with frequency on the horizontal axis and severity on the vertical axis, was tested at the workshop. In general, workshop participants considered the matrix confusing and difficult to use. Further, it was concluded that a two dimensional rating scale would make analysis of the rating difficult.

Three ordinal, seven-point scales were later tested individually by workshop participants. One scale was based on frequency and one on severity. The third, referred to as a hazard scale, considered both frequency and severity. The purpose of the tests was to determine whether the hazard scale or the separate frequency and severity scales provided the most consistent results. The workshop participants were asked to use the scales in the following way:

Hazard Rate each roadside according to accident damage likely for errant vehicles on a scale from one (low likelihood of off-road collision or overturn) to seven (high likelihood of an accident resulting in fatality or severe injury).

Frequency Rate each roadside according to the frequency with which errant vehicles are likely to become involved in off-roadway accidents (that is, collide with fixed objects or overturn) on a scale from one (low likelihood of involvement) to seven (high likelihood of involvement).

Severity Rate each roadside according to the likely severity of off-roadway accidents on a scale from one (low likelihood of fatality or severe injury) to seven (high likelihood of fatality or severe injury).

The thirteen participants were asked to rate 141 photographs of roadsides in rural areas and seventy-eight photographs of roadsides in urban areas (sixty-four without on-street parking, fourteen with on-street parking) based on the above instructions. The ratings were collected and descriptive statistics were examined to determine which scale(s) produced the most consistent ratings. The standard deviations, ranges of the ratings, and other data descriptors for each photograph were used to measure rating consistency.

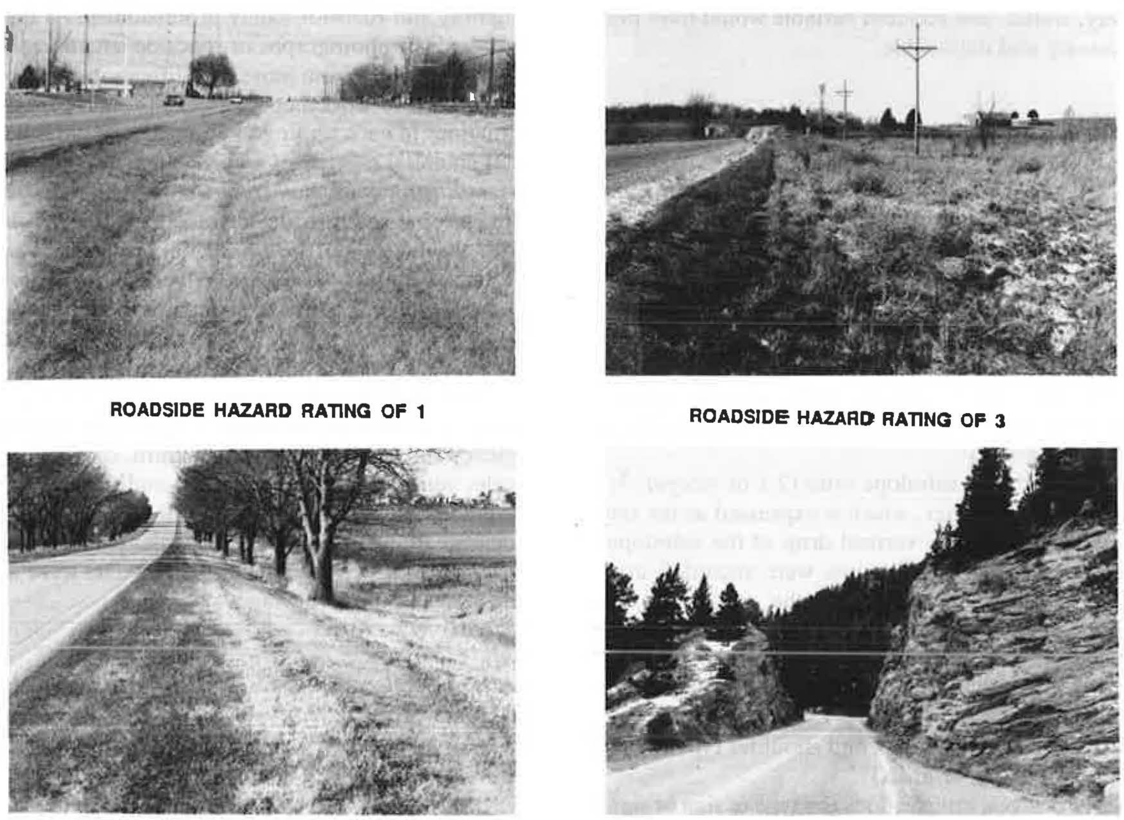
In summary, the hazard scale was the most desirable scale for rural areas, while the separate frequency and severity scales were slightly better suited for urban areas. For statistical analysis purposes (including model development) the hazard scale was favored over the other two scales, since it provided the capability for expressing the roadside condition in one independent variable, which could then be included in an accident model along with lane width, shoulder width, and other roadway features. Therefore, the seven-point hazard scale was selected for this study. Pictorial seven-point roadside hazard scales were developed separately for rural and urban areas. Sample photographs that were included in the final rural hazard rating scale are provided in Fig. 1. In general, steep sideslopes and/or large obstacles close to the roadway correspond to a hazard rating of seven, and clear, level roadsides represent a hazard rating of one.

For each highway section in this study, roadside hazard ratings were recorded each tenth of a mile on each side of the road, for a total of twenty measurements per mile. For each roadway section, the following roadside hazard rating variables were available for analysis:

1. Type of scale used for roadside rating; for example, did the roadside appear more like the rural scale or urban scale
2. Number of roadside ratings of each rating level (number of ratings of one, two, three, four, five, six, and seven)
3. Median (50th percentile), 20th percentile, and 80th percentile roadside ratings

Roadside Recovery Distance In addition to the subjective roadside hazard rating, a measure termed roadside recovery distance was also developed. This measure was defined as follows: The roadside recovery area is a basically flat, unobstructed, and smooth area adjacent to the outside edge of the travel lane (edgeline) within which there is reasonable opportunity for safe recovery of an out-of-control vehicle. The width of the roadside recovery area is the lateral distance from the edgeline to the nearest of the following:

- A hinge point where the slope first becomes steeper than 4:1



ROADSIDE HAZARD RATING OF 1

ROADSIDE HAZARD RATING OF 3

ROADSIDE HAZARD RATING OF 5

ROADSIDE HAZARD RATING OF 7

FIGURE 1 Sample photographs of roadside hazard scale.

- A longitudinal element such as a guardrail, bridge rail, or barrier curb
- An unyielding and therefore hazardous object
- The ditch line of a non-traversable side ditch (considering as an approximation that a ditch is traversable if both foreslope and backslope are 4:1 or flatter)
- Other features such as a rough or irregular surface, loose rocks, or a watercourse that pose a threat to errant vehicles

In this study, the roadside recovery distance was measured from the edgeline (or outside edge of the lane), although it could have been measured from the shoulder edge. Measures of roadside recovery distance were taken from photolog film at 0.1-mile intervals (every 10th frame of film) for each section on both sides of the road, for a total of twenty measurements per mile. A series of calibrated grid overlays (with lines of lateral distances from the edge of the travel lane) was used on the photolog film to measure the clear recovery distance at the selected frames. Since an observer could view about 0.1 mile of the road in each frame and measurements were taken every 0.1 mile, the measurements of roadside recovery distance represented a nearly 100 percent coverage of roadside on both sides of the road.

For each section, the roadside recovery distance measurements were summarized and the following values were computed: minimum, average, maximum, and percentile values (for example, 20th, 50th, and 80th percentile). Separate variables were also computed which provide the percentage of the section with recovery area of ≤ 5 feet, ≤ 10 feet, ≤ 15 feet, ≤ 20 feet, ≤ 25 feet, ≤ 29 feet, and ≥ 30 feet.

Accident Variables

For each roadway section, the accident information collected included the number of years of accident data (5 years in most cases); total number of accidents on the section; number of accidents by severity category (property damage only, A-injury, B-injury, C-injury, and fatal); number of people injured (by injury level) or killed; number of accidents by light conditions and pavement conditions; number of accidents by type (fixed object, rollover, other run-off-road, head-on, opposite-direction sideswipe, same-direction sideswipe, rear end, backing or parking, pedestrian or bike or moped, angle or turning, train-related, animal-related, other or unknown); and number of accidents by type of fixed obstacle struck.

Site Selection

A sample of 4,951 miles of two-lane rural roads (1,944 roadway sections) was selected from the following seven states: Alabama, Michigan, Montana, North Carolina, Utah, Washington, and West Virginia. Sites were selected using stratified random sampling (that is, sites were randomly selected from categories having specified values of lane width, shoulder width, shoulder type, and ADT) so the resulting data would cover the normal range of these important variables. Only two-lane rural and urban-suburban sites were selected, and section lengths ranged from 1 to 10 miles in rural areas and from 0.5 to 5 miles in urban areas. Sections were selected that were relatively homogeneous throughout the section with regard to basic geometric and operational features. For example, a section ended when moderate changes occurred in ADT, the lane width changed by one foot or more, the shoulder width changed by more than 3 or 4 feet, or a noticeable change occurred in the roadside condition.

Data Sources and Methods

The data sources for the accident analysis included field data collection, photologs, state agency records (maps, ADT listings, computerized roadway inventories), police accident records (either computer accident tapes or computer accident summaries), and the Highway Performance Monitoring System (HPMS) computer database. Most of the roadway information was extracted from photologs, including roadside data for individual obstacles, roadside hazard ratings, and measures of roadside recovery distance.

Detailed roadside data and roadway information were recorded from stated photologs. The photologs were 35mm photographs taken from a moving vehicle in equal distances of 100 frames per mile (52.8 feet between frames). Location information was given at the bottom of each frame of film and typically included route number, milepost, county, direction of travel, and date of filming. Teams of technicians viewed frames consecutively for preselected sections and recorded information directly onto data forms. The three data forms used with the photolog film included those for basic roadway data, cross-section data, and detailed roadside obstacle data. For data involving lane and shoulder widths and lateral placement of roadside obstacles, a calibrated grid was placed over the photolog viewing screens for each photolog frame. Data were keyed into a computer file, and extensive data checks and corrections were performed before the file was analyzed.

RESULTS

To identify the traffic and roadway factors most closely related to accident experience, numerous statistical procedures were used, including Chi-square analysis, analysis of variance and covariance, and stepwise regression. Single-vehicle (AS) accidents (includes fixed-object, run-off-road rollovers, and other run-off-road accidents) were considered to be of primary interest. Head-on, opposite-direction sideswipe, and same-direction sideswipe accidents were also found to be related to general roadway design. Thus, those three accident types plus the run-off-road accident types combined together were termed "related" (or AO) accidents for analysis purposes. Total accidents (AT) were also used in the modeling.

MODEL BUILDING

Statistical tests for association were used to determine the traffic and roadway variables most closely related to accidents on two-lane rural roads. Such important variables included ADT, lane width, width of paved and unpaved shoulder, roadside hazard rating, median recovery distance, sideslope steepness, terrain, horizontal curvature, number of driveways per mile, and number of intersections per mile.

The development of accident predictive models involved the use of these variables and variable interactions. The roadside recovery distance was redefined for use in the model to represent the measurement from the outside of the shoulder and not the edgeline as it was previously defined. The reason for the redefinition was that shoulder width was also included in the model and was, therefore, already accounted for. Many logical model forms were examined with the most important candidate data variables, as discussed in detail elsewhere (*3*).

The final recommended model using roadside hazard as the roadside variable was as follows:

$$AO/M/Y = 0.0019\ (\mathrm{ADT})^{0.8824}(0.8786)^{W}(0.9192)^{PA} \times (0.9316)^{UP}(1.2365)^{H}(0.8822)^{TER1}(1.3221)^{TER2}$$

where

$AO/M/Y$ = related accidents (single-vehicle, head-on, opposite-direction sideswipe, and same-direction sideswipe accidents) per-mile, per-year,
ADT = average daily traffic,
W = lane width,
PA = average paved shoulder width,
UP = average unpaved (gravel, stabilized, earth, or grass) shoulder width,
H = median roadside hazard rating,
$TER1$ = 1 if flat terrain, zero otherwise, and
$TER2$ = 1 if mountainous terrain, zero otherwise.

The R^2 value for this model was 0.456, which implies that 45.6 percent of the variation in AO accidents is explained by the traffic and roadway variables included in this model. The relative contribution of each variable to this explained variation was 70.2 percent by ADT, 8.6 percent by W, 1.7 percent by PA, 10.5 percent by UP, 7.2 percent by H, and 1.8 percent by TER. The coefficients were reasonable in terms of the relative importance of the variables, and the relationships were in basic agreement with much of the current literature. In fact, the average rates of total and single-vehicle accidents (by ADT and lane width categories) were found to agree closely with other prominent state and national research studies.

A similar model was also developed using average roadside recovery distance ($RECC$) in place of roadside hazard rating. This model for related (AO) accidents per mile per year is:

$$AO/M/Y = .0076\ (ADT)^{0.8545}\ (.8867)^{W}\ (.8927)^{PA}\ (.9098)^{UP} \times (0.9715)^{RECC}\ (.8182)^{TER1}\ (1.2770)^{TER2}$$

where

$RECC$ = the average roadside recovery distance as measured from the outside edge of the shoulder.

All other terms are as previously defined. The R^2 for this model was 0.461, and each term individually had a significant effect on related accidents. For this model, ADT accounted for 69.4 percent of the explained variation, 8.5 percent by W, 1.7 percent by PA, 10.4 percent by UP, 7.5 percent by $RECC$, and 2.5 percent by TER. Accident rate models (models using accidents per 100 million vehicle miles (mvm)) were also tested, but were found to be no better than the selected model forms in terms of R^2 and standard errors.

The first selected model described above was used to estimate accident reductions (accident reduction factors) expected due to roadway improvements such as lane widening, shoulder widening, and shoulder surfacing. Details of the effects of lane width, shoulder width, and shoulder type from this model are given elsewhere (*3*). The reductions in related (AO) accidents from roadside improvements were produced based on roadside hazard rating (from model 1) and roadside recovery distance (from model 2), as shown in tables 1 and 2, respectively.

Table 1 indicates that a reduction in roadside hazard rating of one (from seven to six, six to five, five to four, etc.) due to a roadside improvement would be expected to reduce the number of related (AO) accidents by 19 percent. Similarly,

TABLE 1 ACCIDENT REDUCTION FACTORS DUE TO REDUCING ROADSIDE HAZARD RATING

Reduction in Roadside Hazard Rating (Number of Levels)	Reduction in Related Accidents (%)
1	19
2	34
3	47
4	52
5	65

TABLE 2 ACCIDENT REDUCTION FACTORS DUE TO INCREASING ROADSIDE CLEAR RECOVERY DISTANCE

Amount of Increased Roadside Recovery Distance (feet)	Reduction in Related Accidents (%)
5	13
8	21
10	25
12	29
15	35
20	44

larger reductions in roadside hazard ratings will result in larger reductions. For example, a reduction in roadside hazard of five (such as from seven to two) would be expected to reduce related accidents by 65 percent. Such a roadside improvement would correspond to correcting an extremely dangerous roadside hazard to a nearly flat sideslope with few obstacles and would likely be inordinately expensive.

The roadside hazard scale is an ordinal scale, so an obstacle with a hazard rating of four is not necessarily twice as hazardous as an obstacle with a rating of two. Thus, it may be difficult to understand how a change in hazard rating of seven to five would yield a similar accident reduction (34 percent) as a change from three to one (both would reduce the hazard rating by two levels). This result is due to the nature of the accident model and the equivalent effect on accidents for each unit of increase in the roadside hazard scale. It should be mentioned, however, that the model will predict a higher number of accidents with a rating of seven than for a rating of three. Thus, a reduction in hazard rating from seven to five will indeed result in greater accident benefits than a reduction from three to one. It should also be mentioned that although the roadside hazard scale is subjective in nature, because it was developed by a panel based on roadside photographs, its association with accidents was found to be good, and it is also easy to apply.

Accident reduction factors were also computed for various increases in roadside recovery distance, as shown in table 2. An increase in recovery distance (measured from the outside edge of the shoulder) of 5 feet would reduce related (*AO*) accidents by 13 percent. Providing 20 feet of additional roadside recovery distance would reduce related accidents by 44 percent, according to the model.

One of the issues of importance in applying accident reduction factors in table 1 and 2 above is determining what action is needed to increase the recovery distance. Examples of such treatments may include tree removal, relocating utility poles, burying utility lines, flattening sideslopes and removing obstacles, providing traversable culverts, and others. Measures to reduce the hazard rating may include all of those cited above plus others such as installing a guardrail in front of a steep slope or rigid objects, or providing breakaway bases to light poles and/or signposts.

Roadside Conditions and Single-Vehicle Accidents

In addition to modeling accidents, more information was desired on actual rates of single-vehicle accidents for various roadside and roadway conditions. Single-vehicle accident rates were computed for various combinations of lane width, shoulder width, and average roadside recovery distance, as shown in table 3. Analysis of covariance procedures, with ADT as the covariate, were used to adjust the mean single-vehicle accident rates for differing values of ADT across the cells of the table. The adjusted mean accident rates decrease as lane width, shoulder width, and roadside recovery distance increase. Of particular note was the low rate of single-vehicle accidents for most cases of 17- to 30-foot roadside recovery distances.

Unadjusted single-vehicle accident rates for urban areas are given in table 4 for various lane width categories. Drastic reductions in single-vehicle accident rates may be observed for increases in average roadside recovery distances, particularly beyond 10 feet. These trends are consistent for all three lane width groups. Such summary tables agree with the results of the accident predictive models in terms of the beneficial effects of roadside improvements.

TABLE 3 MEAN ADJUSTED SINGLE-VEHICLE ACCIDENTS PER 100 MVM FOR LANE WIDTH, SHOULDER WIDTH, AND AVERAGE ROADSIDE RECOVERY DISTANCE USING RURAL SECTIONS

Lane Width, ft.	Shoulder Width, ft.	Average Roadside Recovery Distance, ft.		
		0-8	9-16	17-30
8 - 10	0 - 3	203 (130)	183 (47)	87 (20)
	4 - 5	140 (95)	119 (80)	70 (58)
	6 - 13	144 (19)	85 (104)	43 (67)
11 - 14	0 - 3	146 (100)	133 (95)	58 (92)
	4 - 5	122 (92)	77 (121)	46 (86)
	6 - 13	96 (50)	74 (301)	45 (244)

( ) = Number of sample sections given in parenthesis.

Note: Controlled for ADT.

TABLE 4 SINGLE-VEHICLE ACCIDENT RATE (ACC./100 MVM) BY LANE WIDTH AND AVERAGE ROADSIDE RECOVERY DISTANCE FOR URBAN SECTIONS IN SEVEN STATES

Lane Width (ft)	Average Roadside Recovery Distance (ft)			
	0 to 5	6 to 10	11 to 15	16 to 30
≤ 10	105(14)	76(11)	24(10)	23(5)
11	130(4)	100(15)	54(17)	37(7)
≥ 12	135(15)	97(15)	74(19)	56(11)

( ) = Numbers of sample sections are given in parenthesis.

Sideslope Effects

The next analysis was conducted to determine the effect of sideslope on the rate of single-vehicle and rollover accidents. The analysis of sideslope effects on accident experience was based solely on an analysis of 595 rural roadway sections (1,776.85 miles) in three states (Alabama, Michigan, and Washington) where field measurements of sideslope were taken. The rural sections were not used where only photolog estimates of sideslope were available, since an analysis found that sideslope estimates from photologs were of insufficient accuracy. Thus, even though a reduced sample of rural sections was used for this analysis, the greater accuracy of the field sideslope measurements was considered desirable and the sample size was more than adequate for the detailed analysis and accident modelling that was carried out.

The analysis of sideslope effects on accidents consisted of fitting loglinear regression models to two different dependent variables: single-vehicle accident rate (*AS*) and rollover accident rate (*AR*). The accident rates for *AS* and *AR* were in terms of accidents per 100 mvm. Single-vehicle accidents include three types: fixed object, rollover, and other runoff-road accidents (where each accident was counted only once).

For each of the 595 sample sections, the median (50th percentile) sideslope measurement was used as the most representative sideslope, even though sideslopes may vary considerably within a given section. Each section was then classified into one of the following six sideslope categories: 2:1 or steeper; 3:1; 4:1; 5:1; 6:1; or 7:1 or flatter.

A series of log-linear models were fit to the single-vehicle accident rates, starting with simple models containing only sideslope (*SS*) as an independent variable, then including other relevant variables, such as lane width (*W*), shoulder width (*SW*), roadside recovery distance (*RECC*), ADT, and roadside hazard rating (*H*). Sideslope was included in two different forms: as a continuous variable with values 1, 2, 3, etc. (indicating slopes of 1:1, 2:1, 3:1, etc.) and as a categorical variable with six categories (1:1 and 2:1), (3:1). (4:1), (5:1), (6:1), and (7:1 or flatter). In each model, sideslope was found to have a statistically significant effect, where segments with steeper sideslopes had higher rates of single-vehicle accidents than sections with flatter sideslopes. The best predictive models for single-vehicle accidents were found to contain the variables lane width, shoulder width, roadside recovery distance (as measured from the outside of the shoulder to the nearest roadside hazard), ADT, and sideslope. Roadside recovery distance was measured from the outside of the shoulder because the shoulder width is already accounted for in the model. An examination of the model forms with sideslope as a categorical variable showed that sideslopes of 3:1 or greater had significantly higher single-vehicle accident rates than those of 4:1 or flatter. Thus, the resulting model for the single-vehicle accident rate (*AS*) using two categories of sideslope was as follows:

$$AS = 793.58(1.191)^{SS}(0.845)^{W}(0.974)^{RECC} \times (0.99994)^{ADT}(0.908)^{SW}$$

$$R^2 = .18$$

where

AS = the rate of single-vehicle accidents (in accidents/100 MVM)
SS = median (50th percentile) sideslope measure, where SS = 1 if sideslope is 3:1 or steeper, or zero otherwise
ADT = average daily traffic (50 to 10,000)
W = lane width in feet (8 to 13)
SW = total shoulder width (paved plus unpaved) in feet (0 to 12)
$RECC$ = median (50th percentile) roadside recovery distance from the outside edge of the shoulder to the nearest roadside obstacle or hazard (0 to 30 feet)

In the model given above, each of the roadway variables was significant (including sideslope), in terms of affecting the rate of single-vehicle accidents. Since *SS* in this model takes on only values of zero or one, it follows that having a steep (such as 3:1 or steeper) slope is associated with a 19 percent higher rate of single-vehicle accidents than a flatter slope (such as 4:1 or flatter). This is because a factor of 1.191 (1.191^1 = 1.191) would be multiplied by the remaining terms for a steep sideslope, compared to a factor of 1.000 (1.191^0 = 1) for a sideslope of 4:1 or flatter.

While the results of this model are based on significant effects of sideslope on single-vehicle accident rate, there was a need to further refine the model for more sideslope categories. This would, for example, allow for determining the incremental effects of sideslopes of 2:1 or steeper, 3:1, 4:1, 5:1, 6:1, and 7:1 or flatter. The best sideslope model of this type was as follows:

$$AS = 731.16\,(0.839)^{W}\,(0.99995)^{ADT}\,(0.975)^{RECC}\,(0.909)^{SW} \times (1.373)^{SS1}\,(1.349)^{SS2}\,(1.238)^{SS3}\,(1.164)^{SS4}\,(1.091)^{SS5}$$

where

$SS1$ = 1 if sideslope = 2:1 or steeper, or zero otherwise,
$SS2$ = 1 if sideslope = 3:1, or zero otherwise,
$SS3$ = 1 if sideslope = 4:1, or zero otherwise,
$SS4$ = 1 if sideslope = 5:1, or zero otherwise,
$SS5$ = 1 if sideslope = 6:1, or zero otherwise.

For a sideslope of 7:1 or flatter, the last five terms of the equation would each become 1.0. For a sideslope of 2:1 or 1:1, the last four terms of the equation become 1.0 and the term $(1.373)^{SS1} = (1.373)^1 = 1.373$, so the remaining terms of the equation are multiplied by a factor of 1.373. Likewise, for a sideslope of 3:1, the corresponding factor would be 1.349, and so on.

This model indicates that the rate of single-vehicle accidents decreases steadily for sideslope categories of 3:1, 4:1, . . . to 7:1 or flatter, as illustrated in figure 2. Figure 2 shows a ratio of the single-vehicle accident rate for a given sideslope to the single-vehicle accident rate for a sideslope of 7:1 or flatter. These values are based on the coefficients from the predictive model and using the 7:1 or flatter category as the basis of comparison. A review of figure 2 shows, for example, that the single-vehicle accident rate is 1.24 times higher on roads with a 4:1 sideslope than on roads with a sideslope of 7:1 or flatter. Note that little difference is found for sideslopes of 3:1, compared to those of 2:1 or steeper. This indicates that flattening sideslopes from 2:1 or steeper to 3:1 would be of little, if any, value in reducing single-vehicle accidents.

Based on the model results for various sideslopes, table 5 was developed to show likely reductions in single-vehicle accidents due to various sideslope flattening projects. Table 5 indicates that flattening a sideslope of 2:1 on a two-lane rural highway would be expected to reduce single-vehicle accidents by two percent if flattened to 3:1, 10 percent if flattened to 4:1, and 27 percent if flattened to 7:1 or flatter. Similarly, flattening a 4:1 sideslope to 7:1 or flatter would be expected to yield a 19 percent reduction in single-vehicle accidents.

The R^2 value for the above model was 0.19, which indicates that only 19 percent of the variation in the single-vehicle accident rate is explained by the variables in the model. While this may appear to be less than desirable, it should be remembered that high R^2 values rarely result from predictive modeling of accident experience, due to random accident fluctuations, imperfect accident reporting systems, effects of driver and vehicle factors on accidents, and other reasons. Also, accident rates tend to fluctuate widely, particularly on low volume roads.

In spite of the R^2 value, the model was found to be desirable in terms of reasonableness of the coefficients, significance of the model (at the 0.0001 level), inclusion of important variables (each of which had a significant effect on single-vehicle accidents), logical relationships between accidents and other variables, and reasonable predictive ability compared with real-world data.

Figure 3 shows the single-vehicle accident rate expected for six categories of sideslope and for 9-foot to 12-foot lane widths based on the predictive model. All curves are for sections with an ADT of 1,000, a shoulder width of 4 feet, and a 10-

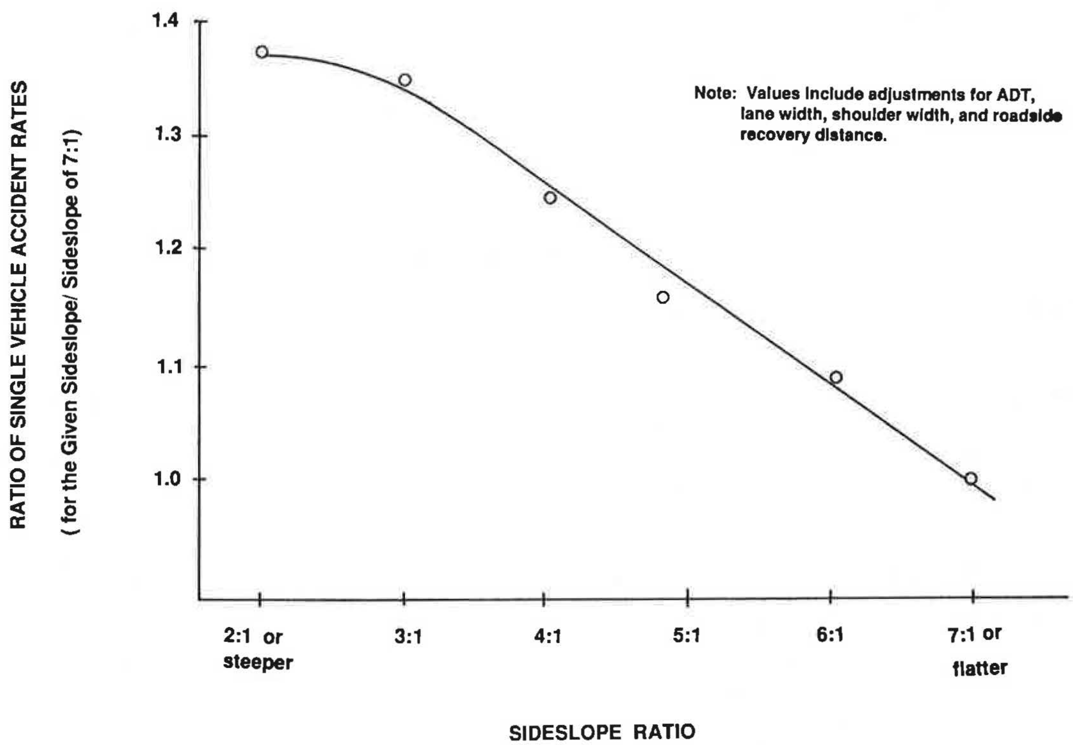


FIGURE 2 Plot of single-vehicle accident rate for a given sideslope versus single-vehicle accident rate for a sideslope of 7:1 or flatter.

TABLE 5 SUMMARY OF EXPECTED PERCENT REDUCTION IN SINGLE-VEHICLE ACCIDENTS DUE TO SIDESLOPE FLATTENING

Sideslope Ratio in Before Condition	Sideslope Ratio in After Condition				
	3:1	4:1	5:1	6:1	7:1 or Flatter
2:1	2	10	15	21	27
3:1	0	8	14	19	26
4:1	-	0	6	12	19
5:1	-	-	0	6	14
6:1	-	-	-	0	8

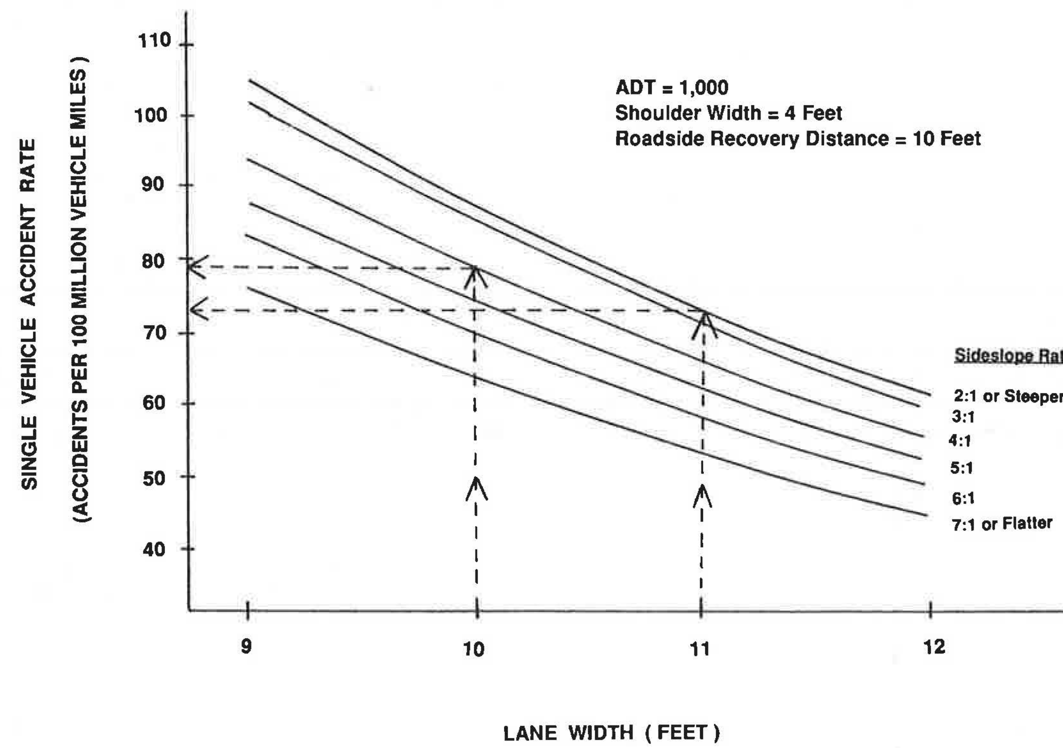


FIGURE 3 Illustration of single-vehicle accident rates for various lane widths and sideslopes.

foot roadside recovery distance beyond the shoulder edge. To illustrate the use of figure 3 for a lane width of 11 feet, sideslopes of 3:1, 4:1, and 6:1 would yield expected single-vehicle accident rates (accidents/100 mvm) of 72, 66, and 58, respectively.

The curves in figure 3 can also be used to determine tradeoffs between the effects of lane width and sideslope. For example, for a roadway section with 1,000 ADT, 4-foot shoulders, 10-foot roadside recovery distance, 10-foot lane width, and a 4:1 sideslope, the expected single-vehicle accident rate is 79 (accidents/100 mvm). Widening this roadway to 11 feet would reduce the single-vehicle accident rate to 73, even if the resulting sideslopes were 2:1. Thus, in this example, one foot of lane widening at the expense of a steeper sideslope should not adversely affect the rate of single-vehicle accidents (although the overall accident severity may possibly be affected if, for example, more rollover accidents occur as a result of steepened sideslopes). While other types of comparisons can also be made using figure 3, the use of the predictive equation would allow for comparing the effects of sideslope changes on the single-vehicle accident rate versus lane and shoulder widening and roadside improvements.

Similar types of log-linear models were fitted using the rollover accident rate (AR) as the dependent variable. The best model for the rollover accident rate was

$$AR = 192.99\,(1.319)^{SS}(0.849)^{W}(0.983)^{RECC} \times (0.99984)^{ADT}(0.958)^{SW}$$

$$\bar{R}^2 = .25$$

where

AR = rollover accidents per 100 million vehicle miles
SS = 1 if sideslope is 4:1 or steeper, or zero otherwise;
all other terms are as previously defined.

This model has only two categories of sideslope, since no consistent trends were found in rollover rate for more defined sideslope groups. Note that in this model, a 4:1 sideslope was included with the steep (3:1 and 2:1 or steeper) group. This could indicate that sideslopes of 5:1 are more desirable than 4:1 slopes in preventing rollover accidents. Another explanation is that some vehicle types, such as mini-cars, are having a rollover accident problem on 4:1 sideslopes as well as on 3:1 and 2:1 slopes, which could partly account for the relatively high rollover accident rate for 4:1 sideslopes.

It should also be remembered that for each of the sample sections, the value of the sideslope used in the modeling was the 50th percentile (median value) of all of the field measurements for that section. A section labelled as having a 4:1 sideslope might actually consist of a range of sideslopes with 4:1 as the median value. Thus, in the database, each section labelled as 4:1 could have as much as 49 percent of the measurements steeper than 4:1 and the rest 4:1 or flatter. It is, therefore, quite possible that the 4:1 sideslope sections have rollover accident rates similar to the 3:1 and steeper category because these sections consist of a substantial portion of 3:1 and 2:1 sideslopes.

Rollover accidents represent only 23 percent of single-vehicle accidents (and only 8 percent of total accidents) in the database, so the relatively small samples of rollover accidents could have resulted in less reliable models than the models using single-vehicle accident rate. Also, the actual density of roadside fixed objects (such as trees) is generally greater on sections with steeper slopes than on sections with flat slopes. Thus, if a vehicle runs off the road onto the sideslope, it may hit an obstacle before having a chance to roll. Because of such considerations, it was believed that the rate of single-vehicle accidents was a better indication of sideslope effects than the rate of rollover accidents.

The single-vehicle accident model discussed earlier (and corresponding accident reductions) for various sideslopes provides perhaps the most reliable results currently available of sideslope effects on accidents. However, there remains considerable uncertainty relative to the precise rollover potential of various sideslopes (in conjunction with ditch types, height of fill, shoulder dropoff, etc.) for different vehicle characteristics.

Roadside Obstacle Types and Accidents

Another analysis involved determining the types of roadside obstacles that are most commonly struck on roads with various traffic volume conditions. The frequency of six types of fixed-object accidents for different ADT categories is summarized in table 6, based on data from six of the states in the current database. Utah accident data were not included because very few obstacle types were recorded in that state's accident file. Obstacle types other than trees, signs, utility poles, mailboxes, bridge ends, and guardrails were defined or recorded differently in different states, making tabulation of those types impossible.

Overall, the most frequently struck obstacles listed on table 6 were trees (14.8 percent) and utility poles (14.1 percent). This finding agrees with Jones and Baum (*10*) who cited these two obstacle types as among the most frequently struck fixed objects. Guardrail (9.6 percent), signs (6.5 percent), mailboxes (4.7 percent), and bridge ends (1.1 percent) were hit less frequently. The "other obstacle" category in table 6 includes all other obstacle types (including earth embankments) in addition to obstacles that were not specifically coded by the police officers.

For roads with ADTs of 4,000 or less, trees are the single most common type of obstacle struck. This may simply be the result of the fact that trees are generally the most common type of obstacle along low-volume rural roads. For roads with ADTs over 4,000, utility poles are the single most frequent type of fixed object struck, which is logical in view of the fact that higher volume roads are generally in the urban and suburban areas where utility poles are frequently placed near the roadway. Guardrail accidents accounted for less than seven

TABLE 6 FIXED-OBJECT ACCIDENTS BY ADT GROUP AND TYPE OF OBSTACLE STRUCK ON URBAN AND RURAL HIGHWAYS

ADT Group	Number of Accidents (Percent of accidents by ADT class)							
	Trees	Signs	Utility Poles	Mail Boxes	Bridge Ends	Guard Rail	Other Obstacles	Total FO Accs.
50-400	31(24.0)	6(4.7)	2(1.6)	2(1.6)	1(0.8)	5(3.9)	82(63.6)	129(100.0)
401-750	92(23.7)	20(5.2)	24(6.2)	10(2.6)	5(1.3)	20(5.2)	217(55.9)	388(100.0)
751-1,000	107(22.4)	9(1.9)	26(5.4)	6(1.3)	2(0.4)	33(6.9)	295(61.7)	478(100.0)
1,001-2,000	278(15.8)	95(5.4)	118(6.7)	46(2.6)	33(1.9)	192(10.9)	997(56.7)	1,759(100.0)
2,001-4,000	467(15.8)	200(6.8)	319(10.8)	144(4.9)	29(1.0)	319(10.8)	1,475(49.9)	2,953(100.0)
4,001-7,500	483(13.8)	235(6.7)	611(17.5)	198(5.7)	31(0.9)	323(9.3)	1,609(46.1)	3,490(100.0)
> 7,500	275(10.9)	198(7.9)	556(22.1)	145(5.8)	31(1.2)	239(9.5)	1,070(42.6)	2,514(100.0)
Total	1,733(14.8)	763(6.5)	1,656(14.1)	551(4.7)	132(1.1)	1,131(9.6)	5,745(49.1)	11,711(100.0)

Note: The data base includes 1,741 urban and rural sections in six states (excludes Utah).

percent of all fixed-object accidents on roads with ADTs of 1,000 or less, but they account for 9.3 to 10.9 percent of fixed-object hits for roads with ADTs of 1,001 or greater. The values in table 6 represent only the frequency of accidents and do not account for the placement or frequency (exposure) of these roadside objects.

It was impossible to determine the relative severity of accident types from the seven-state database, since data were aggregated by sections. However, accident data from the states of Michigan, Utah, and Washington were available for this analysis. These data include the rural two-lane roads, urban two-lane roads, and/or multi-lane roads. Nonetheless, the analysis afforded a reasonable look at the relative severity of different fixed-object (FO) accident types.

The severity of run-off-road fixed-object accidents relative to other common accident types was investigated, and the results are summarized in table 7. The percentage of FO accidents resulting in injury were 35, 36, and 44 for Michigan, Utah, and Washington, respectively. These percentages were lower than the percentages for rollover, head-on, and pedestrian/bicycle accidents; higher than the percentages for sideswipe opposite direction and sideswipe same direction; and about the same as the percentages for rear-end and angle accidents. The percentages of FO accidents resulting in a fatality were 0.8, 2.0, and 1.5 for Michigan, Utah, and Washington, respectively. These percentages again ranked FO accidents in the middle of the eight accident types shown in table 7. In terms of absolute numbers of injury accidents, however, FO accidents were the most frequent of the eight accident types in Michigan, the second most frequent in Washington, and the fourth most frequent in Utah. FO accidents were also the accident type most frequently associated with fatalities in Michigan and in Washington (fifth in Utah). In summary, FO accidents are both frequent and severe compared to other accident types.

The relative severity of the different types of fixed-object accidents is summarized by state in table 8. Fixed-object accidents which resulted in injuries generally ranged from 24 to 64 percent, depending on the type of object struck. Fatalities generally ranged from 0.2 to 6.1 percent. Among the objects associated with the highest percentage of injury and fatality were trees, culverts, bridges (bridge columns and bridge ends),

TABLE 7 SEVERITY OF COMMON ACCIDENT TYPES IN SEVERAL DATABASES

Accident Type	Percent of accidents within type resulting in injury or fatality			
	Accident Severity	State		
		Michigan	Utah	Washington
Run-off-road fixed object	Injury	35 (10137)	36 (827)	44 (15902)
	Fatal	0.8 (228)	2.0 (46)	1.5 (532)
Run-off-road rollover	Injury	55 (6587)	55 (1076)	56 (6488)
	Fatal	1.1 (73)	3.2 (63)	2.1 (245)
Head on	Injury	41 (1922)	50 (237)	60 (803)
	Fatal	2.7 (127)	11.9 (56)	20.4 (272)
Sideswipe Opposite dir.	Injury	21 (27)	30 (162)	41 (1118)
	Fatal	2.4 (3)	1.9 (10)	2.0 (54)
Sideswipe Same dir.	Injury	13 (42)	11 (87)	20 (2012)
	Fatal	1.6 (5)	0.2 (2)	0.2 (20)
Rear end	Injury	27 (2228)	33 (2320)	43 (21239)
	Fatal	0.3 (27)	0.2 (11)	0.2 (96)
Pedestrian or bicycle	Injury	86 (1769)	84 (654)	90 (2007)
	Fatal	7.0 (144)	7.8 (61)	9.8 (218)
Angle	Injury	46 (3145)	31 (2768)	37 (13272)
	Fatal	1.1 (78)	0.1 (55)	0.5 (174)

Note: The Michigan data base consisted of all reported accidents on rural roads in 1983. The Utah data base consisted of accidents reported from mid-1980 to mid-1985 on routes which had portions chosen as sections for the seven-state data base (and thus, included limited amounts of urban and multi-lane road accidents). The Washington data base consisted of all accidents reported in the State from 1980 through 1984.

( ) = The total numbers of accidents of the given type are in parenthesis.

TABLE 8 SEVERITY OF COMMON RUN-OFF-ROAD FIXED-OBJECT ACCIDENT TYPES IN SEVERAL DATA BASES

Accident Type	Percent of total accidents resulting in injury or fatality			
	Accident Severity	Data Base		
		Michigan	Utah	Washington
Utility/Light Pole	Injury	45 (3385)	39 (163)	47 (2282)
	Fatal	0.8 (58)	1.2 (5)	1.6 (75)
Guardrail	Injury	35 (1392)	42 (130)	41 (3403)
	Fatal	0.7 (28)	4.2 (13)	1.7 (144)
Sign	Injury	25 (1397)	24 (74)	40 (700)
	Fatal	0.4 (22)	1.3 (4)	1.4 (25)
Fence	Injury	28 (851)	35 (139)	40 (594)
	Fatal	0.2 (7)	1.0 (4)	1.7 (26)
Tree	Injury	47 (4419)		53 (984)
	Fatal	1.8 (171)		3.4 (64)
Culvert	Injury	49 (250)		64 (277)
	Fatal	3.3 (17)		2.1 (9)
Bridge Rail	Injury	41 (178)		41 (1060)
	Fatal	0.7 (3)		1.6 (42)
Bridge Column	Injury			54 (53)
	Fatal			6.1 (6)
Bridge End	Injury			53 (72)
	Fatal			5.2 (7)
Barrier Wall	Injury			41 (908)
	Fatal			0.5 (10)
Earth Embank-ment	Injury			53 (1793)
	Fatal			1.6 (55)
Rock	Injury			49 (891)
	Fatal			1.1 (21)
Mailbox	Injury			40 (132)
	Fatal			0.0 (0)
Fire Hydrant	Injury			30 (44)
	Fatal			0.7 (1)

Note: The Michigan data base consisted of all reported accidents on rural roads in 1983. The Utah data base consisted of accidents reported from mid-1980 to mid-1985 on routes which had portions chosen as sections for the seven-state data base (and thus, included limited amounts of urban and multi-lane road accidents). The Washington data base consisted of all accidents reported in the STate from 1980 through 1984.

( ) = The total numbers of accidents of the given type are in parenthesis.

rocks, utility poles, and earth embankments. Objects associated with the lowest percentages of injury and fatality were signs, mailboxes, fire hydrants, barrier walls, and fences. Trees, utility and light poles, guardrails, and earth embankments are the objects involved in the most FO injury and fatal accidents.

SUMMARY AND CONCLUSIONS

The purpose of this study was to determine the effects of various roadside features on accident experience. Detailed traffic, accident, roadway, and roadside data were collected on 4,951 miles of two-lane rural roads in seven states. Statistical analyses and log-linear modeling were used to determine the effects of various roadside and roadway features on single-vehicle and other related accident types. Roadside measures used in the analysis included a roadside hazard scale (a seven-point pictorial scale), the roadside recovery distance (clear zone distance), and field measurements of roadside side slope.

A reduction of one rating value on the seven-point roadside hazard scale (such as a five hazard rating to a four rating) due to a roadside improvement is estimated to result in a 19 percent reduction in related (*AO*) accidents. A 34 percent reduction in related accidents may be expected for a two-point reduction in hazard rating, a 47 percent reduction for a three-point decrease in roadside hazard rating, and a 52 percent accident reduction for a four-point decrease in hazard rating. Similar effects on accidents were found using a different predictive model when roadside recovery distance was increased. Reductions in related accidents were found to be 13 percent, 25 percent, 35 percent, and 44 percent, when the roadside recovery distance (as measured from the outside edge of shoulder to the nearest roadside obstacles or hazards) was increased on a section by an additional five feet, 10 feet, 15 feet, and 20 feet, respectively. These results were based on log-linear models that controlled for the effects of lane width, width of paved and unpaved shoulders, traffic volume, and terrain.

The effects of sideslope on accident experience were determined using a sample of 595 rural roadway sections (1,776 miles) in Alabama, Michigan, and Washington where field sideslope measurements were taken. Based on log-linear modeling that controlled for the effects of ADT, lane width, shoulder width, and roadside recovery distance, increased rates of single-vehicle accidents and rollover accidents were found for steeper sideslopes. The rate of single-vehicle accidents decreased steadily for sideslopes of 3:1 to 7:1 or flatter. However, only a slight reduction (2 percent) in single-vehicle accidents was found for a 3:1 sideslope compared to a sideslope of 2:1 or steeper. Expected reductions in single-vehicle accidents due to sideslope flattening ranged from 2 to 27 percent, depending on the sideslope in the before and after condition. For example, flattening sideslopes of 2:1 or steeper to 3:1, 4:1, 5:1, 6:1, or 7:1 or flatter would be expected to result in reductions in single-vehicle accidents of two percent, 10 percent, 15 percent, 21 percent, and 27 percent, respectively. Improvements to existing 3:1 sideslopes would reduce single-vehicle accidents by 8 percent, 19 percent, and 26 percent due to flattening them to 4:1, 6:1, and 7:1 or flatter, respectively.

Overall, trees and utility poles are the roadside fixed obstacles most often struck, while guardrails, signs, mailboxes, and bridge ends are less frequently struck. On roads with traffic volumes of 4,000 vehicles per day or less, trees are the obstacles most often struck, while utility poles are the obstacles most frequently struck on roadways with higher volumes. Roadside objects associated with the highest percentages of severe (injury plus fatal) accidents include culverts, trees, utility and light poles, bridges, rocks, and earth embankments, while signs, mailboxes, fire hydrants, barrier walls and fences were associated with lower percentages of severe accidents.

RECOMMENDATIONS

The results of this study clearly show the importance of roadside conditions on accidents for two-lane roads, and the safety effects of improving roadside conditions were quantified. It is recommended that highway agency officials use this information to determine where roadside improvements are justified. For example, on future 3R projects and highway reconstruction projects, the benefits of various roadside improvements should be determined using the information described in this paper. By estimating the costs for such roadside improvements such as sideslope flattening, removing trees, and relocating utility poles, the cost effectiveness may be determined.

Agencies could also consider the safety impacts of various roadside conditions when designing new highway segments, in order to minimize roadside hazards. Highway agencies should also be sensitive to highway sections where roadside improvements are feasible. In addition, when locations are identified which have an unusually high incidence of single-vehicle accidents, the accident reduction factors contained in this paper may be useful for computing expected accident benefits from roadside improvements and thus for weighing various project alternatives.

ACKNOWLEDGMENTS

This paper was based on joint study for the Federal Highway Administration and Transportation Research Board by Goodell-Grivas, Inc. The Highway Safety Research Center of the University of North Carolina served as a Subcontractor for the study. Appreciation is given to Justin True, the FHWA Contract Manager and John Deacon and Robert Skinner, who served as the TRB contract managers. Special thanks are also given to numerous state representatives in Alabama, Michigan, Montana, North Carolina, Utah, Washington, and West Virginia, who cooperated with the project team and provided necessary data and information. The field sideslope measurements used in the analysis were collected by Analysis Group, Inc., under a separate contract for FHWA. The authors assume responsibility for the accuracy of the information contained herein.

REFERENCES

1. S. A. Smith, J. Purdy, H. W. McGee, D. W. Harwood, A. D. St. John, and J. C. Glennon. *Identification, Quantification, and Structuring of Two-Lane Rural Highway Safety Problems and Solutions*. Volumes I and II, Report Nos. FHWA/RD/83/021 and 83/022, Federal Highway Administration, Washington, D.C., 1983.
2. C. V. Zegeer, J. G. Mayes, and R. C. Deen. *Cost-Effectiveness of Lane and Shoulder Widening of Rural, Two-Lane Roads in Kentucky*. Bureau of Highways, Kentucky Department of Transportation, 1979. Also in *Transportation Research Record 806*, TRB, National Research Council, Washington, D.C., 1981.
3. C. V. Zegeer, J. Hummer, L. Herf, D. Reinfurt, and W. Hunter. *Safety Effects of Cross-Section Design for Two-Lane Roads*. FHWA/RD/87/008, Federal Highway Administration, Washington, D.C., 1986.
4. F. D. Newcomb, and D. B. Negri. *Motor Vehicle Accidents Involving Collision with Fixed Objects*. New York State Department of Motor Vehicles, 1971.
5. E. A. Rinde. *Conventional Road Safety: Phase 1—A Study of Fixed Objects*. California Department of Transportation, 1979.
6. T. J. Foody and M. D. Long. *The Specification of Relationships Between Safety and Roadway Obstructions*. Report No. OHIO-DOT-06-74, Ohio Department of Transportation, 1974.
7. J. W. Hall, C. J. Burton, D. G. Coppage, and L. V. Dickinson. Roadside Hazards on Non-Freeway Facilities. In *Transportation Research Record 601*, TRB, National Research Council, Washington, D.C., 1976.
8. C. V. Zegeer and M. R. Parker. *Cost-Effectiveness of Coun-*

termeasures for Utility Pole Accidents. Report No. FHWA/RD/87/008, Federal Highway Administration, Washington, D.C., January 1983.
9. K. K. Mak and R. L. Mason. *Accident Analysis—Breakaway and Non-breakaway Poles Including Sign and Light Standards Along Highways*. Federal Highway Administration, Washington, D.C., 1980.
10. I. S. Jones and A. S. Baum. *An Analysis of the Urban Utility Pole Problem*. Federal Highway Administration, Washington, D.C., 1980.
11. J. L. Graham and D. W. Harwood. *NCHRP Report 247: Effectiveness of Clear Recovery Zones*. TRB, National Research Council, Washington, D.C., 1982.
12. J. D. Weaver and E. L. Marquis. Roadside Slope Design for Safety. *Transportation Engineering Journal*, American Society of Civil Engineers, 1976.
13. K. Perchonok, T. A. Ranney, S. Baum, D. F. Morrison, and J. D. Eppick. *Hazardous Effects of Highway Features and Roadside Objects*. Calspan Field Service, Inc., Federal Highway Administration, Washington, D.C., 1978.
14. L. Graf, J. U. Boos, and J. A. Wentworth. Single-Vehicle Accidents Involving Utility Poles. In *Transportation Research Record 571*, TRB, National Research Council, Washington, D.C., 1976, pp. 36–43.
15. T. C. Edwards, J. E. Martinez, W. F. McFarland, and H. E. Ross. *NCHRP Report 77: Development of Design Criteria for Safer Luminaire Supports*. HRB, National Research Council, Washington, D.C., 1968.
16. J. C. Glennon and C. J. Wilton. A Methodology for Determining the Safety Effectiveness of Improvements on All Classes of Highways. *Volume I: Effectiveness of Roadside Safety Improvements*. Report No. FHWA/RD/75/23, Federal Highway Administration, Washington, D.C., 1974.
17. D. E. Cleveland and R. Kitamura. Macroscopic Modeling of Two-Lane Rural Roadside Accidents. In *Transportation Research Record 681*, TRB, National Research Council, Washington, D.C., 1978.

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

Intersection Channelization Guidelines for Longer and Wider Trucks

DANIEL B. FAMBRO, JOHN M. MASON, JR., AND NANCY STRAUB CLINE

Turning characteristics of large trucks, such as offtracking (the difference in paths of the front-most and rear-most inside wheels of a vehicle as it negotiates a turn) and swept-path width (the amount of offtracking plus the width of the truck), require special consideration in the design of at-grade intersections. Five large truck combinations, representative of the longer and wider trucks permitted by the Surface Transportation Assistance Act of 1982, were selected and their paths computer-simulated traversing different turning radii at several angles of turn. The findings were tabulated as guidelines for intersection channelization designed to accommodate these longer and wider trucks. The results include several specific truck turning templates; tables containing cross-street width occupied and swept-path width for various combinations of design vehicle, curb radii, and degree of turn; and recommended guidelines that illustrate conditions where channelization is feasible when designing for larger trucks. These guidelines include the minimum required curb radii to eliminate encroachment into either opposing or adjacent traffic lanes on the cross-street, the minimum required width of turning roadway, and the approximate size of the space available for channelizing islands.

The introduction of larger and heavier trucks into the traffic stream by recent federal and state legislation has prompted research by the Texas State Department of Highways and Public Transportation (SDHPT) on how to accommodate these vehicles on their highway system. Consequently, the Texas Transportation Institute (TTI) and the Center for Transportation Research (CTR) studied the impact of these larger vehicles on geometric design, traffic operations, and highway safety. The first objective of this study, an annotated bibliography summarizing research concerning operational characteristics and geometric design implications of longer and wider trucks, has been completed and published as TTI Research Report 397-1 (*1*). Another objective, involving the development of channelization guidelines to accommodate longer and wider trucks at at-grade intersections, is the subject of this paper. Related research results are documented in other reports (*2–4*).

Turning characteristics of large trucks, such as offtracking and sweptpath width, require special consideration when designing at-grade intersections. If the curb radius is large enough for trucks to make right turns without encroaching on adjacent lanes, the paved area at the intersection can become so large that through drivers may not understand where to position their vehicles. In such instances, it becomes necessary to construct a channelizing island to properly control traffic. If the curb radius is so small that trucks cannot make right turns without encroaching on adjacent lanes, the truck either encroaches and interferes with adjacent traffic, or its rear wheels run over and possibly damage the curb and/or shoulder. In addition, the front overhand of the truck may strike traffic control devices located near the outside of its turning path, or the right rear trailer tire may strike devices located near the inside of its turning path when offtracking. Turning characteristics of large trucks in a left-turn maneuver must also be considered in the design process; this paper, however, presents the findings of an investigation regarding only right-turn maneuvers.

The objective of the study was to establish a set of guidelines for channelization that would accommodate selective longer and wider trucks at at-grade intersections. To accomplish this objective, the following tasks were performed:

- Reviewed literature concerning truck turning characteristics and intersection channelization;
- Determined truck turning characteristics for various combinations of large design vehicle and intersection geometry; and
- Developed guidelines for design, operation, and channelization of at-grade intersections to accommodate these larger vehicles.

TRUCK TURNING CHARACTERISTICS

Because of a truck's long wheelbase, its rear wheels do not follow the same path as its front wheels when making a turn. The differences in these paths is defined by the terms offtracking and swept path. Offtracking is generally defined as the difference in paths of the front-most inside wheel and rear-most inside wheel of a vehicle as it negotiates a turn (*5*). The distance may also be measured between the tracking of the front and rear outside wheels, or the center of the front and rear axles, but its value will be the same. Offtracking is known to vary directly with the wheelbase of a unit and inversely with the radius of turn. Its magnitude "is affected in combination by the number and location of articulation points, by the length of the arc and the type of curve, and by the speed and turnability of the wheels" (*6*, p. 73).

Swept-path width may be defined as the amount of offtracking plus the width of the truck. It can also be defined as the difference in paths of the front-most outside wheel and

D. B. Fambro, Texas Transportation Institute, Texas A&M University, College Station, Tex. 77843. J. M. Mason, Jr., Department of Civil Engineering, The Pennsylvania State University, 212 Sackett Building, University Park, Pa. 16802. N. S. Cline, City of Dallas, 320 East Jefferson, Dallas, Tex. 75203.

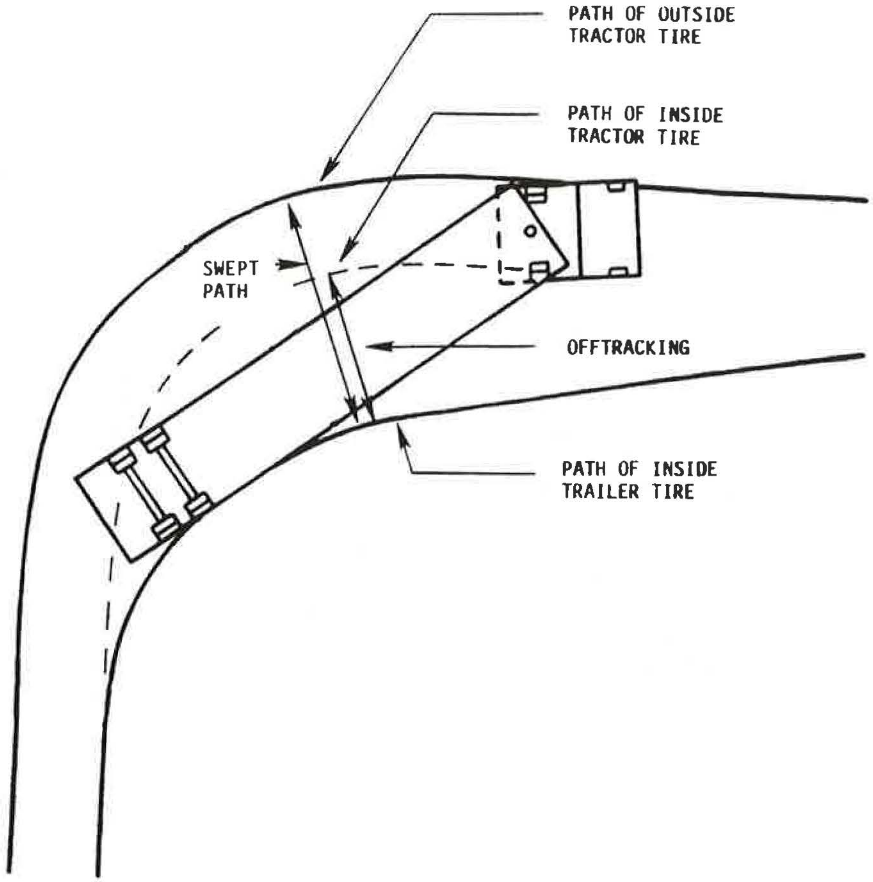


FIGURE 1 Swept-path width and offtracking of a truck negotiating a 90-degree turn.

the rear-most inside wheel of the vehicle as it negotiates low-speed turns. At higher speeds, negative offtracking may occur; that is, the rear-most wheels may actually travel outside the path of the front-most wheels because of side slippage. In this case, swept path would be defined as the difference in paths of the front-most inside wheel and rearmost outside wheel of the vehicle as it negotiates the turn. With the exception of negative offtracking, these terms are illustrated in figure 1 (*7*).

Full-Scale Tests and Formulas

Full-scale tests done on test-track curves of known radius were one of the first methods used to determine offtracking. The tests were extremely accurate because they involved professional drivers and an actual vehicle traversing a measured turn. These tests, however, were based on the assumption that other drivers could repeat this optimum performance in the real world. Also, this method of testing was expensive, and the number of truck-turn combinations that would have to be tested made it necessary to develop less expensive, yet equally reliable methods.

Scale Modeling

Scale modeling was found to be more efficient than working with the actual vehicles. The Tractix Integrator, an instrument used to simulate actual vehicle offtracking characteristics, has been used to develop turning templates for a number of different design vehicles (*5*). The Tractix Integrator provides an immediate plot of the truck path and is especially well-suited for many roadway design situations. Its use, however, is relatively slow and tedious, and special points of interest must be manually added to the centerline paths.

Computer Models

The first computer model that simulated vehicle offtracking was developed by the University of Michigan Transportation Research Center (UMTRI). This modeling package was quite an advancement in working with vehicle offtracking when compared to the previously described methods for studying turning characteristics. The program was developed for a microcomputer environment and designed to be user-friendly. The program output was a scaled plot of the paths followed by the vehicle tires in a format that could be overlaid on drawings of intersections or other situations involving restrictive geometry.

The Truck Offtracking Model (TOM), developed by the California Department of Transportation (Caltrans), is most frequently used for trucks, although it also simulates the offtracking characteristics of any vehicle combination when making a turn (*8*). TOM evolved from the Apple II personal computer offtracking model developed by UMTRI, and the simulation portion of the Apple program was adopted by

Caltrans' Division of Transportation Planning and placed on the state's IBM mainframe computer. TOM was not as user-friendly as the Apple version, but its plotting capacity was much greater, resulting in plots of larger scale and higher quality.

INTERSECTION CHANNELIZATION

At-grade intersection channelization is defined as the separation or regulation of conflicting traffic movements into definite paths of travel by the use of pavement markings, raised islands, or other suitable means to facilitate the safe and orderly movements of both vehicles and pedestrians (*9*). Proper channelization increases capacity, improves safety, provides maximum convenience, and instills driver confidence. Improper and/or over-channelization often have the opposite effect and should be avoided because of the confusion they can cause (*10*). Currently, there are no guidelines for intersection channelization when larger trucks are the design vehicles. The following literature review highlights several references that address channelization at at-grade intersections.

The Highway Research Board (HRB) sponsored two publications on intersection channelization containing examples and critical analyses so that highway and traffic engineers might benefit from a review of other works. Special Report 5 (*10*) provided fifty-nine examples of channelized intersections as of 1952. A revision by the same title was published in 1962 as Special Report 74 (*11*) and provided more examples of channelization to illustrate design practice as of that date. This report also defined the special objectives of intersection channelization, which are to assure orderly movement, increase capacity, improve safety, and provide maximum convenience.

The most recent publication dealing with channelization is a 1986 version of Special Report 74 (*12*), which includes illustrative examples of channelization designs and more detailed guidelines than were provided in the earlier reports. In addition, the report covers channelization of both new and reconstructed intersections in urban and rural environments. Its contents include typical intersection types such as four-way, Y, T, oblique, and multi-leg intersections, as well as freeway ramp intersections with surface streets.

The American Association of State Highway and Transportation Official's (AASHTO's) Policy on the Geometric Design of Highways and Streets—1984 ("Green Book") contains discussions of both offtracking and channelization. The book specifies that larger semitrailer combinations should be used as design vehicles where truck combinations approximating this size will turn repeatedly. Such designs, particularly when used in two or more quadrants of an at-grade intersection, produce large paved areas that may be difficult to control. It is usually desirable to channelize such intersections, requiring larger radii (*7*).

DESIGN VEHICLES

The design vehicles selected for this study were two singles, two doubles, and one triple. They are typical of the larger vehicles currently being operated on the nation's highways. One of the vehicles, the WB-50, was the same as one of the design vehicle configurations defined in the "Green Book" (*7*) and was used to check the study results for accuracy and consistency. The tractor used in each combination had a 16-foot wheelbase with the cab placed behind the engine. This particular tractor was selected because of its longer wheelbase, typical of cab-behind-engine tractors. The five design vehicles are described below, and their dimensions are shown in table 1.

Singles

The first design vehicle, the WB-50, represents the design vehicle with the worst turning characteristics of those contained in the "Green Book." As of 1984, the WB-50 was nearly all-inclusive of the tractor-semitrailer combinations in use. The tractor and trailer in the WB-50 have wheelbases of 16 and 34 feet respectively, with an overall combination length of 50 feet from the front-most axle to the rear-most axle. The WB-55, a larger single, was the second design vehicle selected for the study. Its tractor has a 16-ft wheelbase, and its 48-ft trailer has a 38.5-ft effective wheelbase, for an overall wheelbase of 56 feet from the front-most to rear-most axles. The WB-55 represents the longest single trailer vehicle allowed by the Surface Transportation Assistance Act (STAA) of 1982.

Doubles

The third design vehicle was the WB-70 with a 16-ft tractor, two 28-ft trailers, and an overall wheelbase spacing of 70 feet. It is sometimes referred to as the "western double" and is slightly larger than the WB-60 design vehicle used in the "Green Book." The fourth design vehicle, the WB-105, is frequently referred to as the "turnpike double" and represents, in some western states, the maximum allowable trailer lengths for combination vehicles. It consists of a 16-ft tractor towing two 48-ft trailers, for an overall length of 105 feet. The WB-105 is the most critical of the five design vehicles because, as is discussed later, it has the worst turning characteristics of the vehicles studied.

Triple

The fifth design vehicle, the WB-100, was a tractor-trailer combination with three 28-ft trailers behind a 16-ft tractor, resulting in an overall length from front-most axle to rear-most axle of 100 feet. Because of these relatively short wheelbases, the WB-100 can turn much sharper radii than the WB-105 without encroaching; because of its numerous articulation points, however, its swept path is much greater.

INTERSECTION GEOMETRICS

In addition to the design vehicle, the other parameters investigated in this study were curb return radius and degree of turn. The values for curb return were as specified in table III-19 in the "Green Book." A radius of 25 feet was included in addition to the values in the table of 50, 75, 100, 150, and 200 feet. These radii were drawn to a scale of 1 inch equals 20 feet on sheets of clear mylar so that turning paths of the

TABLE 1 DESIGN VEHICLE DIMENSIONS

		Dimensions (ft)													
		Overall			Overhang										
Design Vehicle Type	Symbol	Ht.	Width	Length	Front	Rear	WB_1	WB_2	S	T	WB_3	S	T	WB_4
Combination Trucks:														
Semitrailer	WB-50	13.5	8.5	55	3	2	16	34.0	---	---	---	---	---	---
Large Semitrailer	WB-55	13.5	8.5	60	3	2	16	39.1	---	---	---	---	---	---
Semitrailer-trailer	WB-70	13.5	8.5	75	3	2	16	20	2.5	7.5	23.0	---	---	---
Large Semitrailer-trailer	WB-105	13.5	8.5	110	3	2	16	37.3	6.7	6.3	37.8	---	---	---
Semitrailer-trailer-trailer	WB-100	13.5	8.5	105	3	2	16	21.9	3.0	6.2	22.3	3.0	6.2	22.3

WB_1, WB_2, WB_3, WB_4 are effective vehicle wheelbases.
S is the distance from the rear effective axle to the hitch point.
T is the distance from the hitch point to the lead effective axle of the following unit.

design vehicle could be superimposed on an intersection layout.

A 2-ft clearance was desirable between the curb radius and the vehicle travel path. Therefore, the actual radii were drawn at 27, 52, 77, 102, 152, and 202 feet, respectively. Another result of the 2-foot clearance was that the lane lines (normally 12 feet) were drawn at 10 feet to show the effective lane width. Sets of the various radii were drawn for turning angles of 60, 75, 90, 105, and 120 degrees, as they were considered to be representative of typical intersection geometry. In addition to the typical angles of turn, a 180-degree turn was simulated for completeness and to define the minimum possible turning radius for each design vehicle.

SIMULATION MODEL

The California Truck Offtracking Model (TOM) (*13*) originally written for an IBM mainframe computer, was modified to run on a VAX 11/750 computer. A brief discussion of required input and resulting output follows.

Inputs

There are five input cards or lines of data that supply the necessary information to the offtracking program. The critical path geometry, described below, is input on card 1. The data on card 2 is the vehicle configuration, that is, the number of units and axle spacing. The simulation parameters, initial x- and y-coordinates and distance increments for simulation calculations, are input on card 3. Card 4 includes all the plotting data necessary to specify the number of paths and additional reference points to be plotted and to define the area in which the paths are to be plotted. The title information is given on card 5.

The critical path geometry in the computer input data stream is the radius of curvature for the turning vehicle and the angle of turn. Computer runs were made for each design vehicle making turns of 60, 75, 90, 105, and 120 degrees. These turns were made at the minimum radius possible (to within 5 feet) and increased at intervals of 10 to 15 feet depending on the minimum turning radius of the design vehicle. The minimum radius was determined by the method described by AASHTO in 1965 (*14*), which states that "the minimum turning radii for the design vehicles (WB-40 and WB-50) was largely determined by the paths of the inner rear wheels." The turning path chosen was one that would result in a minimum radius of the inner rear wheel track of approximately 19 feet when negotiating turns of 90 to 180 degrees.

Outputs

The outputs of TOM were printouts detailing the input values, a table listing offtracking at the beginning of curve (BC), end of curve (EC), the point of maximum offtracking (MOT), and

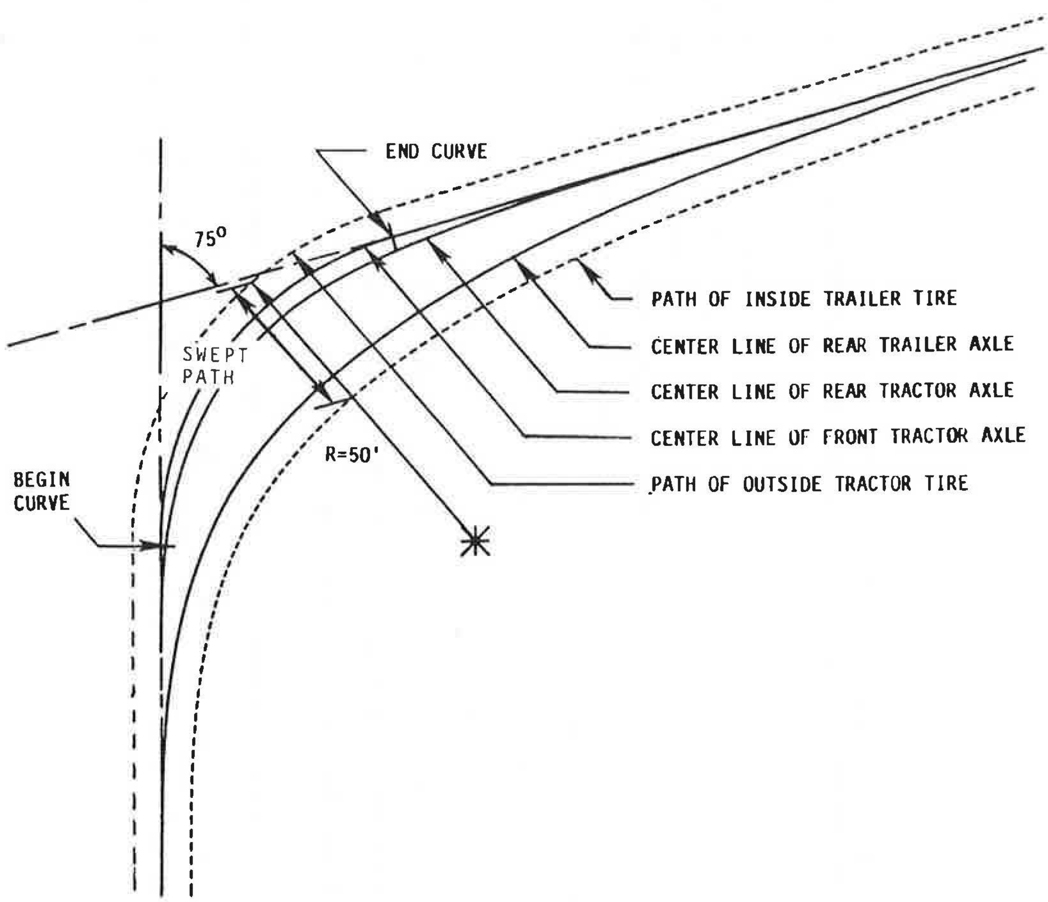


FIGURE 2 Example plot from truck offtracking model (WB-55, 75-degree turn, 50-foot radius).

the plot of the vehicle turning path (see figure 2). It was necessary to modify the plot routine to work with the HP plotter connected to the VAX 11/750 computer used in this study. For convenience as well as comparative purposes, plots were made at a scale of 1 inch equals 20 feet.

The output of the Truck Offtracking Model was verified by preparing a turning template for a vehicle configuration that closely matched that of the WB-50 design vehicle shown in the Leisch turning templates (*15*). A second template was made and compared to a vehicle modeled using the Tractix Integrator (*16*). Both templates drawn by the model closely matched the Leisch and Tractix Integrator templates.

DATA ANALYSIS

The optimum turning radius for each curb return was defined as the smallest turning radius that the design vehicle could negotiate without running over the inside curb, while at the same time minimizing cross street encroachment; that is, the design vehicle's minimum turning radius until the curb return became large enough to allow the vehicle to turn on a longer radius. For each vehicle-geometric combination, the following design parameters were determined:

Cross Street Width Occupied

The cross street width occupied was defined as the amount of encroachment plus a 12-ft lane width (see figure 3). Encroachment was defined as the distance that the vehicle trespassed beyond the 12-ft lane stripe in order to complete its turn. It was assumed that the vehicle positioned itself to the far left of the right-most lane on the approach street and only swung wide when on the cross-street; in other words, the vehicle remained within the 12-ft lane lines when approaching the turn.

Swept-Path Width

Swept-path width was defined earlier as the difference in paths of the front-most outside wheel and rear-most inside wheel of a vehicle as it negotiated a turn (see figure 3). Swept-path width could also be defined as the offtracking plus the width of the vehicle, and it is important in determining the minimum width of turning roadways. The maximum swept-path widths measured from the plots generally agreed with the offtracking values from the computer printouts.

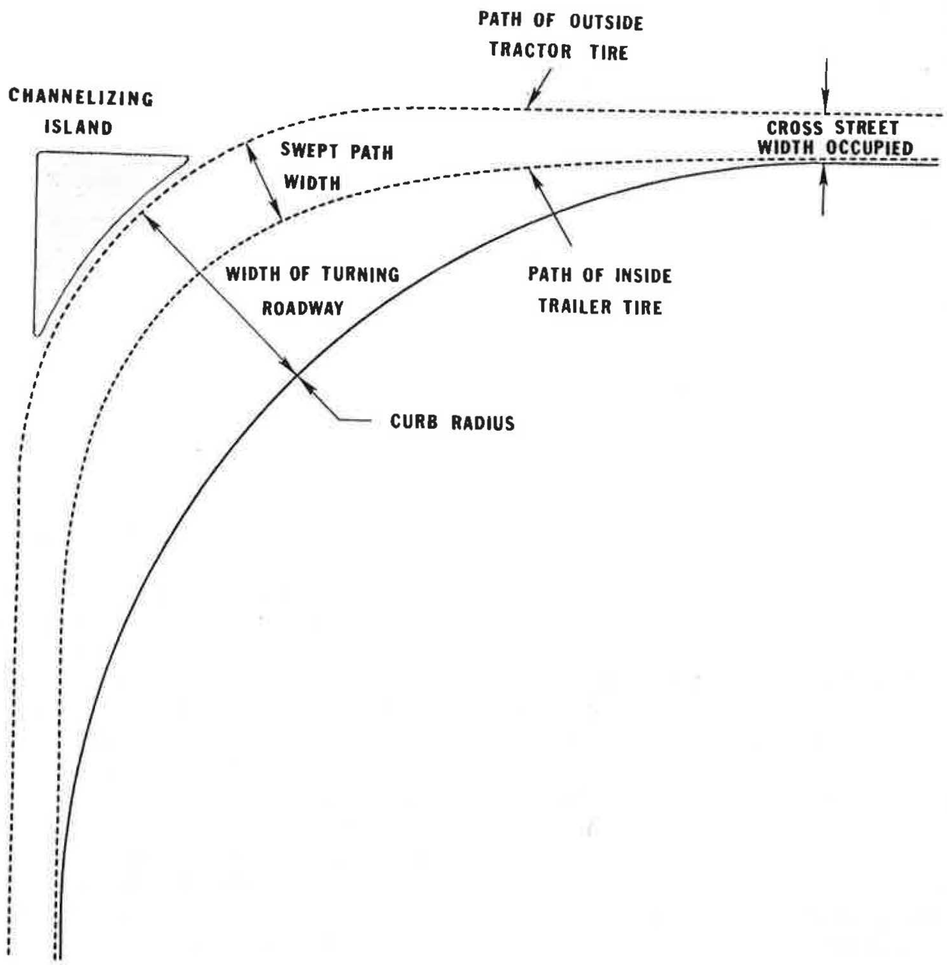


FIGURE 3 Cross street, swept path, and turning roadway width for a truck negotiating a 90-degree intersection turn.

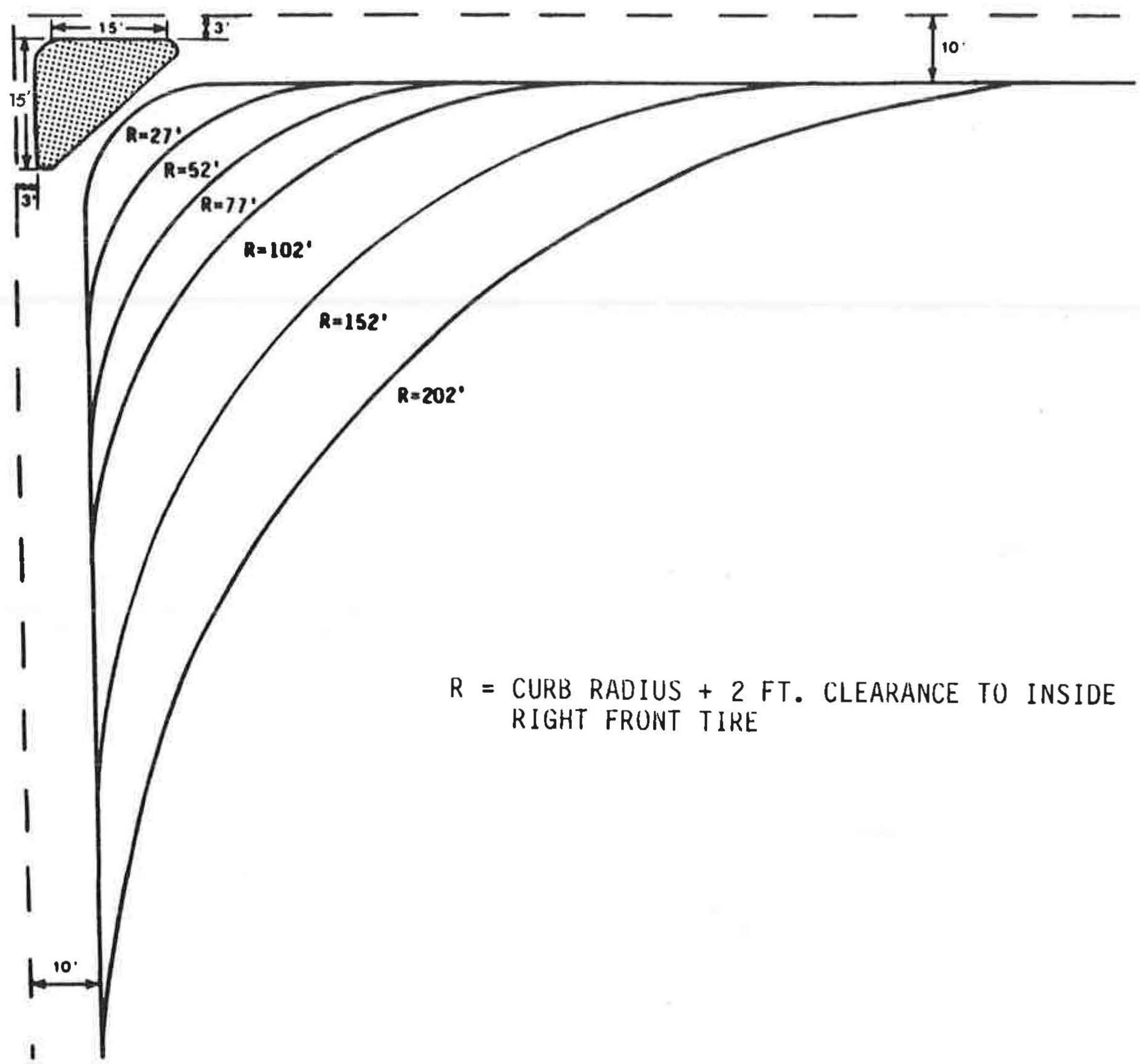


FIGURE 4 Curb radii and minimum island size for a 90-degree intersection turn.

Channelization

The critical design consideration in deciding whether or not to use channelization was the curb return radius of the intersection. It was not the same as the turning radius of the vehicle. In order to determine where there was enough pavement area to justify channelization, the 12-ft lane lines on each street were extended until they intersected. An island was then drawn with the curb radii that would satisfy the preferred criteria in the "Green Book"—3-ft offset from through traffic, 3-ft corner radii, and minimum leg lengths of 15 feet (7) (figure 4).

In order to determine the values for each vehicle-geometric combination, the turning templates were grouped first according to the design vehicle and then according to the angle of turn. For each design vehicle at each angle of turn, the minimum turning path (determined from the 180-degree turns) was placed over the 27-ft curb radius at the same angle of turn. Wheel paths of the vehicles could lie on the line, offset two feet from the curb, because of the allowances previously described. The amount of encroachment beyond the 12-ft lane line was measured at the end of the turning curve, EC, as this was the point where the truck began moving back into its lane. The assumption was made that the vehicle turned from the proper lane of the approach street, and, therefore, all of the encroachment occurred in the cross-street lane. No allowances were made in the simulation for shoulders for the truck to encroach upon.

As the curb radius was increased, the minimum turning path became too tight, and it was necessary to go to a larger turning path. Preferably, the turning path that encroached the least or not at all was the one chosen. If, for example, both the 60-ft turning radius and the 75-ft turning radius could each turn a 150-ft curb radius without encroaching, then the 75-ft turning path would be selected because it had a smaller swept width.

STUDY RESULTS

Computer simulation runs were made for each of the different scenarios, and the resultant output was converted to a more comprehensive format. The study results were broken down into the following five topic areas:

Minimum Turning Radii

The boundaries of the turning paths for a design vehicle making its sharpest possible turn were established by the paths followed by its outer front wheel and inner rear wheel as it made the turn. The minimum turning radii of the outside and inside wheel paths for each of the five design vehicles are given in table 2. The values for the WB-50 vary slightly from those in the "Green Book" due to shorter tractor and longer trailer axle spacings. The minimum turning radii and the transition lengths shown here and in the "Green Book" are for turns made at less than 10 mph. This assumption minimizes the effects of driver characteristics (such as the rate at which the driver approaches centripetal acceleration) and the slip angles of wheels.

Turning Templates

Turning templates for each of the five design vehicles were developed using their minimum turning paths for various angles of turn: 60, 90, 120, and 180 degrees (figures 5–9). For each of the four angles of turn, templates were prepared by drawing each design vehicle on a sheet of mylar and then tracing its turning path onto the same sheet. They were originally drawn at a scale of one inch equals 20 feet, and three of them, figures 4 through 6, were reduced for inclusion in this paper.

Cross Street Width Occupied

Table 3 illustrates the effect of the angle of intersection on turning paths of various design vehicles on streets without parking lanes. It was structured similarly to table IX-3 in the "Green Book." Dimensions d_1 and d_2 were defined as the widths occupied by the turning vehicle on the main street and cross street, respectively, while negotiating turns through various angles. Both dimensions are measured from the right-hand curb to the point of maximum encroachment on either adjacent or opposing lanes (figure 10). These widths generally increase with increasing angle of turn and decrease with increasing curb radii. The right-turn maneuver modeled in this study assumed that the vehicle positioned itself to the far left of the right-most lane on the approach street and only swung wide when on the cross-street. This assumption results in the worst case scenario on the cross-street. Therefore, the dimension d_1 equals 12 feet, and d_2 is the value shown in table 3.

The values for the WB-50 design vehicle in table IX-3 of the "Green Book" should have closely resembled values for the WB-50 vehicle used in this study. The values from table IX-3 in the "Green Book" indicate that AASHTO WB-50 has less severe turning characteristics than the WB-50 with a slightly shorter tractor wheelbase that was used in this study. It should be remembered, however, that a longer semitrailer wheelbase is associated with a shorter tractor wheelbase. Since offtracking is a function of the sum of the squares of the different wheelbase lengths, a decrease in a short wheelbase will be more than offset by a corresponding increase in a long wheelbase. Thus, the larger values of cross-street width occupied are consistent with the theory.

Assuming a road with two 12-ft lanes in either direction, a truck must be able to turn without occupying more than 24 feet of the cross street width. Referring to table 3, none of the vehicles can negotiate any of the turning angles (60 to 120 degrees) at either a 25-ft or a 50-ft curb radius without occupying more than 24 feet of cross street width. At a 75-ft curb radius, however, all of the vehicles except the WB-100 (triple) and the WB-105 (turnpike double) can make the turns and stay within 24 feet of cross-street width. These two larger vehicles can make the turn within the stated constraints at radii of 100 and 150 feet, respectively.

If the example were modified and there were a 10-ft shoulder or parking lane provided on the cross street, the available cross street width would be 34 feet. Under these circumstances, the less critical design vehicles (WB-50, WB-55, and WB-70) could turn at 50-ft curb radii, and the WB-100 could turn

TABLE 2 MINIMUM TURNING RADII OF DESIGN VEHICLES

Design Vehicle Type	Semitrailer Combination	Semitrailer Combination (Large)	Semitrailer-Full Trailer Combination	Semitrailer-Full Trailer Combination (Large)	Semitrailer Full Trailer-Full Trailer Combination
Symbol	WB-50	WB-55	WB-70	WB-105	WB-100
Configuration	3-S2	3-S2	3-S1-2	3-S2-4	2-S1-2-2
Minimum Turning radius (ft.)	45	50	50	65	55
Minimum Inside radius (ft.)	20.5	19	24.3	25.8	25.6

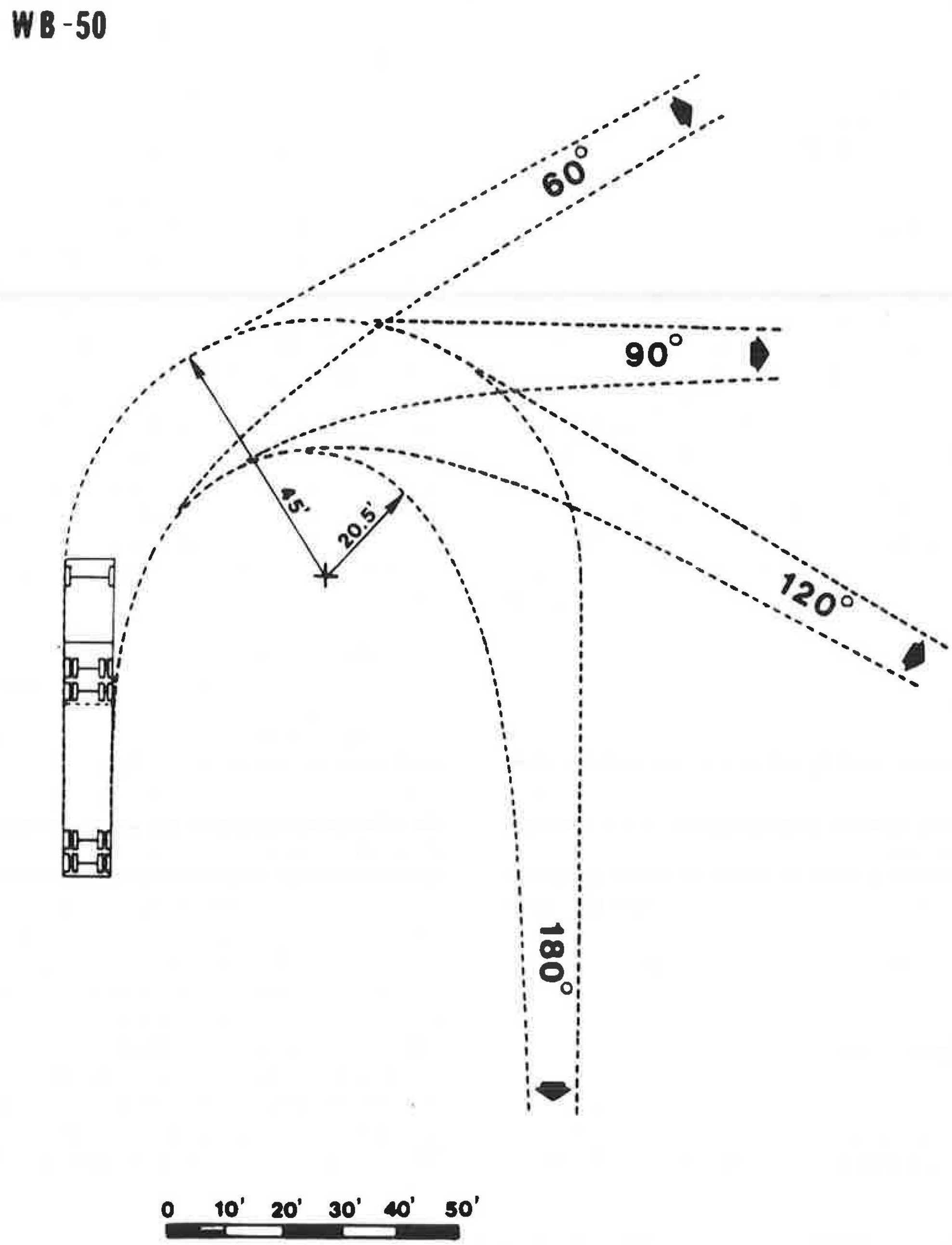


FIGURE 5 Minimum turning paths for WB-50 design vehicle.

the smaller angle turns at a 75-ft radius and all angles at the 100-ft curb radii. The most critical design vehicle (WB-105), however, can still only turn 150-ft and 200-ft curb radius turns. As the angle of turn increases past 90 degrees, the turning problems of the WB-105 become much more pronounced, especially at a 105-degree, 150-ft curb radius where the other four vehicles maneuver well.

Turning Roadway Width

Table 4 contains the values for the swept width of the various design vehicles shown for various angles of turn and curb radii. The swept width is a function of the optimum turning radius of the vehicle at a certain angle and curb return. By close inspection of table 3, it was possible to determine the point at which the minimum turning radius of each design vehicle reached the point where it was no longer the optimum, and a larger turning radius (with a smaller swept width) could negotiate the curb radius equally well, if not better, than the minimum. This point was identified by the decrease in the swept width for a particular design vehicle at a certain degree of turn as the radius increases. The 65-ft minimum radius of the turnpike double was never replaced by a greater radius as the curb radii increased up to 200 feet.

The greater the swept width of a vehicle negotiating a turn, the greater the width of turning pavement necessary. Although the "Green Book" classifies pavement widths for turning

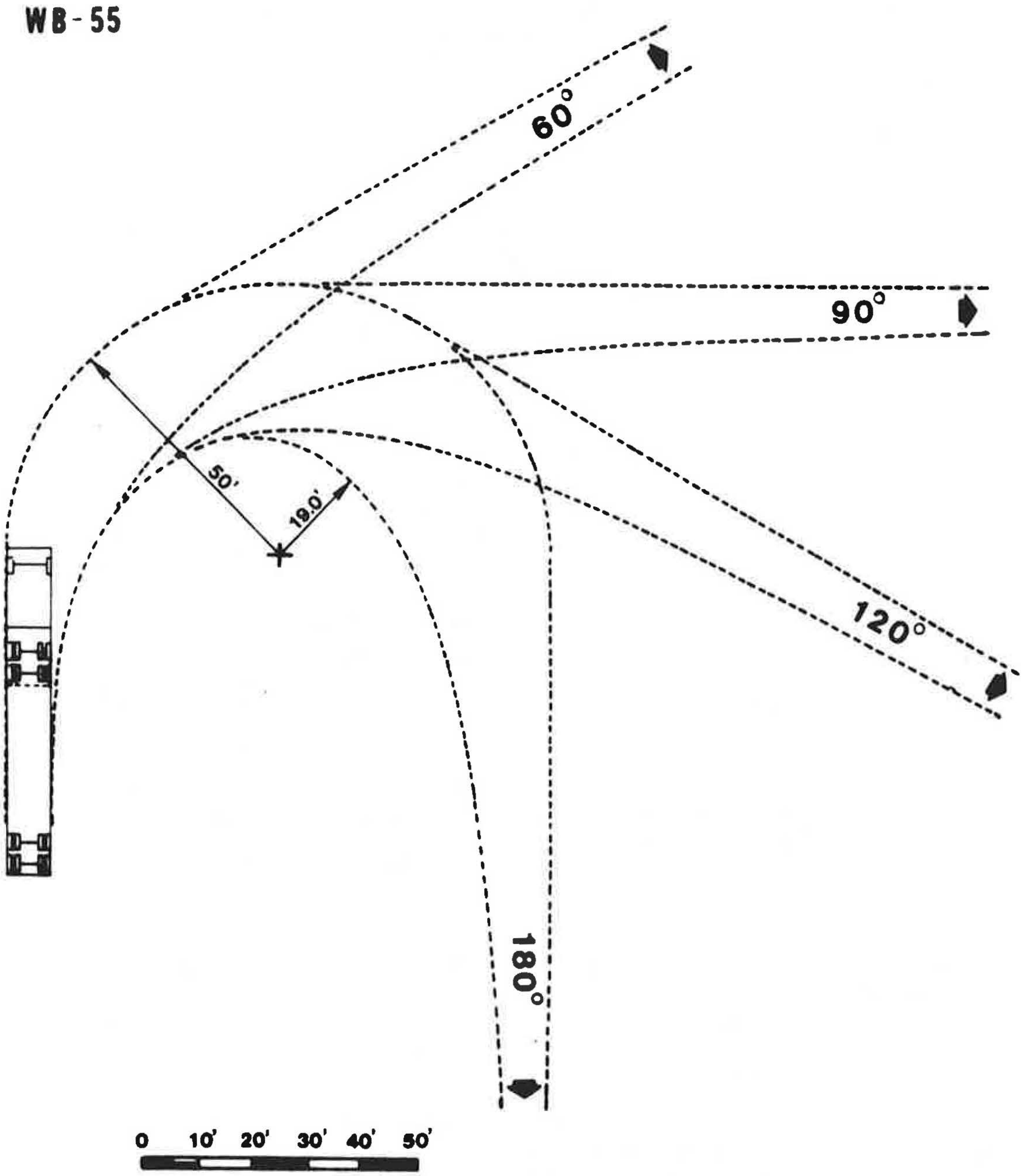


FIGURE 6 Minimum turning paths for WB-55 design vehicle.

roadways for several types of operations, Case 1—one-lane, one-way operation with no provision for passing a stalled vehicle—is the type of operation that was considered in this study.

The WB-105 (turnpike double) had a wheelbase just slightly longer than the WB-100 (triple); its swept path width, however, was much greater due to its greater axle spacings. The sum of the squares of axle spacings and the number of points of articulation govern the way a vehicle will offtrack around a curve. The number of articulations will affect the shape of the curve, while the sum of the squares will determine the magnitude of offtracking (*5*). Because of this, the two 48-ft trailers of the turnpike double cause more severe offtracking than the 28-foot trailers of the triple.

Channelization Guidelines

The boxed-in area in both tables 3 and 4 are the conditions where the curb radius combines with the optimum turning radius in such a way as to leave room for an island of at least 100 square feet in size, the minimum size of channelized island recommended by the "Green Book" (*7*). Conditions where channelization is feasible are the larger curb radii and frequently the larger degrees of turn. Channelization is recommended at a 200-ft curb radius for all of the vehicles except the turnpike double at 60- and 70-degree turns. As the curb radius decreases, the angle of turn, in combination with the design vehicle, influences whether channelization is feasible. Overall, as the angle of turn increases beyond 90 degrees, the

TABLE 3 CROSS STREET WIDTH OCCUPIED BY TURNING VEHICLE FOR VARIOUS INTERSECTION ANGLES AND CURB RADII

Angle of Turn (Degrees)	Design Vehicle	Curb Radius 25 ft.	50 ft.	75 ft.	100 ft.	150 ft.	200 ft.
60	WB-50	33.5	24.0	17.0	14.0	12.0	12.0
	WB-55	40.0	29.8	21.5	17.3	13.0	12.0
	WB-70	38.8	23.7	19.5	15.0	12.0	12.0
	WB-100	46.5	36.2	27.0	18.0	12.0	12.0
	WB-105	56.0	46.5	37.0	29.0	18.0	12.0
75	WB-50	37.0	26.0	16.5	13.5	12.0	12.0
	WB-55	44.0	34.7	21.5	16.8	12.0	12.0
	WB-70	43.0	34.0	20.0	14.5	12.0	12.0
	WB-100	52.0	41.0	28.5	17.0	12.0	12.0
	WB-105	65.0	36.0	42.5	30.0	17.0	12.0
90	WB-50	43.0	26.0	17.0	13.0	12.0	12.0
	WB-55	53.0	37.0	21.8	17.0	13.0	12.0
	WB-70	53.0	36.0	21.0	14.0	12.0	12.0
	WB-100	66.0	46.5	31.0	17.5	12.0	12.0
	WB-105	81.0	63.0	48.0	33.0	17.3	12.0
105	WB-50	52.0	32.0	18.0	13.0	12.0	12.0
	WB-55	62.0	42.0	23.5	18.0	12.0	12.0
	WB-70	61.5	42.0	23.0	14.0	12.0	12.0
	WB-100	74.0	52.0	32.5	19.0	12.0	12.0
	WB-105	95.0	75.0	55.0	39.0	18.0	12.0
120	WB-50	59.0	40.0	23.0	14.5	12.0	12.0
	WB-55	80.0	51.0	35.0	21.0	13.5	12.0
	WB-70	72.0	52.0	34.0	17.0	12.0	12.0
	WB-100	84.5	63.0	47.0	29.0	13.0	12.0
	WB-105	106.0	85.0	68.5	49.0	21.0	12.0

Note: Boxed-in areas are conditions with enough room for an island of at least 100 square feet in size, i.e., they may require channelization.

skewed intersection angle leaves an open pavement area that, when combined with curb radii of 75 to 200 feet and a fairly narrow swept width, results in a good-size island area to channelize the right turns. At the 60-degree and 75-degree turns, the geometry is such that few of the combinations warrant channelization.

Table 5, similar to table IX-4 in the "Green Book," contains minimum designs and channelization guidelines for turning roadways. The parameters that govern the design are angle of turn, design vehicle, curb radius, width of lane, and approximate island size. For each design vehicle, table 5 lists a suggested island size and width of turning lane at each angle of turn that might need channelization, that is, those conditions that were boxed-in in tables 3 and 4. As the curb return radius increases towards 200 feet, the area of the island becomes larger and the width of the turning lane decreases. The size of islands for the larger turning angles indicates the otherwise unused and uncontrolled areas of pavement that were eliminated by the use of islands. Turning roadways for flat-angle turns, less than 75 degrees, involve relatively large radii and require designs to fit site controls and traffic conditions.

Because the truck configurations spiral into a curve, it would be desirable to fit the edge of the pavement closely to the minimum path of the design vehicle by using three-centered compound curves or simple curves with tapers to minimize the amount of unused pavement. The unnecessarily wide turn-lane widths in table 5 (see figure 3) are an indication that simple radius curves are not well suited to the turning paths of large trucks.

CONCLUSIONS

The selection of a design vehicle is a critical decision in intersection design. It is generally based on the largest standard or typical vehicle type that would regularly use the intersection. Where reliable vehicle classification counts are available, they can be used to select a design vehicle. More often, selection is based on the area type and functional classification of the intersecting highways (*12*).

The adoption of the Truck Offtracking Model that was developed by Caltrans for use in this study was advantageous for studying truck turning characteristics because it was capable of simulating various truck paths in a relatively short time period as compared to other methods. It is a powerful program once the user is familiar with all of the items which may be varied. The procedure used herein could be used for intersection design where there are high volumes of large or unique trucks. The results are of particular value at truck terminals, major ramp terminal intersections, and commercial and industrial developments.

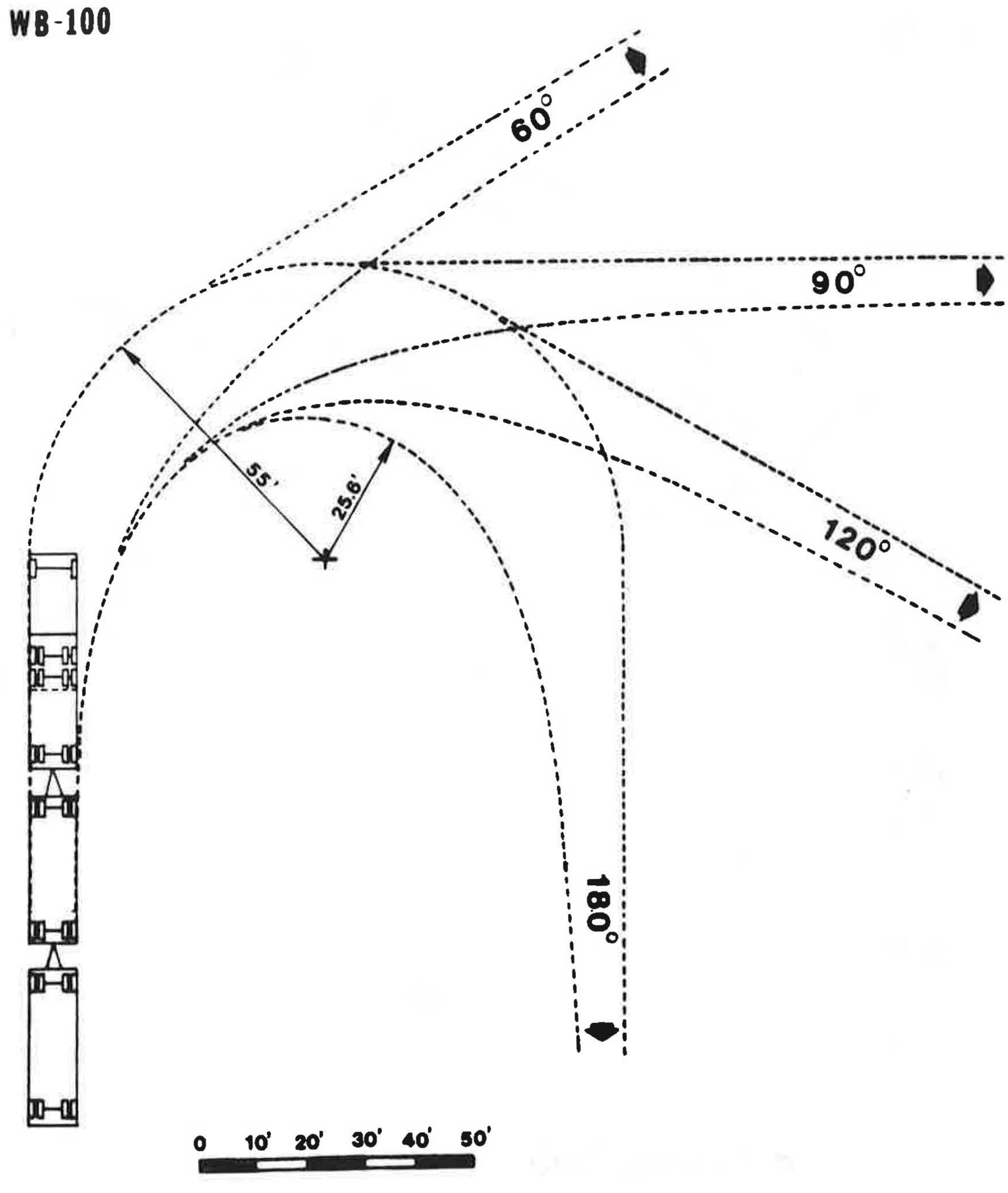


FIGURE 8 Minimum turning paths for WB-100 design vehicle.

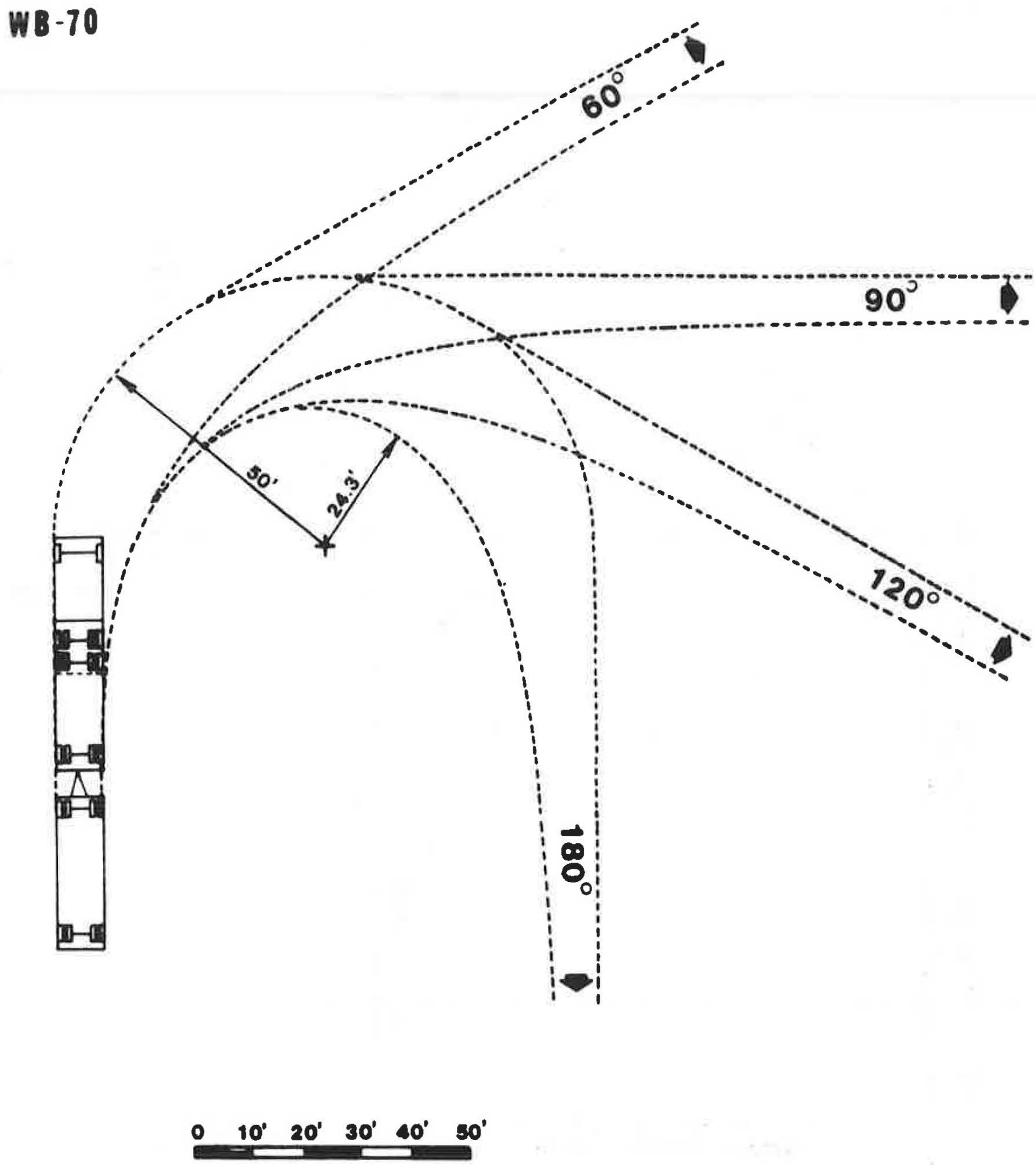


FIGURE 7 Minimum turning paths for WB-70 design vehicle.

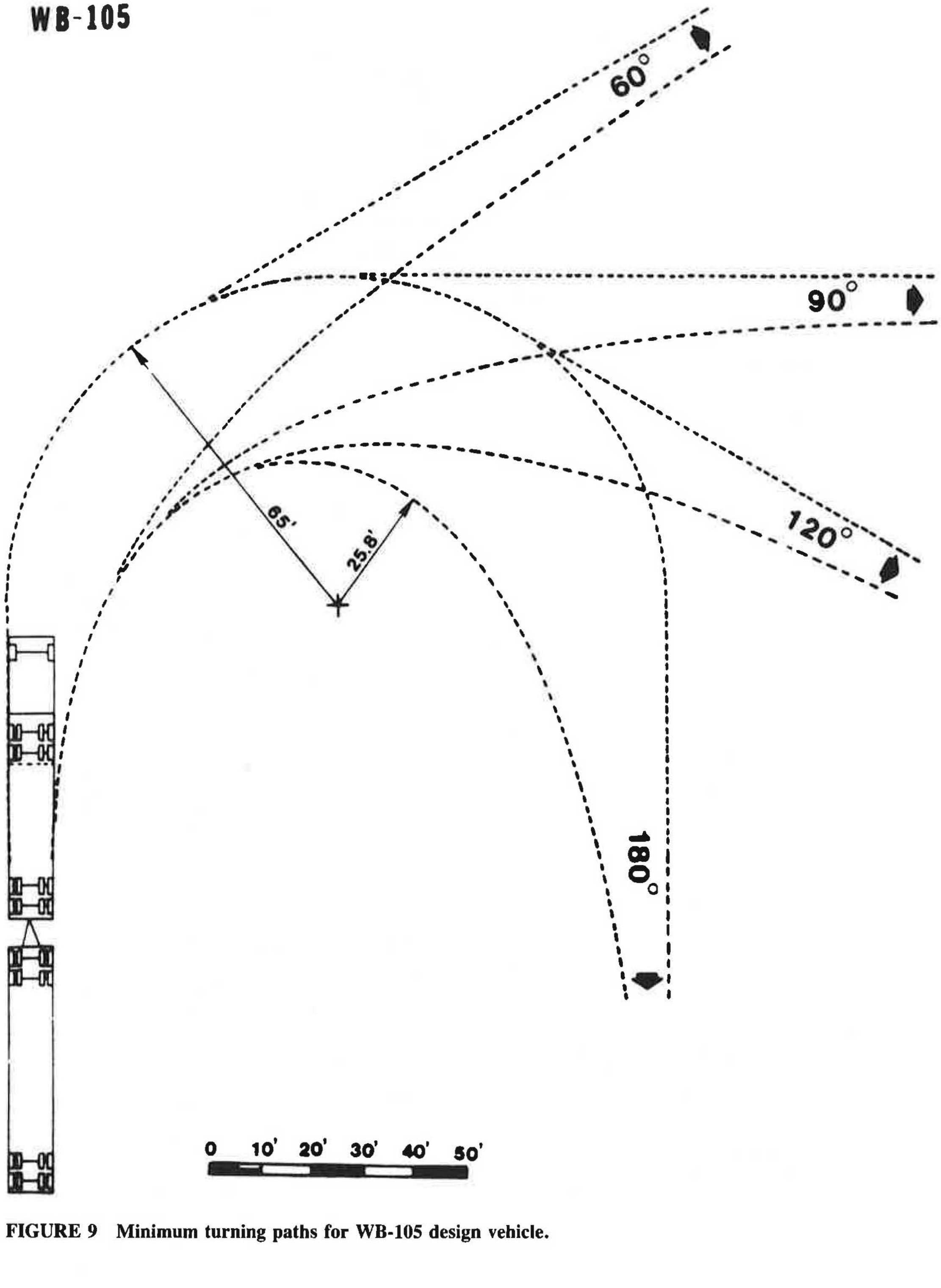


FIGURE 9 Minimum turning paths for WB-105 design vehicle.

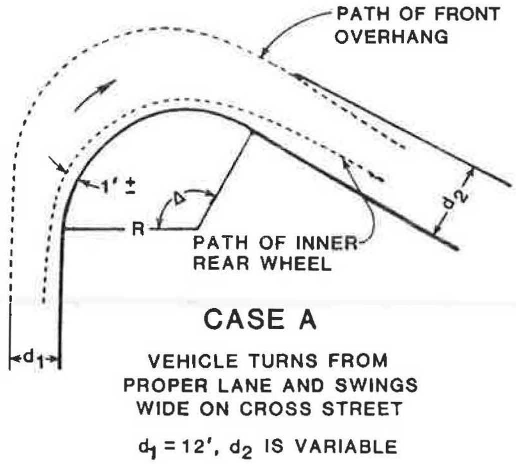


FIGURE 10 Cross street width occupied by turning vehicle.

TABLE 4 SWEPT-PATH WIDTH OCCUPIED BY TURNING VEHICLE FOR VARIOUS INTERSECTION ANGLES AND CURB RADII

Angle of Turn (Degrees)	Design Vehicle	Curb Radius 25 ft.	50 ft.	75 ft.	100 ft.	150 ft.	200 ft.
60	WB-50	18.0	18.0	18.0	18.0	16.5	15.0
	WB-55	20.5	20.5	20.5	20.5	18.8	17.0
	WB-70	19.0	19.0	19.0	19.0	16.5	15.5
	WB-100	20.7	20.7	20.7	20.7	20.0	17.8
	WB-105	24.9	24.9	24.9	24.9	24.9	24.9
75	WB-50	19.5	19.5	19.5	19.5	16.0	15.0
	WB-55	22.5	22.5	22.5	22.5	20.0	18.0
	WB-70	20.0	20.0	20.0	20.5	17.5	16.5
	WB-100	22.5	22.5	22.5	22.5	21.8	17.5
	WB-105	27.7	27.7	27.7	27.7	27.7	27.7
90	WB-50	20.5	20.5	20.5	20.5	16.5	15.0
	WB-55	24.0	24.0	24.0	24.0	22.2	18.5
	WB-70	21.5	21.5	21.5	21.5	18.0	16.5
	WB-100	24.2	24.2	24.2	24.2	23.1	19.0
	WB-105	30.0	30.0	30.0	30.0	30.0	30.0
105	WB-50	21.3	21.3	21.3	21.3	16.5	15.0
	WB-55	25.5	25.5	25.5	25.5	23.3	19.0
	WB-70	22.5	22.5	22.5	22.5	18.5	16.5
	WB-100	25.5	25.5	25.5	25.5	24.4	19.5
	WB-105	32.1	32.1	32.1	32.1	32.1	32.1
120	WB-50	22.0	22.0	22.0	22.0	19.0	15.0
	WB-55	26.8	26.8	26.8	26.8	26.8	22.0
	WB-70	23.5	23.5	23.5	23.5	21.0	18.5
	WB-100	26.5	26.5	26.5	26.5	26.5	22.0
	WB-105	33.8	33.8	33.8	33.8	33.8	22.0

Note: Boxed-in areas are conditions with enough room for an island of at least 100 square feet in size, i.e., they may require channelization.

ACKNOWLEDGMENTS

The research reported herein was performed as a part of a study entitled "Longer and Wider Trucks on the Texas Highway System" by the Texas Transportation Institute and sponsored by the Texas State Department of Highways and Public Transportation in cooperation with the U.S. Department of Transportation, Federal Highway Administration. The contents of this paper reflect the views of the authors, who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration or the State Department of Highways and Public Transportation.

TABLE 5 MINIMUM DESIGNS AND CHANNELIZATION GUIDELINES FOR TURNING ROADWAYS

Angle of Turn (degrees)	Design Vehicle	Curb Radius (ft.)	Width of Turning Lane (ft.)	Approximate Island Size (sq. ft.)
60	WB-50	200	27	250
	WB-55	200	22	160
	WB-70	200	22	160
	WB-100	200	27	160
	WB-105	-	-	-
75	WB-50	150	28	320
	WB-55	150	30	160
	WB-70	150	23	200
	WB-100	200	34	300
	WB-105	-	-	-
90	WB-50	150	30	670
	WB-55	200	38	900
	WB-70	150	22	560
	WB-100	200	40	900
	WB-105	200	54	260
105	WB-50	150	32	980
	WB-55	150	41	740
	WB-70	150	31	1320
	WB-100	200	41	1940
	WB-105	200	57	940
120	WB-50	150	40	1640
	WB-55	200	45	3400
	WB-70	150	39	1600
	WB-100	200	48	2580
	WB-105	200	60	1740

REFERENCES

1. J. M. Mason, Jr., L. Griffin, N. Straub, C. J. Molina, Jr., and D. B. Fambro. *Annotated Bibliography of Research on Operational Characteristics and Geometric Implications of Longer and Wider Trucks*. Texas Transportation Institute Research Report 397–1, February 1986.
2. C. J. Molina, Jr., C. J. Messer, and D. B. Fambro. *Passenger Car Equivalencies for Large Trucks at Signalized Intersections*. Texas Transportation Institute Research Report 397–2, November 1986.
3. T. Chira-Chavala. *An Analysis of Truck Accident Involvement and Truck Accident Severity on the Texas Highway System*. Texas Transportation Institute Research Report 397–4, November 1986.
4. D. Burke. *Larger Trucks on Texas Highways*. Texas Transportation Institute Research Report 397–5F, November 1986.
5. D. S. Millar and C. M. Walton. Offtracking of the Larger, Longer Combination Commercial Vehicles. In *Transportation Research Record 1026*, TRB, National Research Council, Washington, D.C., 1985, pp. 62–65.
6. *NCHRP Report 141: Changes in Legal Vehicle Weights and Dimensions*. TRB, National Research Council, Washington, D.C., 1973.
7. *A Policy On the Geometric Design of Highways and Streets—1984*. American Association of State Highway and Transportation Officials, Washington, D.C., 1984.
8. K. T. Fong and D. C. Chenu. Simulation of Truck Turns With a Computer Model. In *Transportation Research Record 1100*, TRB, National Research Council, Washington, D.C., 1986, pp. 20–28.
9. *Manual on Uniform Traffic Control Devices for Streets and Highways*. Federal Highway Administration, Washington, D.C., 1978.
10. *Special Report No. 5: Channelization—The Design of Intersections at Grade*. HRB, National Research Council, Washington, D.C., 1952.
11. *Special Report No. 74: Channelization—The Design of Highway Intersections at Grade*. HRB, National Research Council, Washington, D.C. 1962.
12. T. R. Neuman. Intersection Channelization Design Guide. *National Cooperative Highway Research Program Report 279*, TRB, National Research Council, Washington, D.C., November 1985.
13. Truck Offtracking Model (TOM), *Program Documentation and User's Guide*. Draft Report, Division of Transportation Planning, California Department of Transportation, Sacramento, 1985.
14. *A Policy on Geometric Design of Highways and Streets—1965*. American Association of State Highway Officials, Washington, D.C., 1965.
15. J. E. Leisch and Associates. *Turning Vehicle Templates: A Transportation Design Aid*. Transportation Design Techniques, Inc., Evanston, Illinois, 1977.
16. *Highway Design Division Operations and Procedures Manual*. Texas Department of Highways and Public Transportation, Austin, Texas, 1986.

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

Effects of Turns by Larger Trucks at Urban Intersections

JOSEPH E. HUMMER, CHARLES V. ZEGEER, AND FRED R. HANSCOM

This paper gives results and conclusions from part of a study done for the Federal Highway Administration on the safety and operational effects of large truck operations. Computer simulation and manual observations at six intersections in California and New Jersey were used to investigate turns by large trucks at urban intersections. The encroachment of a truck into adjacent lanes during a turn was studied using the computer simulation. The field data examined on a particular truck turn included the encroachment, the time to complete the turn, and the conflicts with other vehicles in the traffic stream caused by the truck. Field observations were made of turning trucks in the traffic stream and also of a control truck of known size driven repeatedly through a study intersection by a professional driver who knew the purpose of the experiment. The results showed that small curb radii, narrow lane widths, and narrow total street widths were among the geometric features associated with increased operational problems. The results also showed that large trucks will have little impact (compared with smaller trucks) at most urban intersections of the types tested, but some adverse operational effects should be expected at some intersections. Trailer length was found to be a more critical element to smooth operations than trailer width for the trucks tested. Many site, driver, and equipment factors should be considered before the decision is made to regulate truck traffic in a certain manner.

The Surface Transportation Assistance Act of 1982 (1982 STAA) required some states to change their restrictions on the sizes of trucks operating on their portions of the national truck network of interstate and other designated Federal-aid highways. Due to the 1982 STAA, states may not impose trailer width limits of less than 102 inches. A 96-inch maximum trailer width had been in effect in most states prior to the 1982 STAA. The 1982 STAA also provided that states allow tractor-semitrailer combinations with semitrailer lengths of up to 48 feet and tractor-semitrailer-trailer combinations with semitrailer and trailer lengths of up to 28 feet on the national network. Previously, states had the freedom to impose maximum semitrailer lengths and in some cases had prohibited tractor-semitrailer-trailer combinations.

The interstate and turnpike systems have generally been built to very high geometric standards. However, other Federal-aid systems often contain lower standard design features, which may impact safety and necessitate limiting operations of the large trucks specified in the 1982 STAA. It was, therefore, timely to evaluate the impacts of large truck operation on roads and streets with restrictive geometry and to provide insights relative to the selection of routes for the national network.

This paper gives results and conclusions from part of a study done for the Federal Highway Administration (FHWA) on the safety and operational effects of large truck operations (*1*). Two particular situations were identified for study: truck negotiation of winding rural roads and truck turns at urban intersections. This paper details only the urban intersection portion of the study.

A review of previous research revealed that some operational problems are expected when large trucks make turns at urban intersections, but many questions on the issue remain unanswered. An analysis in Texas of the impact of different truck sizes on a variety of geometric conditions concluded that increases in allowed truck size may warrant highway design standard changes (*2*). A 1982 study conducted in Ontario, Canada, showed that large trucks, including tractor-semi-trailer-trailer combinations, offtrack (swing wide during turns) farther than smaller trucks (*3*). A study by the Western Highway Institute showed that longer combinations required extra lanes or overlapped into adjacent lanes to negotiate right-angle turns at intersections (*4*). Tractor-semitrailer-trailer combinations were observed in California to use extra lanes, traverse curbs and channels, and use excessive time during turns at intersection (*5*). A 1981 field study that attempted to correlate increase in truck size with operational problems at several sites, including intersections, concluded that increased truck lengths were associated with only negligible operational traffic effects, however (*6*).

Two investigation methods were employed during the study of larger truck turns at urban intersections. A computer simulation technique was used to analyze the offtracking of different sizes of trucks during different turning maneuvers. The simulation provided information that may be useful in selecting routes for the national network. In addition, turning trucks were observed at actual intersections during the study. The field observations allowed comparisons between different truck sizes for particular intersections, which may be useful in predicting the impact of large truck operations.

COMPUTER SIMULATION OF TRUCK TURNS

The turning characteristics of larger trucks were investigated using the Vehicle Offtracking Model and Computer Simulation developed by FHWA and the University of Michigan Transportation Research Institute. The software package

J. E. Hummer, School of Civil Engineering, Purdue University, West Lafayette, Ind. 47907. C. V. Zegeer, Highway Safety Research Center, University of North Carolina, Chapel Hill, N.C. 27514. F. R. Hanscom, Transportation Research Corporation, Haymarket, Va. 22069.

allowed plotting of the positions of the outside edges of the tractor and trailer(s) and the positions of the tires as a given type of truck completed a turn at an intersection with a given configuration. From such a plot, a more useful plot of the area covered by the truck during the turn was made and analyzed.

The simulation was run for eight types of larger trucks: a tractor-semitrailer combination with a 48-foot semitrailer that was 96 inches wide (semi 48), a semi 48 that was 102 inches wide (semi 48 wide), a tractor-semitrailer combination with a 55-foot semitrailer that was 96 inches wide (semi 55), a semi 55 which was 102 inches wide (semi 55 wide), a tractor-semi-trailer-trailer combination with 28-foot trailers that were 96 inches wide (double 28) a double 28 that was 102 inches wide (double 28 wide), a tractor-semitrailer-trailer-trailer combination with 28-foot trailers that were 96 inches wide (triple 28), and a triple 28 that was 102 inches wide (triple 28 wide). Each of the eight truck types was run over many combinations of angle and radius of turn which are representative of intersections in the United States.

Two measures were used to analyze the offtracking plots produced from the simulation runs. The maximum offtracking distance was recorded for each plot. This distance represents the widest swing of a truck during a turn, as shown in figure 1. The other measure employed was the lane encroachment of the truck during the turn. The lane encroachment was defined as the distance between the curb and the farthest edge of the truck's path measured at the end of the curvature of the curb (at the stop bar of the street onto which the truck is turning). The lane encroachment was measured from the simulation plot by aligning the point of maximum offtracking and the center of the curve of the curb as shown in figure 2. The significance of the lane encroachment is seen, for example, by a truck turning onto a four-lane street with 12-foot wide lanes. If the lane encroachment of the truck for the angle and radius of turn at the intersection is greater than 24 feet, the truck cannot complete the turn without crossing the centerline of the street onto which it is turning. If there is a vehicle idling next to the centerline at the stop bar of that street, the truck cannot complete the turn.

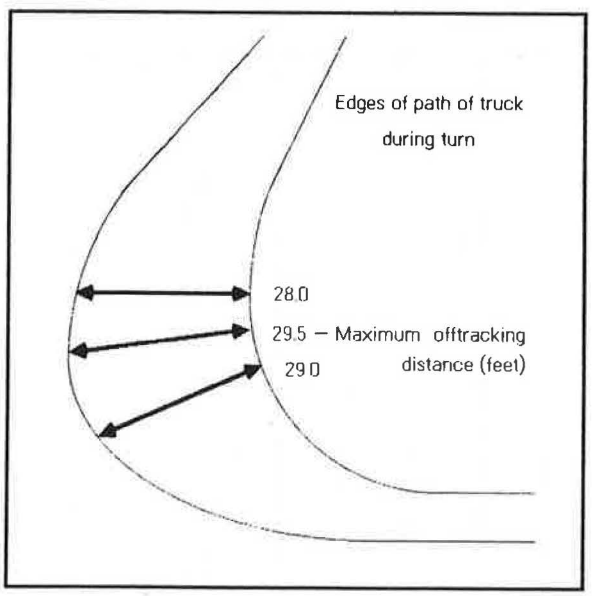


FIGURE 1 Illustration of measurement of maximum offtracking distance.

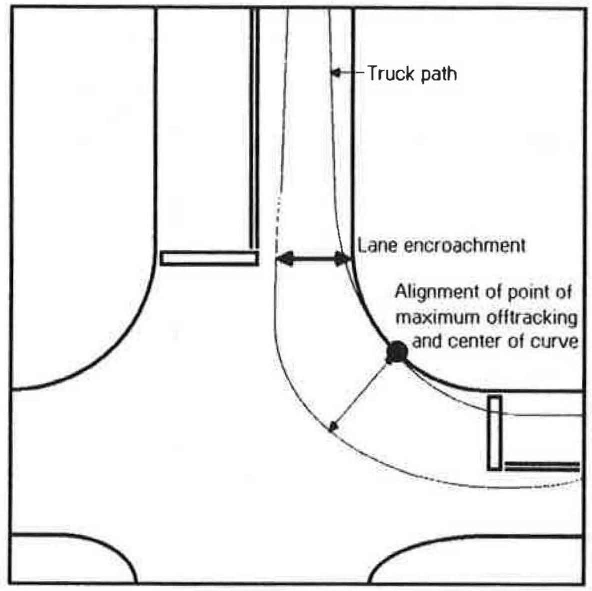


FIGURE 2 Illustration of measurement of lane encroachment.

A summary of the simulation results is given in tables 1 and 2 for the maximum offtracking distance and the lane encroachment measures, respectively. From tables 1 and 2, it can be seen that the most serious operational problems at intersections can be expected (of the eight large truck types examined) from the semi 55 wide. Other truck types, in descending order of space required, are the semi 55, the semi 48 wide, and semi 48, the triple 28 wide, the triple 28, the double 28 wide, and the double 28.

Tables 1 and 2 also show that, for a given trailer configuration and length, 102-inch wide trucks generally exhibited greater maximum offtracking distances and greater lane encroachments than 96-inch wide trucks, but the difference was usually only 0.5 or 1.0 feet. Thus, for the field observations reported later, the issue of the width of the turning trucks was ignored, and the effort was directed at examining the effects of different trailer lengths and configurations.

Table 2 also provides guidance for the selection of truck routes. In general, lane encroachment magnitudes were large (that is, the truck would cross the centerline on a four-lane street) for most larger angles of turn for radii of 22 and 40 feet. Greater intersection angles did not necessarily mean greater lane encroachment values for most truck types, however. Only the semi 55 and semi 55 wide displayed generally greater lane encroachments with greater intersection angles.

The results shown in tables 1 and 2 should be used judiciously, since the simulation was limited in a number of ways. Differences between individual truck drivers may be great enough to overcome the effects of different-size vehicles, but such variability was not included in the simulation. The reactions of the drivers of other vehicles in the traffic stream and the volume of such other vehicles were also omitted from the simulation. Finally, the speed of a turn was not an output of the simulation. This is a serious limitation since a truck driver who slows a great deal in order to complete a turn without encroaching on the centerline or curb may cause as great a

TABLE 1 MAXIMUM OFFTRACKING DISTANCES USING SIMULATION

Truck type	Maximum offtracking distances, in feet, for given angles of intersection in degrees and curb radii in feet														
	angle=60			angle=70			angle=90			angle=105			angle=120		
	R=20	R=40	R=60	R=20	R=40	R=60	R=20	R=40	R=60	R=20	R=40	R=60	R=20	R=40	R=60
Semi 48	23.5	21.0	*	*	*	*	31.0	25.5	22.0	35.0	28.0	*	39.0	29.0	*
Semi 48 Wide	24.0	22.0	*	*	*	*	31.0	26.0	22.5	35.0	28.0	*	39.5	29.5	*
Semi 55	*	23.0	21.0	28.0	25.0	22.5	33.5	28.5	*	38.0	31.0	*	43.0	33.5	*
Semi 55 Wide	*	23.5	22.0	29.0	26.0	23.0	34.0	29.0	25.5	38.5	31.5	*	43.0	34.0	27.5
Double 28	20.0	17.5	16.0	21.5	18.5	16.5	25.0	20.0	*	28.0	21.0	17.0	30.0	22.0	17.5
Double 28 Wide	*	18.0	16.0	22.0	18.5	16.5	25.5	21.0	18.0	28.0	21.5	*	30.5	22.5	18.0
Triple 28	*	20.5	18.0	25.0	22.0	19.0	30.0	25.0	*	33.0	26.0	*	37.0	28.0	*
Triple 28 Wide	*	21.0	19.0	26.0	22.5	20.0	31.0	25.5	21.5	34.0	27.0	*	39.0	28.0	*

* - No data were recorded.

TABLE 2 LANE ENCROACHMENT DISTANCES USING SIMULATION

Truck type	Lane encroachment distances, in feet, for given angles of intersection in degrees and curb radii in feet														
	angle=60			angle=70			angle=90			angle=105			angle=120		
	R=20	R=40	R=60	R=20	R=40	R=60	R=20	R=40	R=60	R=20	R=40	R=60	R=20	R=40	R=60
Semi 48	22.0	21.0	*	*	*	*	27.0	22.0	19.0	27.0	22.5	*	28.0	21.5	*
Semi 48 Wide	23.5	21.5	*	*	*	*	26.0	22.0	20.0	27.5	22.5	*	27.5	22.0	*
Semi 55	*	22.5	19.0	27.0	24.0	20.5	30.0	26.0	*	32.0	27.0	*	33.5	26.0	*
Semi 55 Wide	*	22.5	19.0	27.5	25.0	21.0	29.5	26.0	23.0	31.0	26.0	*	33.0	25.0	21.0
Double 28	18.5	16.5	14.0	20.0	17.0	15.0	20.0	17.0	*	21.5	17.0	*	20.5	15.5	12.5
Double 28 Wide	*	17.0	15.0	20.5	17.0	15.0	21.0	17.0	15.0	22.0	18.0	*	21.5	17.0	14.0
Triple 28	*	18.5	16.5	22.5	20.0	16.5	25.0	20.5	*	26.5	21.5	*	24.5	19.5	*
Triple 28 Wide	*	19.0	17.0	24.0	20.5	16.5	25.0	21.0	18.0	27.0	22.0	*	24.5	20.0	*

* - No data were recorded.

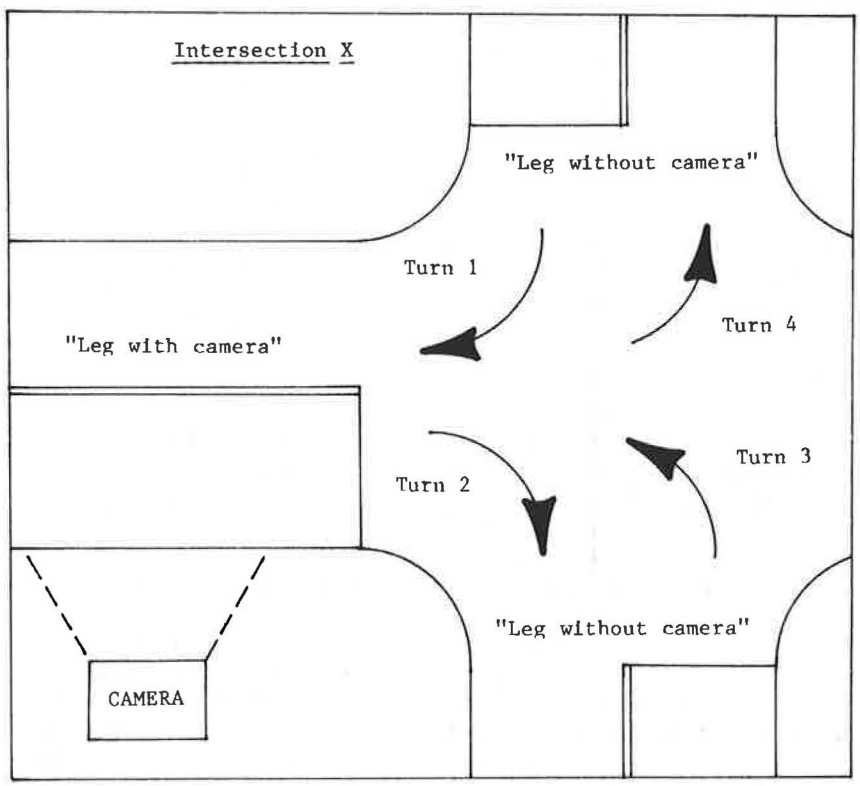
traffic operation or safety problem as a truck driver who does encroach on the centerline or curb.

FIELD OBSERVATION METHODOLOGY

The discussion of the field observation methodology and data in the following sections is given in terms of a specific turn at an intersection, or site. Each site was assigned a two-digit number for identification as shown in figure 3. The first digit of the site number represents the intersection number (for example, 1 through 6, since data were collected at six intersections). The second digit of the site number represents the specific turn at the intersection. A "1" in the second digit represents a right turn onto the leg of the intersection where the data collection camera was stationed, a "2" represents a right turn from the leg with the camera, a "3" represents a left turn onto the leg with the camera, and a "4" represents a left turn from the leg with the camera.

Intersections for the field observation of turning trucks were chosen on the basis of a number of criteria. It was desired that the study should include at least two states in different regions of the United States, and New Jersey and California were chosen. Those states had relatively large samples of large trucks operating on non-freeway routes, and officials in those states were willing to cooperate with the study. Intersections within those states were sought that had large volumes of turning truck traffic as well as certain geometric and traffic features, such as available observer positions, no channels or median barriers, no protected signal phases, no recessed stop bars, 90-degree turns, and minimal pedestrian volumes. It was desired that other geometric features such as lane widths, numbers of lanes, and curb radii vary between the observed intersections. Six intersections that were considered to best fit these criteria were selected. Some of the geometric features that varied among intersections are shown in table 3. The intersections range from a major intersection of seven-lane and five-lane arteries to a three-legged unsignalized intersection between four-lane and two-lane collector streets.

Both left and right turns by trucks were observed in the field. The measures of effectiveness (MOEs) examined during observations of truck turns included encroachments into adja-



"Site X1" refers to intersection X, turn 1.

"Site X2" refers to intersection X, turn 2.

"Site X3" refers to intersection X, turn 3.

"Site X4" refers to intersection X, turn 4.

FIGURE 3 Field observation site numbering system.

TABLE 3 FIELD OBSERVATION SITE CHARACTERISTICS

Characteristic \ Intersection	1	2	3	4	5	6
State	NJ	NJ	Calf.	Calif.	Calif.	Calif.
Number of legs	4	4	4	4	3	3
Number of lanes, leg with camera	3	2	7	5	2	4
Number of lanes, legs without camera	4	4	5	4	4	4
Avg. lane width (ft.), leg with camera	16	11	12	10.5	12	10
Avg. lane width (ft.), legs without camera	12	10.5	12	10	10	11
Width of lane from which turn 1 was made (ft.)	No Turn	11	13	11	10	12
Width of lane from which turn 2 was made (ft.)	17	11	14	13	12	10
Avg. curb radius (ft.), turn 1	No Turn	45	55	32	12	40
Avg. curb radius (ft.), turn 2	21	55	55	35	12	32
Signalized?	Yes	Yes	Yes	Yes	No	Yes
Protected turn phases?	For turn 3 only	No	No	No	Not appli-cable	No

cent lanes, over the centerline, and over the curb; traffic conflict events such as weaving, stopping, and backing by vehicles into the traffic stream and by the truck; and the clearance time of the truck through the intersection. The clearance time was defined for trucks making right turns and trucks making rolling left turns (in other words, no impeding traffic forced the truck to stop beyond the stop line) as the difference between the time the front tires of the truck crossed the stop bar of the origin street into the intersection and the time the rear tires of the truck crossed the stop bar of the destination street. For left-turning trucks delayed by impeding traffic when they were beyond the stop bar of the origin street, the clearance time was defined as the difference between the time the truck began rolling forward and the time the rear tires crossed the stop bar on the destination street. Since there were very few rolling left turns completed by the trucks at most sites, the analyses were not biased by the use of the two definitions.

The hypotheses tested using the field observations were that larger trucks did not degrade operations at particular turns as measured by the MOEs in comparison to pre-STAA trucks. Larger trucks of interest were the semi 48 and the double 28. The semi 55, triple 28 and other larger trucks were not in common use at the times and locations of testing so adequate samples were not available for observation. Pre-STAA trucks of interest were the tractor-semitrailer combinations with semitrailer lengths of 40 feet (semi 40) and 45 feet (semi 45).

Manual observation was used to collect MOE data on turning trucks. A team of three observers stationed on different corners of the intersection examined turning trucks selected for study, with each observer recording only those MOEs for which he/she had the best view (each observer looked for different MOEs, depending on the turn the truck was making). A fourth observer recorded clearance time, using a stopwatch. A fifth observer photographed each truck selected for study. The slides of the photographs, taken from a known distance at ground level, were later projected onto a screen, scaled off, and used to obtain the truck dimensions. Other clues, such as the number of 4-foot wide panels on the side of the trailer and the trailer size printed on the side of the trailer, were used to corroborate the scaled estimates of the truck dimensions.

Up to four different turns were observed at each intersection—each turn originating from or destined for the leg of the intersection on which the camera was stationed. Trucks approaching the intersection apparently ready to make one of the four turns were assigned an identification number and communication between the observers via walkie-talkie ensured that all observers were viewing the same truck. Observations were made only during daylight hours with dry pavement conditions.

The manual data collection method proved sensitive and accurate. Pretests with several people recording conflict and encroachment data at one observer position simultaneously and independently showed a high degree of correlation among observers. The photographic method of estimating truck size, when checked with trucks of known dimensions, proved sufficiently accurate to obtain trailer lengths within one foot of the actual length.

During the test period at the two New Jersey intersections (intersection numbers 1 and 2), control trucks were used to ensure adequate samples of certain types of trucks. These control trucks (a semi 40, a semi 48, and a double 28) were driven through the intersections repeatedly by a professional driver who knew the purpose of the testing.

FIELD OBSERVATION DATA

Data were collected on a total of 1,151 turning trucks, as shown in table 4. The sample included 412 semi 40s (108 control trucks and 304 trucks in the traffic stream), 443 semi 45s (all traffic stream), 177 semi 48s (90 control and 87 traffic stream), and 119 double 28s (61 control and 58 traffic stream). The samples per intersection ranged from 132 trucks at intersection 3 to 308 at intersection 1. Small samples of semi 48s and double 28s were collected at some intersections. It is not assumed that the sample of turning trucks observed is representative of the states of California and New Jersey or of the United States. Summary data from the field tests are given in tables 5, 6, and 7 for turn time, the proportion of trucks committing at least one encroachment, and the proportion of trucks causing at least one vehicle conflict, respectively.

COMPARISONS AMONG SITES

During the analysis of the field observation data, comparisons were made among sites to see where the most operational problems from large trucks can be expected and to see whether the sites were similar enough to warrant pooling the data. Pooling the data for different sites would allow larger sample sizes of semi 48 and double 28 observations to be formed which would allow more powerful testing among truck types.

Turn times for the traffic stream semi 45 (for which observations were plentiful at most sites) were compared for each pair of right turns at signalized intersections using the *t*-test. The tests revealed that the right turns from the leg with the camera at intersections 1 and 3 (sites 12 and 32) had significantly faster turn times (at the 0.05 level) than several other sites. These differences were not surprising, since table 3 shows that those sites had a relatively wide turn lane and a relatively long curb radius, respectively. Thus, the data from the remaining signalized right turn sites were pooled for comparisons of turn times between different truck types. In a similar series of *t*-tests using semi 45 turn times on signalized left turn sites, the left turn to the leg with the camera at intersection 1 and both left turns at intersection 4 (sites 13, 43, and 44, respectively)

TABLE 4 SAMPLE SIZES OF TRUCK TYPES AT INTERSECTIONS

Truck type	Number of trucks observed at intersection						
	1	2	3	4	5	6	All intersections
Control - Semi 40	48	60	0	0	0	0	108
Control - Semi 48	60	30	0	0	0	0	90
Control - Double 28	29	32	0	0	0	0	61
Traffic - Semi 40	63	30	44	42	67	58	304
Traffic - Semi 45	94	67	42	65	121	54	443
Traffic - Semi 48	14	9	17	21	6	20	87
Traffic - Double 28	0	0	29	17	2	10	58
All truck types	308	228	132	145	196	142	1151

TABLE 5 FIELD OBSERVATIONS OF TURN TIME

Truck type	Inter-section number	Mean turn time (seconds) with sample size in parentheses			
		Right Turn		Left Turn	
		Turn 1	Turn 2	Turn 3	Turn 4
Control - Semi 40	1	(0)	7.56 (24)	7.21 (24)	(0)
	2	(0)	6.52 (30)	7.80 (30)	(0)
Control - Semi 48	1	(0)	7.98 (31)	9.15 (27)	(0)
	2	(0)	8.41 (15)	8.23 (14)	(0)
Control - Double 28	1	(0)	8.58 (16)	7.95 (16)	(0)
	2	(0)	8.22 (16)	9.16 (16)	(0)
Traffic - Semi 40	1	(0)	7.93 (26)	7.48 (16)	8.16 (21)
	2	7.76 (15)	8.35 (5)	6.85 (6)	10.95 (4)
	3	8.42 (4)	6.27 (14)	8.51 (13)	7.70 (13)
	4	12.63 (16)	11.88 (6)	11.70 (10)	10.32 (10)
	5	8.87 (5)	8.82 (23)	9.36 (36)	9.54 (2)
	6	8.22 (24)	7.79 (1)	8.85 (11)	9.80 (22)
Traffic - Semi 45	1	(0)	8.75 (42)	7.91 (34)	8.76 (18)
	2	10.17 (19)	8.66 (13)	9.06 (23)	10.31 (12)
	3	7.79 (9)	7.38 (15)	9.73 (12)	7.62 (6)
	4	10.40 (22)	11.96 (10)	10.51 (18)	10.76 (15)
	5	10.40 (6)	10.30 (49)	8.76 (63)	9.60 (3)
	6	8.62 (20)	6.73 (1)	9.39 (5)	9.13 (28)
Traffic - Semi 48	1	(0)	8.14 (8)	7.65 (5)	11.18 (1)
	2	6.77 (4)	7.87 (2)	8.93 (1)	13.19 (2)
	3	9.36 (3)	7.14 (4)	7.44 (4)	7.07 (6)
	4	12.42 (8)	15.71 (1)	9.03 (6)	11.72 (6)
	5	(0)	9.67 (4)	7.68 (2)	(0)
	6	7.31 (4)	5.95 (1)	9.86 (1)	9.46 (14)
Traffic - Double 28	3	6.66 (4)	6.34 (1)	9.66 (12)	11.62 (12)
	4	9.22 (8)	8.45 (1)	(0)	12.09 (8)
	5	(0)	(0)	9.24 (2)	(0)
	6	6.67 (3)	17.69 (1)	(0)	9.18 (6)

exhibited significantly different turn times (at the 0.05 level) than other sites. Site 13 had lower turn times, probably due to the protected turn signal phase for that turn. Sites 43 and 44 had higher turn times, due perhaps to the combination of narrow turn lanes and narrow destination streets. Data from the remaining signalized left turn sites were pooled in comparisons between truck types using turn times.

The proportion of semi 40s, semi 45s, and double 28s that committed at least one encroachment was compared for each pair of sites using the Kruskal-Wallis One-Way Analysis of Variance test. Significant differences were found to exist (at the 0.05 level) between each site and at least three other sites. Individual site characteristics apparently play a large role in the incidence of encroachments by turning trucks. A similar statistical analysis using vehicle conflict MOEs was not possible due to small numbers of conflicts at most sites, but inspection of the data does suggest variations in rates of conflict between sites. Thus, the conflict and encroachment data from different sites were not pooled.

A combination of several site characteristics appear to affect the encroachment rates, including lane widths, curb radii, stop bar location, and the number of lanes. Encroachment rates were relatively high at the right turn onto the leg with the camera at intersections 4 and 6 (sites 41 and 61, respectively) which has narrower turn lanes and narrower widths on the destination street than some other sites. Conversely, there was a relatively low proportion of encroachments at the right turn onto the leg with the camera at intersection 3 (site 31) where there was a wide turn lane and a long curb radius. Encroachment rates were relatively high at the left turn onto the leg with the camera at intersections 1 and 5 (sites 13 and 53, respectively), with only one lane on the target streets and stop bars set close to the intersection, and at the left turn from the leg with the camera at intersection 4 (site 44) with

TABLE 6 ENCROACHMENT DATA FROM FIELD OBSERVATIONS

Site number	Number of trucks with one or more encroachments/Observed total of trucks						
	Truck type						
	Control Semi 40	Control Semi 48	Control Dbl. 28	Traffic Semi 40	Traffic Semi 45	Traffic Semi 48	Traffic Dbl. 28
12	7/24	31/31	5/14	19/26	29/42	7/8	
13	3/24	25/29	5/15	14/16	28/34	5/5	
14				7/21	7/17	0/1	
21				13/15	19/19	4/4	
22	0/30	15/15	15/16	5/5	13/13	2/2	
23	0/30	11/15	2/16	1/6	8/23	0/1	
24				2/4	11/12	2/2	
31				0/4	4/9	1/3	2/4
32				6/14	8/15	4/4	0/1
33				0/13	0/12	0/4	0/12
34				1/13	4/6	0/6	3/12
41				15/15	22/22	8/8	8/8
42				6/6	10/10	1/1	1/1
43				5/10	9/18	4/6	
44				7/10	13/15	6/6	4/8
51				6/6	6/6		
52				22/23	48/49	3/4	
53				16/36	41/63	2/2	1/2
54				2/2	3/3		
61				23/24	19/20	4/4	3/3
62				1/1	1/1	1/1	1/1
63				0/11	0/5	0/1	
64				8/22	13/28	6/14	1/6
Total encroachments	10	85	27	179	316	60	24
Total number of trucks	108	90	62	303	442	87	58

a very narrow turn lane. Both left turns at intersection 3 sites (33 and 34), however, with relatively wide left turn lanes and wide destination streets, had virtually no encroachments.

COMPARISONS AMONG TRUCK TYPES

Comparisons were made among the data for control and for traffic-stream trucks of a given size at a given site, with a view toward pooling those observations. In general, *t*-tests on turn times for sites with sufficient sample sizes showed few differences between control and traffic-stream trucks. However, *Z*-tests on proportions of conflicts and encroachments for sites with sufficient samples showed many differences between control and traffic-stream trucks. This is reasonable, since the drivers of the control trucks were aware of the experiment and repeated the same turns many times. These drivers were familiar with each site and were likely to exercise special care in making turns, particularly trying to avoid encroaching curbs or centerlines. Thus, in the comparisons among different truck types, the control and traffic-stream observations for a particular truck size at a particular site were not pooled.

The turn-time data were analyzed statistically using the *t*-test to compare two truck types for a particular site or pool of sites whenever there were at least five observations for each truck type. The *t*-test results, summarized in table 8, show that there were insufficient samples of turning trucks at many sites to conduct *t*-tests. For sites with sufficient samples, the test most often supported the hypothesis that there was no difference between truck types. The hypothesis was rejected for two important cases, however. First, in comparisons between semi 40 and semi 48 control trucks at two different sites, one right turn and one left turn, the semi 40 completed turns significantly faster. In both of those comparisons, the mean time for the semi 40 turn was about seven seconds while the mean time for the semi 48 was about nine seconds. It is not

TABLE 7 VEHICLE CONFLICT DATA FROM FIELD OBSERVATIONS

Site number	Number of trucks which caused one or more vehicle conflicts/ Observed total of trucks						
	Truck type						
	Control Semi 40	Control Semi 48	Control Dbl. 28	Traffic Semi 40	Traffic Semi 45	Traffic Semi 48	Traffic Dbl. 28
12	0/24	9/31	0/14	2/26	1/42	1/8	
13	7/24	9/29	3/15	2/16	11/34	1/5	
14				3/21	0/18	0/1	
21				2/15	7/19	2/4	
22	0/30	0/15	0/16	0/5	0/13	0/2	
23	4/30	5/15	6/16	1/6	6/23	1/1	
24				0/4	0/12	0/2	
31				1/4	1/9	1/3	0/4
32				2/14	0/15	0/4	0/1
33				1/13	0/12	1/4	0/12
34				1/13	0/6	1/6	0/12
41				3/16	4/22	4/8	0/8
42				1/6	2/10	0/1	1/1
43				0/10	1/18	1/6	
44				2/10	7/15	2/6	3/8
51				0/6	3/6		
52				1/23	5/49	1/4	
53				8/36	10/63	0/2	1/2
54				0/2	1/3		
61				1/24	3/20	0/4	0/3
62				0/1	0/1	0/1	1/1
63				0/11	0/5	0/1	
64				1/22	3/28	0/14	0/6
Total conflicts	11	23	9	32	65	16	6
Total number of trucks	108	90	62	304	443	87	58

clear why two sites showed differences while at two other sites the comparison of control truck turn times for the semi 40 and semi 48 had no differences. Second, the double 28 proved significantly slower in one comparison of right turn time (a control truck comparison with the semi 40 at site 22) and in four comparisons of left turn time (a control truck comparison with the semi 40 at site 23 and traffic stream comparisons for the pooled data with the semi 40, semi 45, and semi 48). The differences in mean turn times for these comparisons were usually 1.5 to 2.5 seconds. It appears that the double 28 generally had longer turn times where the intersection characteristics were less restrictive, since site 22 had a long curb radius, site 23 had a recessed stop bar, and the pooled data were heavily influenced by data from intersection 3 with less restrictive geometry.

The data in table 6 show that there were differences in the proportions of trucks committing at least one encroachment between truck types at some sites. The differences for the control trucks are large. The semi 48 committed encroachments significantly more often (at the 0.05 level) than the semi 40 at all four sites observed and significantly more often (at the 0.05 level) than the double 28 at sites 12, 13, and 23. The control double 28 committed encroachments at a significantly greater rate (at the 0.05 level) than the semi 40 at site 22 and marginally more often (not statistically significant at the 0.05 level) at sites 13 and 23. The differences between truck types were less apparent for the traffic-stream trucks than for the control trucks, due to smaller samples of the semi 48 and double 28 or to the effects of differences among individual truck drivers who were unaware of the purposes of the observers. Statistical tests were inappropriate for most possible comparisons due to the small samples of semi 48s and double 28s.

Table 7 shows that the proportions of trucks causing a conflict did not vary much at particular intersections between truck types. For control trucks, the semi 48 caused conflicts

TABLE 8 SUMMARY OF *t*-TESTS ON TURN TIME DATA

Truck type comparison		Right turn sites							Left turn sites					
		12	22	32	41	52	61	Site* Group A	13	14	23	53	64	Site* Group B
Control trucks	Semi 40 vs. Semi 48	●	▲	▨	▨	▨	▨	▨	▲	▨	●	▨	▨	▨
	Semi 40 vs. Double 28	●	▲	▨	▨	▨	▨	▨	●	▨	▲	▨	▨	▨
	Semi 48 vs. Double 28	●	●	▨	▨	▨	▨	▨	●	▨	●	▨	▨	▨
Traffic trucks	Semi 40 vs. Semi 45	●		●	●	●	●	●	●	●		●	●	●
	Semi 40 vs. Semi 48	●			●			●	●				●	●
	Semi 40 vs. Double 28				●			●					●	▲
	Semi 45 vs. Semi 48	●			●			●	●				●	●
	Semi 45 vs. Double 28				●			●					●	▲
	Semi 48 vs. Double 28				●			●					●	▲

Note: Sites not shown had insufficient samples for t-test or no data collected for all comparisons.

* - Site Group A includes sites 21, 22, 31, 41, and 42; B includes sites 14, 23, 33, 34, 63, and 64.

□ - Insufficient sample size for t-test.

▨ - No data collected.

● - No significant (0.05 level) difference in average turn time.

▲ - Significant (0.05 level) increase in mean turn time for second truck type.

marginally more often than the semi 40 and the double 28 at site 12, and the semi 48 and double 28 caused conflicts marginally more often than the semi 40 at site 23. Among traffic-stream trucks, a marginal difference among truck types was apparent only at site 41 between the semi 48 and the other truck types. Statistical tests again were generally inappropriate due to the small samples.

Until this point in the report, the fact that many semi 48s have moveable rear axles has not been mentioned. The computer simulation was performed with the rear axles of the semi 48 and semi 55 placed as far to the rear of the semitrailer as possible, and the control truck was also set up in this way. However, for the sample of semi 48s observed in the field, there was a noticeable variety in the position of the rear axles. The photographs of the turning semi 48s were thus examined for rear axle position. Of the 87 traffic-stream semi 48s, 43 had axles placed forward (six to nine feet from the center of the rear set of wheels to the rear of the semitrailer), 36 had axles placed back (three to six feet from the center of the rear set of wheels to the rear of the semitrailer), and eight had axle placements that could not be measured from the photographs. Since the rear axle placement affects offtracking and could affect truck performance on turns in terms of the MOEs studied in the field, the data for semi 48s were examined for the effects of different axle placements. The turn times for the pooled right turns and the pooled left turns were used to compare the semi 48 with axles forward to axles back. For the right turns, the trucks with axles back had a mean time of 11.3 seconds, compared to a mean of 7.3 seconds for the trucks with axles forward. This difference was statistically significant at the 0.05 level using the *t*-test with 16 degrees of freedom. For left turns, the difference in mean turn times was negligible and statistically insignificant. Insufficient samples were available to analyze encroachments or conflicts for the axle positions.

The final step of the data analysis involved a look at the effect of the presence of a vehicle near the turning truck. There was concern that a given truck turned differently depending on whether there was a vehicle beside the truck before the turn or waiting at the stop bar in the center lane of the destination street (in other words, the truck was not free to swing wide during the turn) and that this bias was reflected in the turn time and encroachment results given previously. In addition, there was concern that analysis of the conflict data was biased against high-volume intersections, since low-volume intersections would have a greater proportion of turning trucks with no chance of conflicts (no other vehicles present to conflict with the truck). However, a duplication of the analyses described above using only the data recorded when there were other vehicles present (approximately four-fifths of all observations) showed that no important changes in the results already reported were necessary.

CONCLUSIONS

In reviewing the study results, the limitations of the study methods must be kept in mind. The simulation was limited because the differences among individual truck drivers, the reactions of the drivers of other vehicles in the traffic stream, and the speed of the turn were not modeled. The field observations were limited because they were based partially on a control truck with a professional driver knowledgeable of the purpose of the observations and because of the small samples of traffic-stream truck data gathered at some sites. The results and conclusions should not be generalized to cover truck types or types of intersections that were not specifically tested.

No blanket regulations on truck routes should be based on this study. Many site, driver, and equipment variables must be examined before the decision to regulate truck traffic in a certain manner can be made. The computer simulation and field observation results showed that different types of trucks perform differently at different intersections and that small curb radii, narrow lane widths, and narrow destination roadways were among the geometric factors associated with increased operational problems.

Semi 48s and double 28s will have little impact on traffic operations at most intersections like those tested, but limited operational problems should be expected at some intersections. The simulation demonstrated that trailer width is not as critical to smooth operations as trailer length, over the ranges of trucks and intersections simulated. Among the larger trucks simulated, the semi 55 would be expected to cause the most operational problems at a given intersection, followed by the semi 48, the triple 28, and the double 28. In field tests, the semi 48 sometimes turned slower, committed more encroachments, and caused more conflicts than the semi 40. The double 28 sometimes turned slower, committed more encroachments, and caused more conflicts than the semi 40, but committed fewer encroachments and caused fewer conflicts than the semi 48. The axle position of the semi 48 made a difference in right turn time, with the larger offtracking of the truck when the axles are back causing a longer turning time, but did not make a difference in left turn time.

Tests in this research were conducted under ideal conditions. Many of the important field test results were based on an experienced driver operating a truck in good condition through a familiar intersection with dry pavement during the day. There remains a need for study of large truck operations under less-than-ideal conditions. Future examinations of large truck operation should include problems associated with inexperienced or impaired drivers, faulty equipment, and wet pavement, for instance.

ACKNOWLEDGMENT

This paper is based on work performed by the authors at Goodell-Grivas, Inc., of Southfield, Michigan, for the Office of Safety and Traffic Operations Research and Development of the Federal Highway Administration. The views and opinions expressed in this paper are those of the authors and do not necessarily reflect the views of the Federal Highway Administration. The authors assume sole responsibility for the accuracy of the data and conclusions presented in this paper.

REFERENCES

1. C. V. Zegeer, J. E. Hummer, and F. Hanscom. *The Operation of Larger Trucks on Roads With Restrictive Geometry*. FHWA/RD-86/157 and FHWA/RD – 86/158 (two volumes), FHWA, U.S. Department of Transportation, July 1986.
2. O. F. Gericke and M. Walton. Effect of Truck Size and Weight on Rural Roadway Geometric Design (and Redesign) Principles and Practices. In *Transportation Research Record 806*, TRB, National Research Council, Washington, D.C., 1981, pp. 13–21.
3. *Test and Demonstration of Double and Triple Trailer Combinations*. Ontario Ministry of Transportation and Communications, Aug. 1982.
4. *Offtracking Characteristics of Trucks and Truck Combinations*. Research Committee Report Number 3, Western Highway Institute, San Francisco, California, September 1970.
5. *Triple Trailer Study in California*. California Department of Public Works in cooperation with the California Highway Patrol, March 1972.
6. F. R. Hanscom. *The Effect of Truck Size and Weight on Accident Experience and Traffic Operations*, Volume II, *Traffic Operations*. FHWA/RD – 80/136. FHWA, U.S. Department of Transportation, July 1981.

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

Abridgment

Magnitude and Severity of Drainage-Structure-Related Highway Accidents

H. DOUGLAS ROBERTSON

The Federal Highway Administration sponsored a study to determine the nature and magnitude of accidents related to roadside drainage structures. Accident data from national and state databases for the years 1981–1984 were analyzed with respect to their relationship to drainage structures. The findings revealed that drainage structures were involved in approximately 9 percent of all accidents on Federal-aid roads and were the first object struck in approximately 4.5 percent of all accidents. A high incidence of fatalities and serious injuries were associated with these accidents. Most of the accidents involved a single vehicle that struck a curb, ditch, embankment, or culvert. Drainage-structure-related accidents predominantly involved a single vehicle and occurred in a higher proportion at night and in adverse weather compared to the same characteristics for all accidents. Based on the findings related to roadway characteristics, drainage-structure accidents were over-represented on Federal-aid secondary roads, at non-junctions, in curves and on grades, and on wet surfaces. This paper contains a brief summary of the study results. A complete documentation of the methodology and findings may be found in FHWA Report DTFH61-85-C-00065.

Safety enhancement is a high priority on federally funded 3R and 4R (resurfacing, restoration, rehabilitation, and reconstruction) programs. Much effort has been directed at reducing roadside hazards through removing, relocating, or protecting fixed objects from errant vehicles. In spite of these improvements, fixed objects were the most harmful event in 47.1 percent of the fatal single-vehicle accidents in 1983.

Research to date has focused largely on improvements to utility poles, sign supports, guardrails, median barriers, and bridge rails. Concern has been expressed that drainage structures may also pose significant safety hazards in run-off-road accidents. Thus the Federal Highway Administration (FHWA) sponsored a study to determine the nature and magnitude of accidents in which drainage structures were involved.

To identify the nature and magnitude of hazardous conditions that are associated with drainage structures is neither a straightforward nor an easy task. While there is a tremendous amount of data available on highway accidents, there are no known accident databases that either uniformly or directly code accidents involving vehicles that strike the various types of drainage structures. The information contained in those databases that code drainage structures is limited and generic in nature.

University of North Carolina at Charlotte, Civil Engineering Department, Charlotte, N.C. 28223.

FINDINGS

The following discussion is based on the results of an analysis of the 1982, 1983, and 1984 National Accident Sampling System (NASS) raw data files and the 1983 NASS weighted data file, which contains an estimate of total accidents and their characteristics. The NASS data are for Federal-aid roads only and represent police-reported accidents where the first harmful event was coded as a collision with a drainage structure. In addition, the NASS raw data files contain information on situations where one or more of the first four objects contacted by any vehicle involved in the accident was a drainage structure.

Computer printouts of one-way and two-way variable tables were obtained from the 1983 NASS weighted data file for both drainage-structure-related accidents and for all accidents on Federal-aid roads. The following discussion summarizes the key findings from an analysis of those tables.

The first question addressed was, "What is the magnitude of drainage-structure-related accidents?" The answer is summarized in table 1. Drainage-structure-related accidents, as defined by first harmful event, constitute approximately 4.5 percent of annual accidents on Federal-aid roads in the United States, or 176,120 of the 3,934,006 police-reported accidents. Accidents involving curbs are the most frequently occurring, representing 31.5 percent of drainage-structure-related accidents and 1.4 percent of all accidents.

The question that came to mind, however, was just how well did the first-harmful-event criterion serve as an indicator of drainage-structure-related accidents? To answer that question, further analyses were performed, using first the 1983 NASS raw data file.

A search of the vehicle file revealed 664 cases in which one or more of the up to four objects struck was either a culvert, ditch, curb, or soft embankment. A cross check of these same cases in the accident file revealed that in 333 of the 664 cases (50 percent) the first harmful event was either a culvert, ditch, curb, or soft embankment. The immediate conclusion, then, was that first harmful event underestimated the number of drainage-structure-related accidents by a factor of two.

To ensure that this was not an anomaly in the 1983 data, a similar analysis was performed on the 1982 and the just-completed 1984 NASS data files. In both cases, drainage structure first harmful events occurred in 51 percent of the cases where a drainage structure was coded as one or more of the objects struck in the vehicle file. Thus, it was concluded

TABLE 1 DRAINAGE STRUCTURE ACCIDENTS BY FIRST HARMFUL EVENT

Object Struck	Frequency	Percent	% of Total Accidents*
Curb	55,440	31.5	1.41
Ditch	37,282	21.2	0.95
Embankment (soft)	33,313	18.9	0.85
Culvert	24,885	14.1	0.63
Wall	20,790	11.8	0.53
Embankment (hard)	4,410	2.5	0.11
Total	176,120	100.0	4.48

* 1983 NASS estimate of 3,934,006 police-reported accidents on Federal-aid roads.

TABLE 2 SEVERITY OF DRAINAGE STRUCTURE ACCIDENTS

Injury-Severity	Frequency	Percent	% Of Total Accidents*	% Of Total Accidents w/ Same Severity
Fatal	2,136	1.2	0.06	9.3
Incapacitating	18,152	10.5	0.47	7.2
Non-incapacitating	26,327	15.3	0.69	5.0
Possible	39,470	23.0	1.03	6.8
None	85,753	50.0	2.24	3.5

*1983 NASS weighted estimate of 3,831,841 accidents on Federal-aid roads with known accident severity.

TABLE 3 COMPARISON OF THE SEVERITY OF DRAINAGE-STRUCTURE ACCIDENTS TO THE SEVERITY OF ALL ACCIDENTS (IN PERCENT)

Injury Severity	Drainage Structure	All Accidents
Fatal	1.2	0.6
Incapacitating	10.5	6.6
Non-incapacitating	15.3	13.8
Possible	23.0	15.0
None	50.0	64.0
Total	100.0	100.0

that drainage-structure-related accidents are involved in approximately 8 to 9 percent of all police-reported accidents on Federal-aid roads.

In addition to occurrence, it is important to assess the severity of these accidents. Table 2 shows that one or more fatalities occurred in 1.2 percent of and incapacitating injuries occurred in 10.5 percent of the drainage-structure-related accidents based on the first-harmful-event criterion from the 1983 NASS weighted data file. In terms of all accidents, fatal accidents represented 0.06 percent and incapacitating injuries represented 0.47 percent. The last column of table 2 indicates that drainage-structure-related accidents represent 9.3 percent of all fatal accidents and 7.2 percent of all incapacitating injury accidents on Federal-aid roads. On the other hand, they represent only 3.5 percent of the accidents with no injuries.

To provide yet another perspective and a basis of comparison, table 3 shows the distribution of all accidents and reveals that drainage accidents are almost twice as severe, in terms of fatalities and incapacitating injuries, as all accidents. One-half of all drainage-structure accidents involve injuries compared to 38 percent of all accidents.

Table 4 characterizes the relative severity of the accidents

TABLE 4 SEVERE (FATAL OR INCAPACITATING INJURY) ACCIDENTS BY TYPE OF DRAINAGE OBJECT STRUCK

Object Struck	Frequency	Percent of Type Object
Embankment (hard)	1,349	30.6
Embankment (soft)	4,425	13.3
Curb	7,324	13.2
Ditch	3,694	9.9
Wall	1,805	8.7
Culvert	1,691	6.8

TABLE 5 COMPARISON OF LIGHT CONDITIONS (IN PERCENT)

Light Condition	Drainage Structure Accidents	All Accidents
Daylight	44.5	62.2
Dark	28.1	11.0
Dark, Lighted	22.8	22.4
Dawn	1.8	1.0
Dusk	2.8	3.4
Total	100.0	100.0

TABLE 6 COMPARISON OF WEATHER CONDITIONS (IN PERCENT)

Weather	Drainage Structure Accidents	All Accidents
No Adverse	69.3	78.4
Rain	18.2	14.8
Sleet	1.6	0.3
Snow	3.7	5.7
Fog	6.6	0.6
Other	0.6	0.2
Total	100.0	100.0

involving each of the drainage-structure types. It shows the number of fatal or incapacitating-injury accidents for each object type and the percent they represent of the total accidents of that type. For example, 30.6 percent (1,349) of the total hard embankment accidents (4,410 from table 1) involve fatalities or incapacitating injuries. While hard embankments exhibit the highest proportion of severe accidents, they occur with the lowest frequency.

It must be remembered that this analysis does not account for exposure. For example, there are more miles of soft embankment than hard ones; therefore, the rate of occurrence (per vehicle miles traveled) might not be the lowest. From table 4, it appears that curb-related accidents are severe and occur with the greatest frequency of all the drainage structure categories.

The remaining findings are based on selected general accident characteristics of drainage accidents compared and contrasted to the same characteristics of all accidents from the 1983 NASS weighted data file. These findings in part describe the nature of drainage-structure-related accidents.

Drainage structure accidents occur at a higher proportion in the dark than all accidents, 28 percent compared to 11 percent (table 5). Table 6 indicates that a higher proportion of drainage-structure accidents occurs in adverse weather than is the case for all accidents.

The incidence of drainage-structure accidents in curves and on grades is twice that of all accidents (tables 7 and 8). Finally, table 9 shows that a higher proportion of drainage structure accidents occurs on a wet road surface than is the case for all accidents.

TABLE 7 COMPARISON OF ROADWAY ALIGNMENT (IN PERCENT)

Alignment	Drainage Structure Accidents	All Accidents
Straight	62.1	83.0
Curved	37.0	17.0
Total	100.0	100.0

TABLE 8 COMPARISON OF ROADWAY PROFILE (IN PERCENT)

Grade	Drainage Structure Accidents	All Accidents
Level	57.7	75.1
Grade (-2%)	40.3	23.0
Hillcrest	0.8	1.0
Sag	1.2	0.9
Total	100.0	100.0

TABLE 9 COMPARISON OF ROADWAY SURFACE CONDITION (IN PERCENT)

Surface Condition	Drainage Structure Accidents	All Accidents
Dry	64.8	70.2
Wet	27.8	21.5
Snow or Slush	3.0	4.5
Ice	4.4	3.8
Total	100.0	100.0

CONCLUSIONS

Based on the findings of the NASS accident data analysis, drainage-structure-related accidents represent eight to nine percent of the total highway safety problem on Federal-aid roadways. These accidents are quite severe. In terms of all accidents, those involving curbs occur most frequently, while in terms of accident severity, hard embankments are the most dangerous. The review of scene photographs suggests that curb design improvements and, in some cases, curb removal would have reduced the severity, if not the occurrence, of many of the curb accidents reviewed.

Drainage-structure-related accidents occur in a higher proportion at night and in adverse weather compared to the same characteristics for all accidents. Based on the findings related to roadway characteristics, drainage-structure accidents are overrepresented in curves, on grades, and on wet surfaces.

REFERENCES

1. *Fatal Accident Reporting System*. National Highway Traffic Safety Administration, Washington, D.C., 1983.
2. H. D. Robertson, S. Basu, K. Colpitts, S. Stein, F. Johnson, and G. K. Young. *Safer Drainage Systems*. Report DTFH61–85–C–00065, Federal Highway Administration, Washington, D.C., July 1986.

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

Effective Use of Passing Lanes on Two-Lane Highways

DOUGLAS W. HARWOOD, CHRIS J. HOBAN, AND DAVEY L. WARREN

Passing lanes have been found to be effective in improving overall traffic operations on two-lane highways. Many of the traffic operation problems on rural two-lane highways result from the lack of passing opportunities due to limited sight distance and heavy oncoming traffic volumes. Passing lanes can provide an effective method for improving traffic operations on two-lane highways at a lower cost than required for constructing a four-lane highway. The paper presents guidelines for effectively locating, designing, signing, and marking passing lanes to improve traffic operations. A procedure for estimating the operational effectiveness of passing lanes in terms of improved service is presented. The paper also presents an evaluation of the effectiveness of passing lanes in reducing accidents on two-lane highways.

A passing lane is an added lane provided in one or both directions of travel on a conventional two-lane highway to improve passing opportunities. This definition includes passing lanes in level or rolling terrain, climbing lanes on grades, and short four-lane sections. The length of the added lane can vary from 1,000 feet to as much as three miles. Figure 1 illustrates a plan view of a typical passing lane section.

Many of the traffic operational problems on rural two-lane highways result from the lack of passing opportunities due to limited sight distance and heavy oncoming traffic volumes. Passing lanes provide an effective method for improving traffic operations on two-lane highways by providing additional passing opportunities at a lower cost than required for constructing a four-lane highway. This lower-cost approach is appropriate because there is a growing backlog of rural roads requiring improvement, and the funds are simply not available to four-lane every two-lane highway that experiences poor levels of service.

FUNCTIONS OF PASSING LANES

Passing lanes have two important functions on two-lane rural roads:

- To reduce delays at specific bottleneck locations, such as steep upgrades where slow-moving vehicles are present and
- To improve overall traffic operations on two-lane highways by breaking up traffic platoons and reducing delays caused by inadequate passing opportunities over substantial lengths of highway

The first function, to reduce delays at bottleneck locations, has been recognized for some time, and guidelines for the provision of climbing lanes on grades have been established. The second function, to improve overall traffic operations, has evolved more recently, particularly as a result of the lack of funds for major road improvements. In practice, many passing lanes perform both functions, and it is often difficult to make a clear operational distinction between the two. The distinction is important, however, in planning and design. The evaluation of a climbing lane considers only the bottleneck location, with the objective of improving traffic operations at the bottleneck to at least the same quality of service as adjacent road sections. For passing improvements, on the other hand, the evaluation should consider traffic operations for an extended road length, typically 5 to 50 miles. Furthermore, the location of the passing improvement can be varied and selecting an appropriate location is an important design decision.

LOCATION AND CONFIGURATION

When passing lanes are provided at an isolated location, their function is generally to reduce delays at a specific bottleneck, and the location of the passing lane is dictated by the needs of the specific traffic problem encountered. Climbing lane design guidelines, for example, usually call for the added lane to begin before speeds are reduced to unacceptable levels and, where possible, to continue over the crest of the grade so that slower vehicles can regain some speed before merging. Requirements for sight distance and taper lengths further define the location of such lanes. In some cases, construction of a climbing lane over the full length of a grade may be too expensive, and the use of shorter lanes over part of the grade may be considered. Recent research at the University of California (*1*) suggests that single short climbing lanes of approximately 1,500 feet near the midpoint of the grade, or two such lanes at the one-third and two-thirds points, are cost-effective methods for providing passing opportunities on long sustained grades. The location of a climbing lane drop on an upgrade section has been found to produce no adverse safety problems, provided sight distance is adequate (*2*).

When passing lanes are provided to improve overall traffic operations over a length of road, they are often constructed at regular intervals. The designer can choose from a number of alternative configurations (*3*), as illustrated in figure 2.

D. W. Harwood, Midwest Research Institute, 425 Volker Boulevard, Kansas City, Mo. 64110. C. J. Hoban, Australian Road Research Board, 500 Burwood Highway, Vermont South 3133, Victoria, Australia. D. L. Warren, Federal Highway Administration, 6300 Georgetown Pike, McLean, Va. 22101.

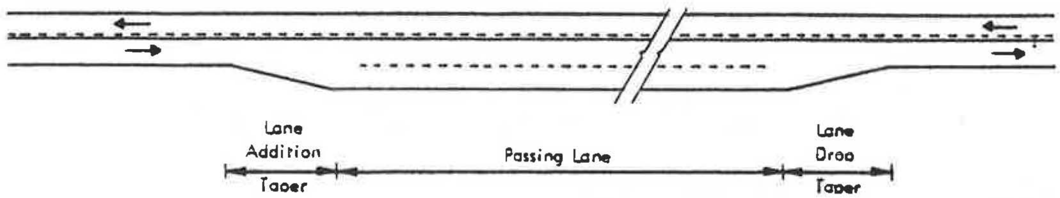


FIGURE 1 Plan view of typical passing lane section.

Factors that should be considered in choosing the location and configuration for passing lanes are discussed below.

Location

A primary objective in choosing the location for a passing lane should be to minimize construction costs, subject to other constraints. Data from several states indicate that the cost of constructing a passing lane can vary from $200,000 to $750,000 per mile, depending on terrain. Climbing lanes in mountainous areas can cost as much as $1,800,000 per mile. Thus, the choice of a suitable location for a passing lane may be critical to its cost-effectiveness. A construction cost profile indicating the longitudinal variation of construction cost per mile along the road can be a useful tool in selecting passing lane locations.

The passing lane location should appear logical to the driver. The value of passing lanes is more apparent to drivers at locations where passing sight distance is restricted than on long tangent sections that already provide good passing opportunities. In some cases, a passing lane on a long tangent may encourage slow drivers to speed up, thus reducing the passing lane effectiveness. At the other extreme, highway sections with low-speed curves are not appropriate for passing lanes, since passing may be unsafe.

The passing lane location may be on a sustained grade or on a relatively level section. If delay problems on a grade are severe, the grade will usually be the preferred location for a passing lane. However, if platooning delays exist for some distance along a road, locations other than upgrades should also be considered for passing lanes. While speed differences between vehicle types are often greater on upgrades than on level or rolling sections, particularly if heavily loaded trucks are present, construction costs and constraints may be greater at such locations. Some types of slow vehicles, such as recreational vehicles, are not slowed by upgrades as dramatically as heavy trucks; passing lanes in rolling terrain may provide opportunities to pass such vehicles that are just as good as the opportunities provided by passing or climbing lanes on upgrades. Passing lanes are also effective on level terrain where the demand for passing opportunities exceeds supply.

The passing lane location should provide adequate sight distance at the lane-addition and lane-drop tapers.

The location of major intersections and high-volume driveways should be considered in selecting passing lane locations, to minimize the volume of turning movements on a road section where passing is encouraged. Low-volume intersections and driveways do not usually create problems in passing lanes. Where the presence of higher-volume intersections and driveways cannot be avoided, special provisions for turning vehicles should be considered. The prohibition of passing by vehicles travelling in the opposing direction should also be considered on passing lane sections with high-volume intersections and driveways.

Locations with other physical constraints, such as bridges and culverts, should be avoided if they restrict the provision of a continuous shoulder.

Passing lanes can also be constructed as part of realigning a road segment that has safety problems.

Configuration

Separated or adjoining passing lanes (shown as (c) through (f) in figure 2) are often used in pairs, one in each direction, at regular intervals along a two-lane highway.

Where pairs of adjoining passing lanes are used and passing by opposing direction vehicles is prohibited, the use of configuration (e) in figure 2 has the advantage of building platoons before the passing lane, whereas the reverse configuration tends to rebuild platoons more quickly after the passing lane. This configuration is also preferred because the lane-drop areas of the opposing passing lanes are not located adjacent to each other.

Transitions between passing lanes in opposing directions should be carefully designed; intersections, bridges, two-way left-turn lanes, or painted medians can often be used effectively to provide a buffer area between opposing passing lanes.

Alternating passing lanes (shown as (g) and (h) in figure 2) are sometimes appropriate where a wide pavement is already available. However, the provision of passing lanes over 50 percent of the road length is probably excessive. Drivers may also feel unduly constrained when passing is prohibited on the other 50 percent of the road length if sight distance is good and traffic volumes are low.

Short, four-lane sections, both divided and undivided, are particularly appropriate where the ultimate design is for the highway to have four lanes. Construction of short, four-lane sections at the least expensive locations can provide a substantial proportion of the benefits of the ultimate design for a relatively small proportion of the total cost, particularly if major bridge work or right-of-way acquisition can be avoided. This staged four-laning will generally return a high benefit-cost ratio, while economic justification for the remaining stages will increase with increasing traffic volumes in future years. Where the ultimate design is uncertain or the need for it is many years away, however, the use of lower cost options should be considered.

Overlapping passing lanes (shown as (i) and (j) in figure 2) are often used at crests where a climbing lane is provided on each upgrade.

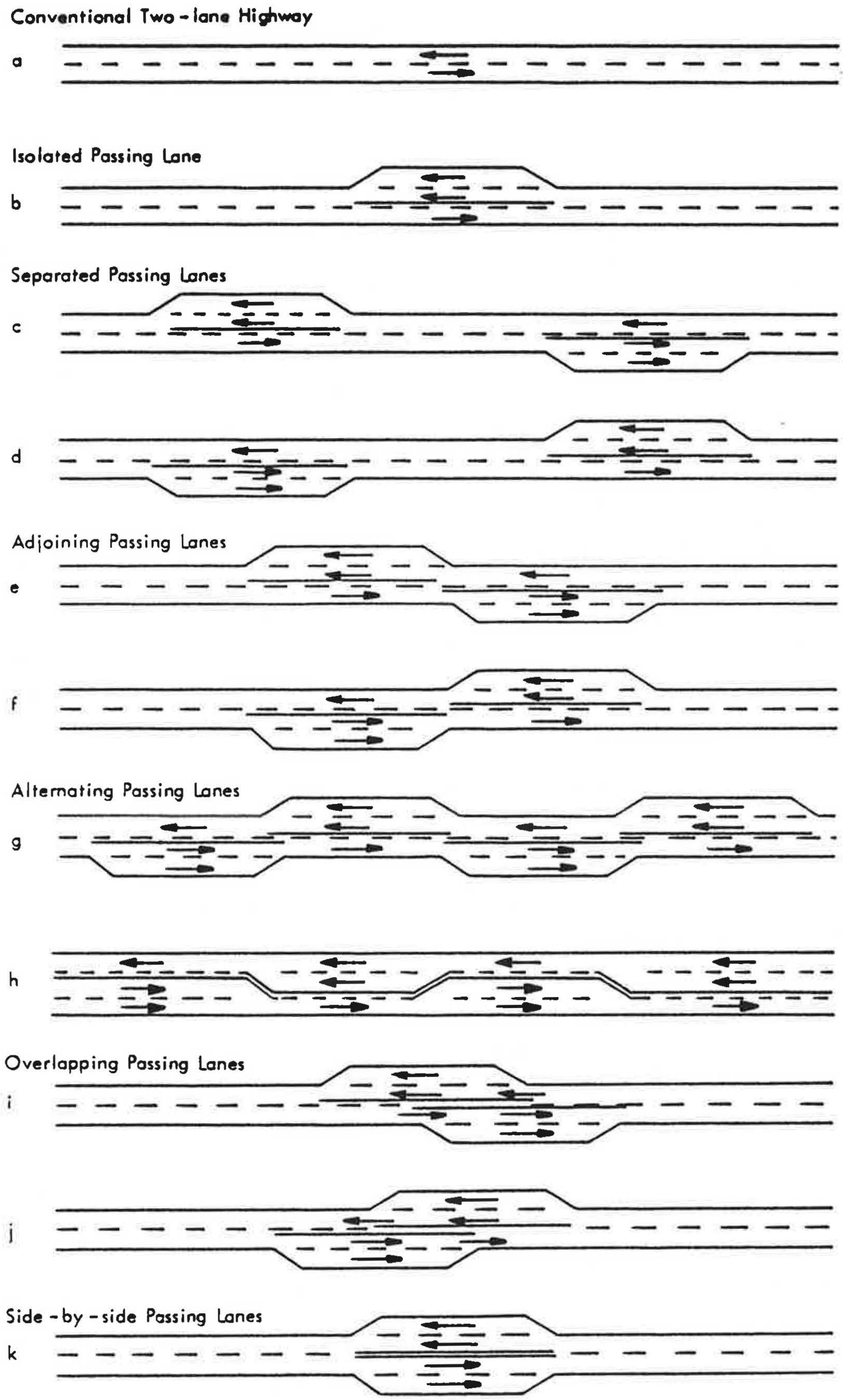


FIGURE 2 Alternative configurations for passing lanes.

GEOMETRIC DESIGN

The length of the passing lane, the lane and shoulder widths, and the lane-addition and lane-drop taper designs should be considered in the geometric design of passing lanes. The following guidelines for geometric design were developed by Harwood and Hoban (*5*).

Passing lanes should generally be from 0.5- to 1.0-mile long, excluding tapers. Passing lanes less than 0.5-mile long are usually not effective in creating additional passing opportunities, and passing lanes over 1.0-mile long are usually not cost-effective (*5, 6*). The choice of an optimal design length for passing lanes on two-lane highways should be a function of the traffic flow rate and is addressed in a later section of this paper.

The lane widths in a passing lane section usually should not be narrower than the lane widths on the adjacent sections of two-lane highway; 12-foot lane widths are desirable. It is also desirable for passing lane sections to have a minimum four-foot shoulder width on either side of the highway. Wherever possible, the shoulder width in a passing lane section should not be narrower than the shoulder width on the adjacent sections of two-lane highway.

The lane-addition and lane-drop transition areas at the beginning and end of a passing lane should be designed to encourage safe and efficient traffic operations. Many highway agencies have used relatively short lane-addition and lane-drop tapers at passing lanes. However, the use of longer tapers should be encouraged to minimize traffic conflicts and to get the greatest operational benefit from the investment in passing lanes.

The lane-drop taper at the downstream end of a passing lane should be designed in accordance with the requirements for lane reduction transitions set by the FHWA Manual on Uniform Traffic Control Devices for Streets and Highways (MUTCD), Section 3B-8. The recommended geometric configuration is to terminate the right lane with a lane-drop taper and merge the traffic from both lanes into a single lane. In a few cases, such as alternating passing lanes on a three-lane pavement of constant width, dropping the left lane is appropriate. The lane-drop taper length should be computed from the formula $L = WS$, where L is the taper length in feet, W is the width of the dropped lane in feet, and S is the prevailing off-peak 85th percentile speed in miles per hour (mph). At the termination of a 12-foot lane, the required taper length for a 60-mph prevailing speed is 720 feet. A wide shoulder is desirable at the lane-drop taper to provide a recovery area in case drivers encounter a merging conflict.

There is no MUTCD requirement for the length of the lane-addition taper at the upstream end of a passing lane. The diverge maneuver does not require as much length as the merge maneuver, but a good lane-addition transition design is needed for effective passing lane operations. The recommended length for a lane-addition taper is half to two-thirds of the length of a lane-drop taper, or 360 to 480 ft in the example of the 60-mph design speed presented above.

Passing lanes are most effective if the majority of drivers enter the right lane at the lane-addition transition and use the left lane only when passing a slower vehicle. Little or no operational benefit is gained from passing lanes if most drivers continue in the left lane. The geometric design of the lane-addition transition area, together with appropriate signing and marking (discussed below) should encourage drivers to enter the right lane of the passing lane section.

Safe and effective passing lane operations require adequate sight distance on the approach to both the lane-addition and lane-drop tapers. Inadequate sight distance in advance of the lane-addition taper may result in lack of readiness by vehicles wishing to pass, so that some of the length of the passing lane is wasted. When sight distance approaching the lane-drop taper is limited, vehicles may merge too early or too late, resulting in erratic behavior and poor use of the passing lane. Therefore, passing sight distance appropriate for the speed of the highway on the approach to each taper is recommended. Above-minimum passing sight distance in the taper areas is desirable.

TRAFFIC CONTROL DEVICES

The signing and marking of passing lanes is partially addressed in the MUTCD (*4*), which indicates the appropriate centerline markings for passing lanes and the signing and marking of lane-drop transitions areas. The following discussion extends the MUTCD criteria to provide a consistent set of traffic control devices for use at passing lanes, as illustrated in figure 3. The recommended signing and marking practice presented here were developed by Harwood and Hoban (*5*) from review of the practices of 13 states (*6*) and the practices used in Australia and Canada (*3*). The recommended practice is presented here not to suggest that it should be adopted in precisely this form by every highway agency, but to illustrate the types of signs and markings that are needed for effective operation of passing lanes.

Signing

Signing is needed to convey information to drivers at three locations at passing lane sites:

- In advance of the passing lane,
- At the lane addition, and
- In advance of the lane drop.

Advance Signing

A sign with the legend PASSING LANE ½ MILE should be placed 0.5 mile in advance of each passing lane. This sign provides advance notice of the passing lane to the drivers of both slow-moving vehicles and following vehicles so that they can prepare to make effective use of the passing lane. Additional advance signs are desirable two to five miles in advance of a passing lane. Such advance signing may reduce the frustration and impatience of drivers following a slow-moving vehicle because they know they will soon have an opportunity to pass. Driver frustration and impatience when following slow-moving vehicles has been shown to be a potential safety problem on two-lane highways. Hostetter and Seguin (*7*) found, for example, that when forced to follow a slow-moving vehicle for up to 5 miles, almost 25 percent of drivers passed illegally in a no-passing zone.

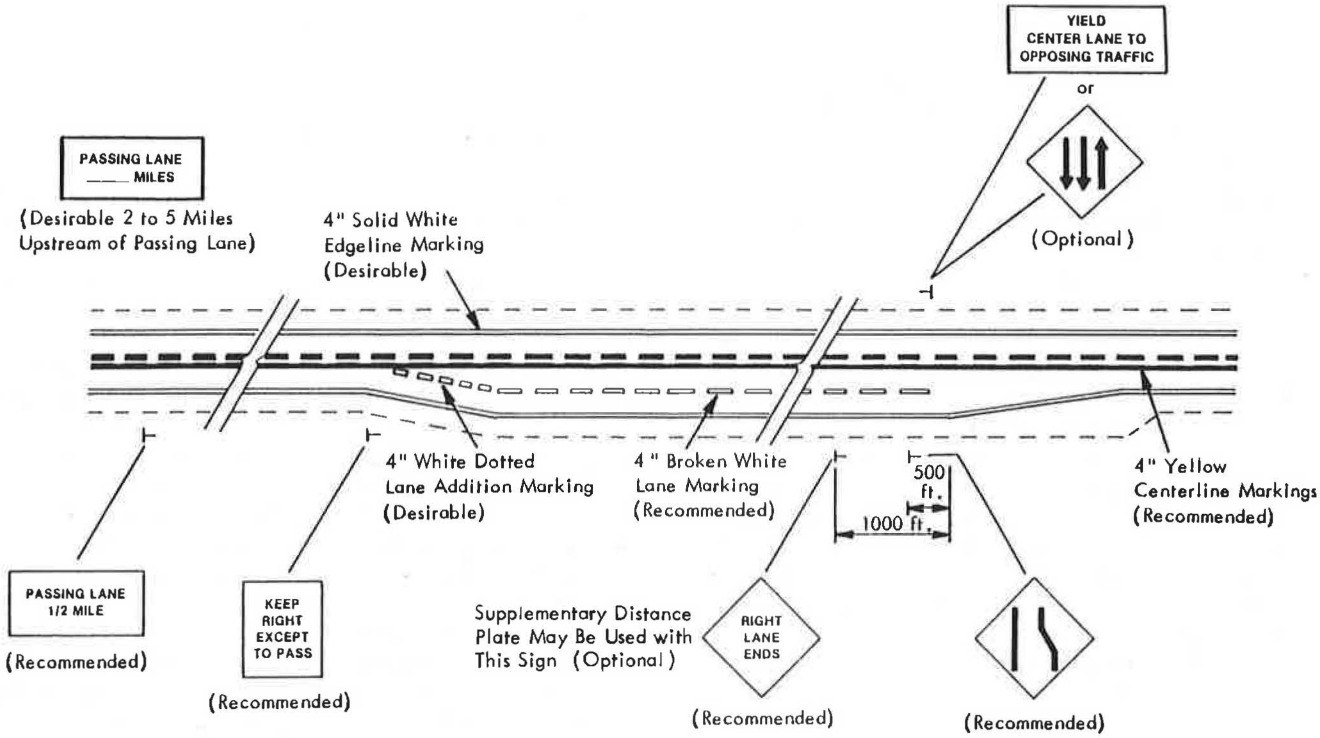


FIGURE 3 Recommended signing and marking practices for passing lanes.

Lane-Addition Signing

A black-on-white regulatory sign with the legend KEEP RIGHT EXCEPT TO PASS should be placed at the beginning of the lane-addition taper. This sign, in conjunction with the geometrics and pavement markings at the lane-addition taper, informs drivers of the beginning of the passing lane and encourages them to enter the right lane unless they are immediately behind a vehicle they wish to pass. An acceptable alternative legend for this sign is SLOWER TRAFFIC KEEP RIGHT, although this legend is not preferred because it provides less definite instructions to drivers. Sign legends that refer specifically to trucks, such as TRUCKS USE RIGHT LANE, are not recommended because they appear to exclude other vehicle types, such as slow-moving recreational vehicles and passenger cars, which should also be encouraged to use the right lane.

Lane-Drop Signing

The MUTCD requires a black-on-yellow warning sign, either a symbol sign or a text sign, in advance of a lane drop. According to MUTCD table II-1, for a prevailing speed of 60 mph, a single warning sign should be placed 775 feet in advance of a decision point that requires a high degree of judgment, such as a lane-drop merging maneuver. Many highway agencies use two warning signs in advance of the lane-drop transition areas of passing lanes, and this practice is recommended. The first advance warning sign with the legend RIGHT LANE ENDS, should be located 1,000 feet in advance of the lane-drop taper. This sign may carry a supplemental distance plate (for example, 1,000 FEET) below the sign. The second advance warning sign should be the lane reduction transition symbol sign and should be located 500 feet in advance of the lane-drop taper.

Signing for Opposing Traffic

Highway agencies that generally provide signing for passing and no-passing zones on conventional two-lane highways, including the DO NOT PASS sign, the PASS WITH CARE sign, and the pennant-shaped NO PASSING ZONE sign, usually continue this practice in the opposing direction of travel at passing lane sites. Where passing by vehicles travelling in the opposing direction is permitted, some agencies use a regulatory sign specifically appropriate to passing lanes, such as YIELD CENTER LANE TO OPPOSING TRAFFIC, in place of the PASS WITH CARE sign. An alternative sign for use in the opposing direction to a passing lane is the three-arrow sign used in Australia, which is illustrated in figure 3. This sign does not identify whether passing by vehicles travelling in the opposing direction is permitted or prohibited, but it does inform drivers that there are two lanes of oncoming traffic.

Marking

A passing lane section with two lanes in one direction of travel and one lane in the opposite direction of travel should be marked in accordance with MUTCD figure 3-2. A yellow centerline marking should be used to separate the lanes normally used by traffic moving in opposite directions. A broken white lane line is used to separate traffic in lanes normally moving in the same direction. Pavement edge lines are desirable on both sides of the highway in passing-lane sections to guide drivers and to delineate the boundary between the pavement and shoulder.

Passing by vehicles travelling in the opposing direction to a passing lane may be either permitted or prohibited, as illustrated in MUTCD figure 3-2. A study by Harwood and St. John (*6*) found no difference in cross-centerline accident rates between passing lane sections where passing in the opposing direction was prohibited and passing lane sections where passing in the opposing direction was permitted where adequate sight distance was available. Therefore, passing by opposing-direction vehicles may be allowed where sight distance is adequate. This finding indicates that passing lanes where passing by opposing direction vehicles is permitted do not have safety problems of the type that occurred many years ago on three-lane highways with center lanes available for unrestricted use by vehicles travelling in either direction. Passing zones should be marked for the opposing direction of travel in passing lanes where warranted by the same criteria used in marking normal two-lane highways, specified in MUTCD Section 3B-5. For a 60-mph prevailing speed, a no-passing zone is warranted in the opposing direction of travel where sight distance is less than 1,000 feet.

It is not a desirable practice to prohibit passing by vehicles travelling in the opposing direction at all passing lane sites, because this unnecessarily reduces the level of service in that direction of travel. Prohibition of passing in the opposing direction at all passing lanes, regardless of sight distance, may be counterproductive to improved safety, since some drivers travelling in the opposing direction may be tempted to pass despite the prohibition in areas of good sight distance. Some agencies may choose to institute a site-by-site review of passing lanes and prohibit opposing direction passing at particular sites on the basis of unusual geometrics, roadside development, high traffic volumes, or similar factors, in addition to limited sight distance. The prohibition of passing by vehicles travelling in the opposing direction is particularly appropriate at sites with roadside development that generates frequent left-turn movements from the left lane of the treated direction in the passing lane section.

Lane-Addition Markings

The MUTCD does not provide any specific guidance for marking a lane-addition transition area. The recommended pavement marking scheme is illustrated in figure 3. The use of a pavement edge marking in the lane-addition transition area is recommended. A white dotted marking tapering across the left lane immediately prior to the beginning of the lane line is recommended. Several highway agencies have found this marking to be effective in guiding most drivers into the right lane. Drivers who desire to pass immediately upon entering the passing lane are permitted to cross the dotted marking.

Lane-Drop Markings

Pavement markings in the lane-drop transition area should be provided in accordance with MUTCD Section 3B-8, as illustrated in MUTCD figure 3-10. For a 60-mph prevailing speed, the broken white lane line should be discontinued 580 feet prior to the beginning of the lane-drop taper. The use of a pavement edge marking in the lane-drop transition area is recommended.

OPERATIONAL EFFECTIVENESS

The operational effectiveness of passing lanes on two-lane highways has been evaluated extensively in Australia, Canada, and the United States. The results of the recent evaluation of passing lanes in the United States are summarized in the following discussion to provide guidance on where passing lanes should be used and what operational benefits should be expected. International research has also demonstrated the effectiveness of passing lanes. Australian research has resulted in the development of minimum-volume warrants for passing lanes based on average daily traffic (ADT) volumes and percent of highway length providing passing opportunities over the previous 2 to 6 miles (*8*). Canadian research has developed a concept based on the percentage of highway length with "assured" passing opportunities to determine where passing lanes are needed (*9, 10*). Summaries of these results have also been presented by Harwood and Hoban (*5*).

The research approach used in the United States has focused on tying the operational effectiveness of passing lanes to the levels of service for two-lane highways used in Chapter 8 of the 1985 Highway Capacity Manual (HCM) (*11*). These levels of service, illustrated in table 1, are defined in terms of the percentage of travel time spent delayed, such as travelling in platoons behind other vehicles. The percent of time delay was chosen as the measure of service for the 1985 HCM because it is more sensitive to variation in flow rate than other candidate measures, such as vehicle speeds (*12*). On steep grades, the average upgrade speed serves as an additional criterion to define the levels of service.

The operational effectiveness of passing lanes in the United States was previously evaluated based on field data by Harwood and St. John (*6*) and Harwood, St. John, and Warren (*13*). This field evaluation compared the quality of traffic operations (level of service) upstream and downstream of passing lanes. Field evaluations cannot compare the quality of traffic operations on a highway section with and without passing lanes, but comparisons of this type can be made with a computer simulation model. Therefore, simulation modeling of passing lanes was recently conducted with a computer model known as TWOPAS (*14*).

TWOPAS is a microcomputer simulation model of traffic operations on two-lane, two-way highways. TWOPAS is a

TABLE 1 LEVEL OF SERVICE CRITERIA FOR TWO-LANE HIGHWAYS

Level of Service	Percent Time Delay on General Segments	Average Upgrade Speed (mi/hr) on Specific Grades
A	≤ 30	≥ 55
B	≤ 45	≥ 50
C	≤ 60	≥ 45
D	≤ 75	≥ 40
E	> 75	≥ 25-40
F	100	< 25-40

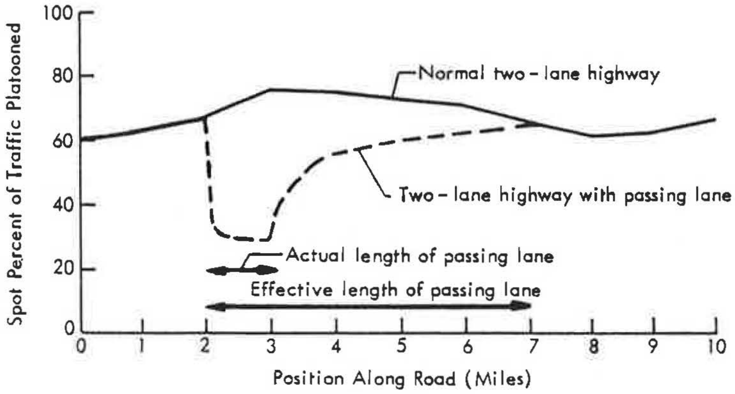


FIGURE 4 Example of the effect of a passing lane on two-lane highway traffic operations.

modified version of the TWOWAF model used in the development of Chapter 8 of the 1985 HCM. TWOPAS has the added capability to simulate the operational effects of passing and climbing lanes. The TWOWAF model was validated from field data for conventional two-lane highways by St. John and Kobett (*15*) and by Messer (*12*), and the added capability to simulate passing and climbing lanes was validated from field data by Harwood and St. John (*14*). The latter effort found good agreement between model results and field data for traffic platooning and traffic speeds upstream and downstream of passing lanes.

Figure 4 presents a conceptual illustration of the effect of a passing lane on traffic operations on a two-lane highway. The solid line in this figure shows the normal fluctuation of platooning on a two-lane highway with the availability of passing sight distance. When a passing lane is added, the percentage of vehicles following in platoons falls dramatically and stabilizes at about half the value for the two-lane road. Because platoons are broken up in the passing lane, its effective length extends for a considerable distance downstream of the passing lane. Thus, the installation of passing lanes on parts of a two-lane highway can improve traffic operations on the entire highway. The next section of the paper illustrates the determination of the effective length of passing lanes for different lengths and traffic flow rates, based on computer simulation results.

Effective Length of a Passing Lane Used for Analysis

Figure 5 illustrates the effects of passing lanes of various lengths on traffic platooning within a passing lane and downstream of a passing lane for flow rates of 400 and 700 vehicles per hour (vph) in one direction of travel. Figure 5 is based on the percentage of vehicles delayed in platoons at specific spot locations on the highway. It can be seen in figure 5 that the level of traffic platooning within a passing lane is less than half of the level observed upstream of the passing lane. Traffic platooning remains at a reduced level downstream of a passing lane. For a flow rate of 400 vph, the effects of passing lanes can still be substantial seven miles downstream of the beginning of the passing lane, especially for longer passing lanes. At the higher flow rate of 700 vph, nearly all of the operational benefits of the passing lane are gone within five miles, although there is a small residual effect even at seven miles downstream. The length of the passing lane has a strong influence on the improvement in traffic operations immediately downstream of the passing lane, but this differential between passing lane lengths largely disappears farther downstream.

The results in figure 5 indicate that the effective length of a passing lane can vary from three to eight miles, depending on passing lane length, traffic flow and composition, and downstream passing opportunities.

The concept of effective length is needed for analysis purposes to determine the overall effect of a passing lane on level of service over an extended highway section. For most cases, effective length can be estimated from figure 5, with adjustments for factors that might hasten or slow the downstream overtaking or catch-up process. If the two-lane highway downstream of the passing lane has few passing opportunities, for example, the effective length determined from figure 5 should be reduced.

In some cases, the effective length of a passing lane is constrained by other road features, such as small towns, four-lane sections, or additional passing lanes a few miles downstream. In these situations, the distance to the downstream constraint should be used as the effective length for analysis purposes, if this is less than that estimated from figure 5.

Effectiveness Over an Extended Road Section

Figure 6 illustrates the effectiveness of passing lanes of various lengths in improving traffic operations on two-lane highways, based on results obtained with the TWOPAS simulation model. The curves presented in figure 6, for passing lanes of varying lengths, represent their effectiveness in increasing traffic speeds and decreasing the percent of time vehicles spend delayed in platoons on a two-lane highway in moderately rolling terrain. The vehicle speed and platooning measures in figure 6 are averages over an eight-mile highway section with the passing lane located at the beginning; thus, these curves represent the combined effects of improved traffic operations in the passing lane and downstream of the passing lane. Figure 6 illustrates that passing lanes produce relatively small increases in vehicle speeds, but can dramatically decrease vehicle platooning.

An eight-mile highway section is used in figure 6 because

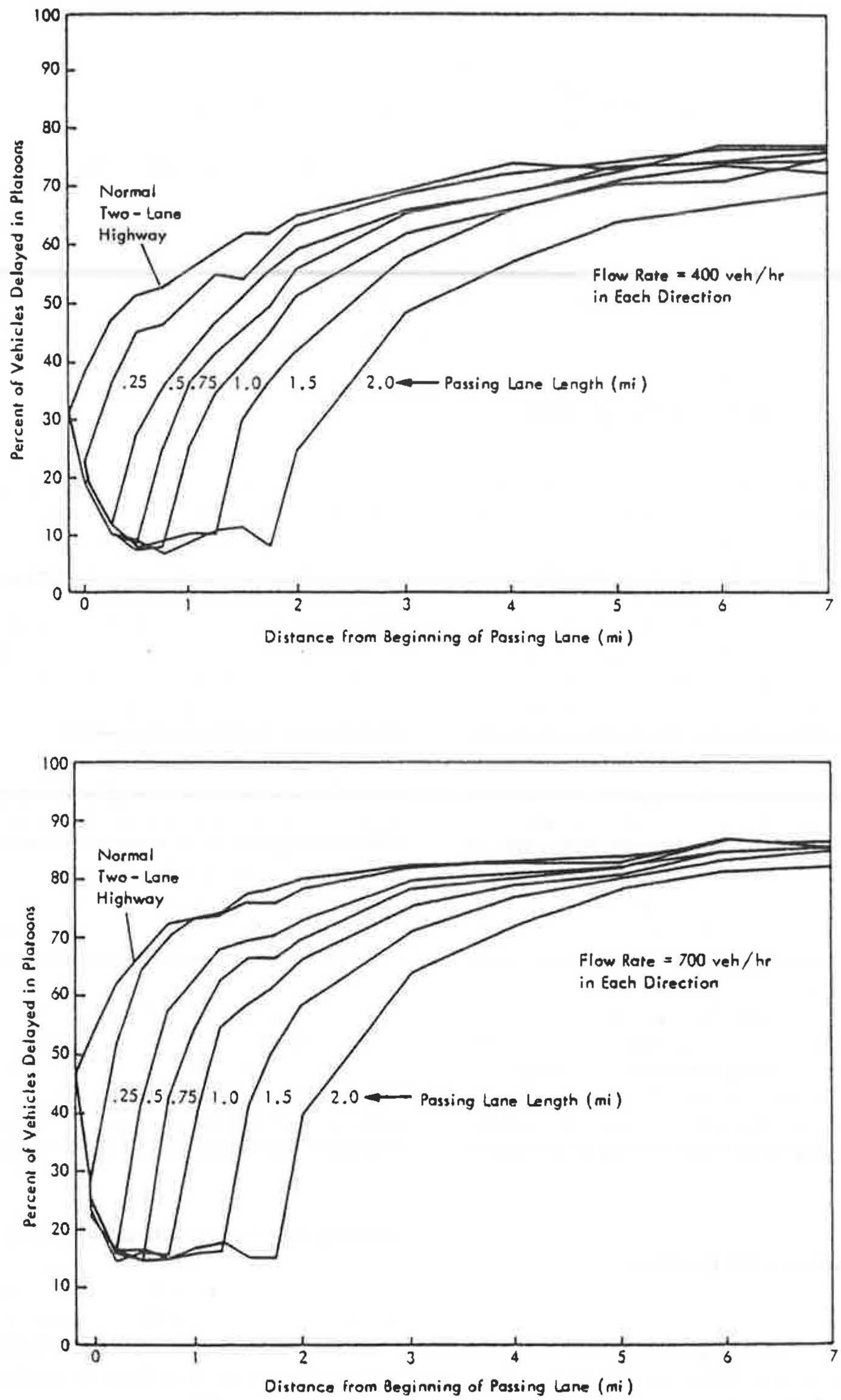
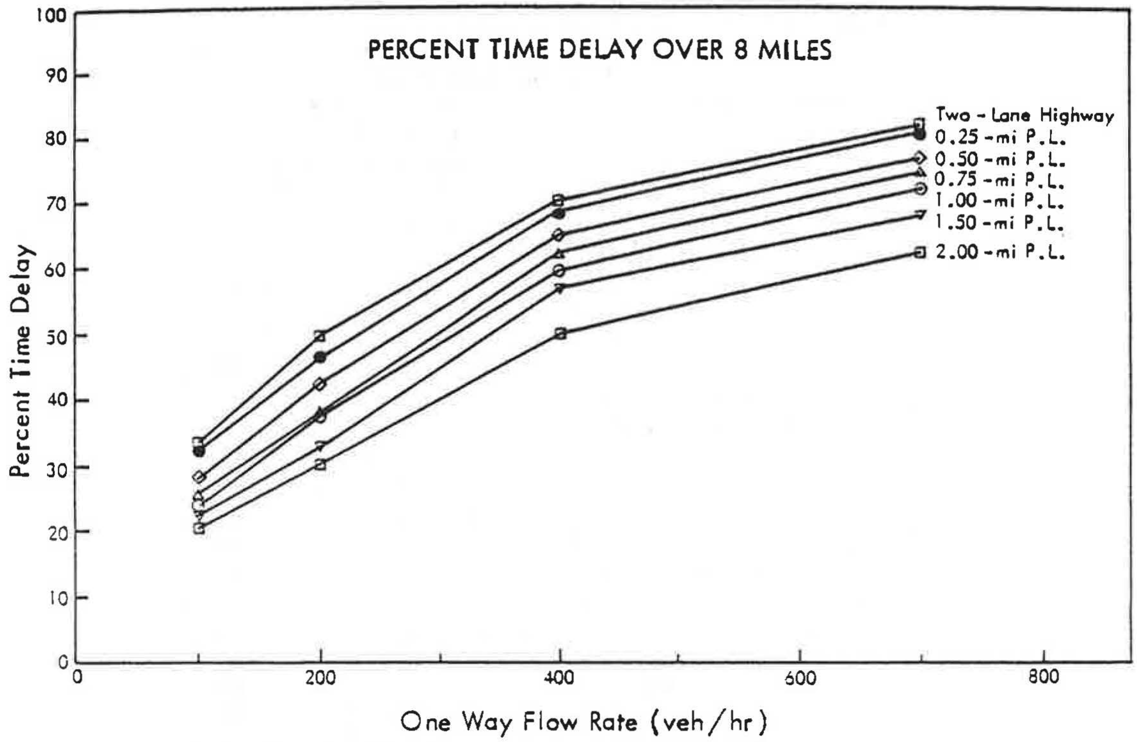


FIGURE 5 Gradual increase in percentage of vehicles delayed in platoons downstream of passing lanes.



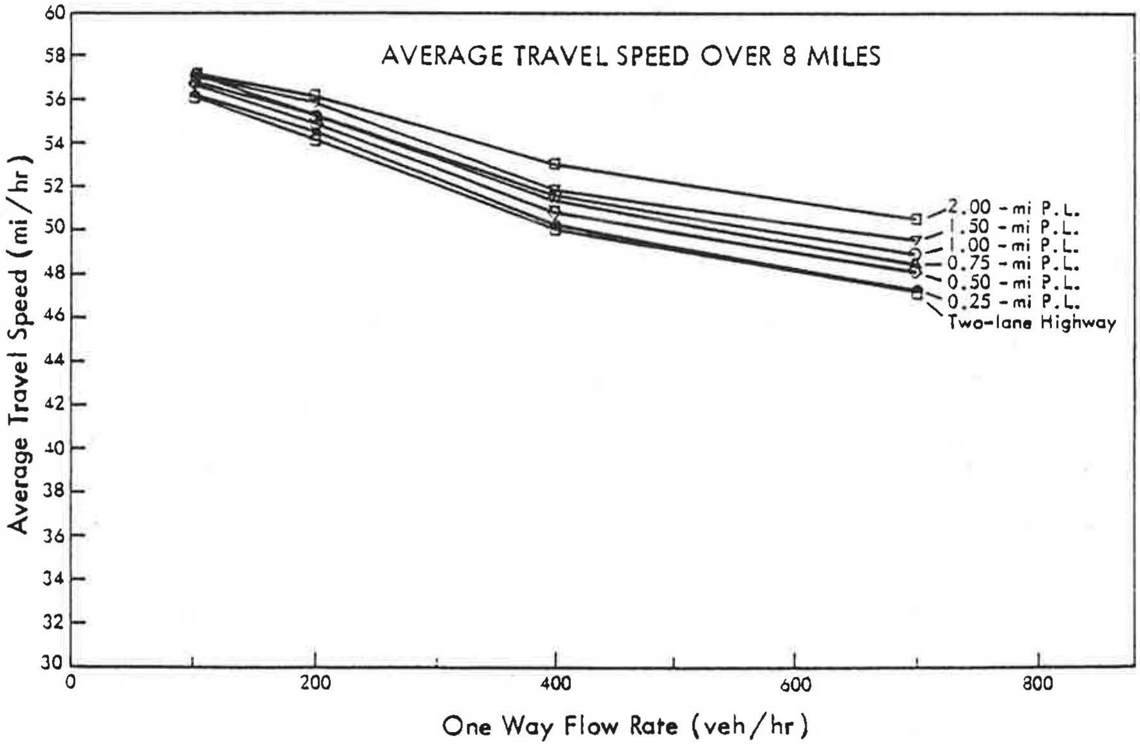


FIGURE 6 Computer simulation results for operational effectiveness of passing lanes.

TABLE 2 EFFECT OF PASSING LANES ON PERCENT TIME DELAY OVER AN EXTENDED ROAD LENGTH

Effective Length (mi)	PERCENT TIME DELAY — Passing Lane Length (mi) 0	0.25	0.50	0.75	1.00	1.50	2.00
One-way Flow Rate = 100 veh/hr							
3	33	30	20	17	17	17	17
5	33	31	25	22	19	17	17
8	33	32	28	26	24	22	20
One-way Flow Rate = 200 veh/hr							
3	50	39	29	25	25	25	25
5	50	44	37	31	29	25	25
8	50	46	42	38	37	33	30
One-wqy Flow Rate = 400 veh/hr							
3	70	67	57	49	43	35	35
5	70	68	62	57	54	49	38
8	70	69	65	62	60	57	50
One-way Flow Rate = 700 veh/hr							
3	82	79	69	63	55	45	41
5	82	80	74	71	66	60	52
8	82	81	77	75	72	68	63

the effective length of a passing lane includes both the passing lane itself and the downstream section of two-lane highway where platooning is lower than it would be without the passing lane. Table 2 presents the estimated reductions in percent time delay for three different effective lengths—3, 5, and 8 miles—as well as for different lengths of passing lane.

The selection of the design length of a passing lane is discussed in the following sections. Once the design length and the effective length used for analysis are determined, table 2 can be used to predict the percent time delay and, hence, the level of service on a highway section which includes a passing lane.

It should be noted that the base values of percent time delay for a normal two-lane highway in table 2 are higher than those specified in the HCM (see table 1) for ideal conditions. This is because the simulated results were derived for non-ideal conditions of terrain, no-passing zones, and traffic composition. Since these conditions can vary from one case to another, it is recommended that table 2 be entered using a given base value of percent time delay, rather than the traffic flow. In other words, the estimated two-lane highway percent time delay should be used to select the appropriate row of table 2, regardless of traffic flow. Linear interpolation in table 2 is acceptable.

Optimum Design Length for Passing Lanes

The optimum design length for a passing lane can be determined through a cost-effectiveness analysis. This can be illustrated by the data in table 3, which presents the percent time delay over an effective length of eight miles for passing lanes of various design lengths, the difference between the percent time delay for each design length and a conventional two-lane highway, and the ratio of this difference to the design length. This effectiveness ratio, the effectiveness in reducing vehicle platooning per unit length of passing lane, represents the relative cost-effectiveness of passing lanes, if one assumes that the cost of constructing a passing lane is proportional to its length. This assumption is reasonable for most situations, although the cost of constructing passing lanes can vary widely as a function of terrain. The passing lane lengths shown in table 3 were increased by 600 feet, half of the combined length

TABLE 3 REDUCTION IN PERCENT TIME DELAY PER UNIT LENGTH OF PASSING LANE

One-Way Flow Rate (veh/hr)	Passing Lane Length (mi) a/ 0.25	0.50	0.75	1.00	1.50	2.00
100	2.8	8.2	8.1	8.1	6.8	6.2
200	11.1	13.1	14.0	11.7	10.6	9.5
400	2.8	8.2	13.1	9.0	8.1	9.5
700	2.8	8.2	8.1	9.0	8.7	9.0

a/ Unit length of passing lanes increased by 600 ft to account for cost of constructing lane addition and lane drop tapers.

TABLE 4 OPTIMAL DESIGN LENGTHS FOR PASSING LANES

One-Way Flow Rate (veh/hr)	Optimal Passing Lane Length (mi)
100	0.50
200	0.50-0.75
400	0.75-1.00
700	1.00-2.00

of typical lane-addition and lane-drop tapers, in the computation of the effectiveness ratios to account for the cost of constructing these transition areas.

The optimum design lengths for passing lanes, based on the data in table 3, are tabulated in table 4. For flow rates of 200 vph or less in one direction of travel, the highest effectiveness per unit length is obtained for passing lanes with design lengths between 0.5 and 0.75 of a mile. Passing lanes shorter than 0.5 mile or longer than 0.75 mile are not as desirable at this flow rate because they provide less operational benefit per unit length. As flow rate increases above 200 vph, the optimum design length for a passing lane also increases. At a flow rate of 400 vph in one direction of travel, the optimum design length for a passing lane is 0.75 to 1.0 mile. At very high flow rates, such as 700 vph in one direction of travel, the optimum design length of passing lanes ranges from 1.0 to 2.0 miles. However, passing lanes longer than 1.0 mile may not be desirable, even for highways with peak flow rates of 700 vph in one direction of travel, because longer passing lanes would be suboptimal throughout the remainder of the day when traffic volumes are lower.

The effectiveness analysis indicates that short passing lanes are usually more effective per unit length and, therefore, per dollar spent on construction than long passing lanes. Thus, the overall level of service on a highway can often be improved more by constructing three 0.5-mile passing lanes spaced at intervals than by constructing one two-mile passing lane. The optimum design length for passing lanes on a specific section of two-lane highway could be based on the highest hourly flow rate that occurs frequently (for example, on a daily basis) on that specific highway section. The design hour volume, which occurs in only a few hours out of each year, may be too high to serve as the basis for the choice of a cost-effective passing lane length. It may be useful to evaluate traffic operations for several design hours, especially when the composition of traffic differs between weekdays and weekends.

SAFETY EFFECTIVENESS

Safety evaluations have shown that passing lanes and short four-lane sections reduce accident rates below the levels found on conventional two-lane highways.

Table 5 compares the results of two before-and-after evaluations of passing lane installation. These studies include accidents of all types for both directions of travel within the portion of the two-lane highway where the passing lanes were installed. A California study by Rinde (*16*) at 23 sites in level, rolling, and mountainous terrain found accident rate reductions due to passing lane installation of 11 to 27 percent, depending on road width. The accident rate reduction effectiveness at the 13 sites in level or rolling terrain was 42 percent. In data from 22 sites in four states, Harwood and St. John (*6*) found the accident rate reduction effectiveness of passing lanes to be 9 percent for all accidents and 17 percent for fatal and injury accidents. The combined data from both studies indicates that passing lane installation reduces accident rate by 25 percent. No difference was found between the accident rates of passing lanes of level and rolling terrain.

Harwood and St. John (*6*) found no indication in the accident data of any marked safety problem in either the lane-addition or lane-drop transition areas of passing lanes. In field studies of traffic conflicts and erratic maneuvers at the lane-drop transition areas of 10 passing lanes, lane-drop transition areas were found to operate smoothly. Overall, 1.3 percent of the vehicles passing through the lane-drop transition area created a traffic conflict, while erratic maneuver rates of 0.4 and 0.3 percent were observed for centerline and shoulder encroachments, respectively. The traffic conflict and

TABLE 5 ACCIDENT REDUCTION EFFECTIVENESS OF PASSING LANES

Source	Type of Terrain	Total Roadway Width (ft)[a]	No. of Passing Lane Sites	Percent Reduction: All Accidents	Percent Reduction: Fatal and Injury Accidents
Rinde[16]	Level, rolling, and mountainous	36	4	11	-
		40	14	25	-
		42-44	5	27	-
	Level and rolling sites only	36-44	13	42	-
Harwood and St. John[6]	Level and rolling	40-48	22	9	17
Combined Totals for Level and Rolling Terrain			35	25	-

[a] Total roadway width includes both traveled way and shoulders.

TABLE 6 RELATIVE ACCIDENT RATES FOR IMPROVEMENT ALTERNATIVES

Alternative	All Accidents	Fatal and Injury Accidents
Conventional two-lane highway	1.00	1.00
Passing lane section	0.75	0.70
Four-lane section	0.65	0.60

encroachment rates observed at lane-drop transition areas in passing lanes were much smaller than the rates found in lane-drop transition areas at other locations on the highway system, such as work zones.

An evaluation of cross-centerline accidents involving vehicles travelling in opposite directions on the highway found no safety differences between passing lanes with passing prohibited in the opposing direction and passing lanes with passing permitted in the opposing direction where adequate sight distance was available (*6*). The provision for passing by vehicles travelling in the opposing direction does not appear to lead to safety problems at the types of sites and flow rate levels (up to 400 vph in one direction of travel) where it has been permitted by the highway agencies that participated in the Harwood and St. John study. Both types of passing lanes had cross-centerline accident rates lower than those of comparable sections of conventional two-lane highway.

Reviewing a small number of climbing-lane sites in the United States, Jorgensen (*17*) found no change in accident experience. In the United Kingdom, Voorhees (*18*) found a 13 percent reduction in accidents where a climbing lane was provided.

A safety evaluation of nine short, four-lane sections in three states found a 34 percent lower total accident rate and a 43 percent lower fatal and injury accident rate on the short, four-lane sections than rates on comparable sections of conventional two-lane highways (*5*). These differences, although substantial, were not statistically significant because of the limited number of sites available. The cross-centerline accident rates for the short, four-lane sections were generally less than half the rates for the comparable two-lane sections.

Table 6 summarizes the relative accident rates found in recent research for passing lane sections and short, four-lane sections, expressed as ratios between the expected accident rate for each and the expected accident rate of a conventional two-lane highway.

SUMMARY

Passing lanes have been found to be effective in improving overall traffic operations on two-lane highways, and they provide a lower cost alternative to four-laning extended sections of highway. Passing opportunities on two-lane highways can be increased by the installation of passing lanes in level and rolling terrain, of climbing lanes on sustained grades, and of short sections of four-lane highway. The traffic operational effectiveness of passing lanes can be predicted as a function of flow rate, passing-lane length, and the percentage of traffic travelling in platoons, using the procedure presented above. The installation of a passing lane on a two-lane highway reduces accident rate by approximately 25 percent. Recommended geometric design, signing, and marking practices for passing lanes have also been developed. Further guidance on the use of passing lanes and other low-cost methods of improving traffic operations on two-lane highways (such as turnouts, shoulder driving sections, intersection turn lanes, and center two-way left-turn lanes) is provided by Harwood and Hoban (*5*).

ACKNOWLEDGMENTS

The work reported in this paper was conducted under sponsorship of the Federal Highway Administration. However, the findings and conclusions in the paper are those of the authors and do not necessarily represent the views of the Federal Highway Administration.

REFERENCES

1. J. Botha et al. *A Decision-Making Framework for the Evaluation of Climbing Lanes on Two-lane Two-way Rural Roads: Research Summary*. Report No. FHWA/CA/T0-80, University of California, California Department of Transportation, July 1980.
2. W. S. Homburger. *An Analysis of Safety at the Terminals of Climbing Lanes on Two-Lane Highways*. Report No. FHWA–CA–UCB–ITS–RR–86–3, University of California, Berkeley, May 1986.
3. C. J. Hoban and J. F. Morrall. *Overtaking Lane Practice in Canada and Australia*. Research Report No. ARR 144, Australian Road Research Board, 1985.
4. *Manual on Uniform Traffic Control Devices for Streets and Highways*, Federal Highway Administration, Washington, D.C., 1978 and subsequent revisions.
5. D. W. Harwood and C. J. Hoban. *Low-Cost Methods for Improving Traffic Operations on Two-Lane Highways—Informational Guide*. Report No. FHWA–IP–87–2, Federal Highway Administration, Washington, D.C., January 1987.
6. D. W. Harwood and A. D. St. John. *Passing Lanes and Other Operational Improvements on Two-lane Highways*. Report No. FHWA/RD–85/028, Federal Highway Administration, Washington, D.C., July 1984.
7. R. S. Hostetter and E. L. Seguin. The Effects of Sight Distance and Controlled Impedance on Passing Behavior. *Highway Research Board Bulletin 92*, HRB, National Research Council, Washington, D.C., 1965.
8. *Guide to Geometric Design of Rural Roads*. National Association of Australian State Road Authorities, Sydney, Australia, 1985.
9. *Design Procedure for Passing Lanes*. Ontario Ministry of Transportation and Communications, Downsview, Ontario, 1975.
10. A. Werner and J. F. Morrall. A Unified Traffic Flow Theory Model for Two-lane Rural Highways. *Transportation Forum*, Vol. 1, No. 3, 1985, pp. 79–87.
11. *Highway Capacity Manual*. Transportation Research Board

Special Report 209, TRB, National Research Council, Washington, D.C., 1985.
12. C. J. Messer. *NCHRP Project 3–28A Report: Two-Lane, Two-Way Rural Highway Capacity*. TRB, National Research Council, Washington, D.C., February 1983.
13. D. W. Harwood, A. D. St. John, and D. L. Warren. Operational and Safety Effectiveness of Passing Lanes on Two-Lane Highways. In *Transportation Research Record 1026*, TRB, National Research Council, Washington, D.C., 1985.
14. D. W. Harwood and A. D. St. John. *Operational Effectiveness of Passing Lanes on Two-lane Highways*. Report No. FHWA/RD – 86/196, Federal Highway Administration, Washington, D.C., 1986.
15. A. D. St. John and D. R. Kobett. *NCHRP Report 185: Grade Effects on Traffic Flow Stability and Capacity*. TRB, National Research Council, Washington, D.C., 1978.
16. E. A. Rinde. *Accident Rates vs. Shoulder Widths: Two-lane Roads, Two-lane Roads with Passing Lanes*. Report No. CA–DOT–TR–3147–1–77–01, California Department of Transportation, September 1977.
17. Roy Jorgensen and Associates. *Evaluation of Criteria for Safety Improvements on the Highway*. U.S. Department of Commerce, Bureau of Public Roads, 1966.
18. Martin Voorhees Associates. *Crawler Lane Study: An Economic Evaluation*. Department of the Environment, Great Britain, 1978.

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

Design Guide for Auxiliary Passing Lanes on Rural Two-Lane Highways

ALAN R. KAUB AND WILLIAM D. BERG

The objective of this research was to determine the conditions under which the construction of an auxiliary passing lane on two-lane rural highways is economically justified. A conflict-opportunity model was developed which estimates the number of potential passing conflicts with an opposing vehicle that a given traffic volume will generate. By assigning a cost-per-conflict opportunity and adjusting for the length of passing zones available, the passing-accident costs for a given roadway segment were estimated. Based on prior research, a deterministic reduction of this cost was used to estimate the savings that would result from an auxiliary passing lane. The TWOWAF model was then used to simulate delay and travel speeds for trucks and passenger vehicles for typical highway sections both without and with an auxiliary passing lane. Benefit-cost analysis was applied to determine the average daily traffic (ADT) levels at which an auxiliary passing lane would be economically justified as a function of section length, percent passing zones available, cost per conflict, construction cost, and discount rate.

Rural, two-lane highways constitute over 80 percent of the national highway system mileage but carry only approximately 35 percent of the total annual vehicle-miles of travel (*1*). Yet this system is responsible for over 48 percent of all fatal motor vehicle accidents and 30 percent of all injury accidents each year (*2*). On this rural two-lane system, the head-on collision is the second most common type of rural fatal accident, responsible for approximately 5,100 fatalities annually (*3*). One of the most common and complex rural, two-lane operational maneuvers, and one which has the potential to cause head-on or severe accidents is the passing maneuver. But it is also the passing maneuver which has the capability to substantially reduce rural, two-lane travel time and delay. Thus, on the rural two-lane system there exists a need to improve safety performance by reducing severe accidents while maintaining or improving traffic operational performance.

Prior research has suggested that one alternative for improving rural roadway passing performance is to design for passing opportunities such that the following driver will generally not become intolerant to delay by having to seek too diligently for an acceptable passing gap in opposing traffic (*4*). If passing opportunities were provided either by the absence of opposing traffic or by the placement of passing lanes at appropriate locations, much of the accident cost of the passing maneuver might be eliminated. On many rural highways this minimized probability of accident and minimized delay occur frequently where the volumes of traffic are light, and thus the probability of meeting an opposing vehicle while performing the passing maneuver is small. However, where the volume of traffic increases and the percent passing decreases such that delay and the probability of an accident become high, the construction of auxiliary passing lanes or various types of four-lane highways may be justified to provide for additional safe passing opportunities.

A. R. Kaub, Department of Civil Engineering and Mechanics, University of South Florida, Tampa, Fla. 33620. W. D. Berg, 2206 Engineering Building, University of Wisconsin-Madison, Madison, Wis. 53706.

Because of the expense associated with freeway construction, auxiliary passing lanes have begun to receive greater attention. Past research on the operational aspects of passing lanes by Franklin Research Institute (*5*) concluded that road widening, shoulder widening, and added lane construction would have marginal benefit-cost ratios less than 1.0. However, delay benefits were not included in the study because of insufficient data relating delay savings to improvements in operating speed. In another study, Harwood, St. John, and Warren (*6*) performed an operational evaluation of auxiliary passing lane (non-truck climbing) performance and concluded that passing lanes decrease the percentage of vehicles platooned, increase the rate of passing maneuvers, and have a small effect on mean travel speeds. A concurrent safety evaluation of passing lanes indicated that a passing lane can reduce the total accident rate by 38 percent with an approximate 29 percent reduction of fatal and injury accident rates.

Past research on the economic desirability of auxiliary lanes has concentrated on identifying those geometric and traffic conditions under which a truck climbing lane is warranted (*5, 7, 8*). Little consideration has been given to the need for passing lanes where truck climbing lanes are not warranted. The objective of the research reported herein was, therefore, to establish general guidelines for the construction of auxiliary passing lanes on two-lane rural highways based on an economic analysis of road-user benefits versus construction and maintenance costs (*9*). The scope of the research was confined to conditions found on those State Primary Highway System roads having pavement widths of 20 feet or greater. These roads represent approximately 78 percent of the entire State Primary Highway System (*10*).

PASSING CONFLICT MODEL

Models for accident occurrence are generally difficult to develop and calibrate due to the rare nature of an accident. However, in research by Stockton, Mounce, and Walton (*11*), a conflict analysis of the passing maneuver for low-volume, rural, two-lane roadways was performed using the Poisson distribution as the assumed empirical accident model. This analysis considered the probability of simultaneous arrivals of two vehicles of different speeds in one direction and the probability of opposition to the resultant passing maneuver from the opposing vehicle. This methodology was used to develop an expected number of annual conflicts. Although developed for low-volume, rural roadways, the above procedure was judged to offer a reasonable basis for estimating the number of passing conflict opportunities on the higher volume State Primary Highway System. It was further assumed that any passing conflict that occurs with an opposing vehicle can be assigned a proportional share of the total passing-accident costs on two-lane roadways.

In adapting the above passing conflict opportunity model to this research, it was assumed that

1. A conflict opportunity is defined as that maneuver of vehicle A (following), B (lead), or C (opposing), such that the driver of the following vehicle will have less than the AASHTO time exposed to traffic in the left lane (t_2) plus the clearance time (t_3), which is assumed to be a minimum of 16 seconds when the pass is completed (*12*).
2. Average speed is 55 mph, which is the average of all three speeds of the lead vehicle (50 mph), following vehicle (60 mph) and opposing vehicle (assumed 55 mph).
3. Passing sight distance is at least 1,000 feet, which is the minimum operational (distance considered acceptable for passing operations at 60 mph speeds). Where this sight distance is not available, it is assumed the pass will not be completed. This minimum sight distance conforms to the requirements of the Manual on Uniform Traffic Control Devices for Streets and Highways (MUTCD) for the marking of no-passing zones at 60 mph (*13*).
4. The probabilities of passing and arrival of opposition assume that all vehicles arrive during a 1-hour analysis period.
5. A passing situation occurs when a pair of vehicles arrive following a Poisson distribution within an assumed constant headway of 2 seconds or less.
6. The average directional distribution is assumed to be 50/50.

The probability of a passing conflict opportunity occurring can be calculated as follows for a highway with an assumed traffic volume of 250 vehicles per hour (vph) and a 50/50 directional distribution. From the Poisson distribution

$$P(X) = e^{-m}mx/x! \tag{1}$$

The probability that any two vehicles will be close enough for the following driver to desire to pass in any one hour is

$$P(h_t < 2 \text{ sec}) = 1 - P(0) - P(1) = 0.002302 \tag{2}$$

and the number of such passing opportunities per hour, per direction is

$$[P(h_t < 2 \text{ sec})] \times 1800 = 4.15 \tag{3}$$

In the passing maneuver, vehicle A will be exposed to traffic in the left lane for an assumed 16-second time interval. If an opposing vehicle appears within this 16-second interval, then by definition a conflict with the opposing vehicle is assumed to have occurred. The probability of arrival of the opposing vehicle in the 16-second interval is given by

$$P(1 \text{ or more}) = 1 - P(0) = 0.426 \tag{4}$$

The number of such conflicts is given by the product of the number of passing opportunities per hour and the probability of the arrival of an opposing vehicle during the passing maneuver, or 1.77 passing-conflict opportunities per hour, per direction.

The above conflict situation occurs over an 18-second interval (including the two-second headway for vehicle A) during which time vehicle A is traveling at 60 mph and traverses a distance of 0.3 miles. Placing the conflict rate on a vehicle-mile basis

$$\begin{aligned} \text{Conflict opportunities/veh-mi/hr} &= 1.77/0.3/250 \\ &= 0.0236 \end{aligned} \tag{5}$$

Thus, over a 1-mile segment under the above traffic conditions and assumptions, there will develop approximately 5.91 (1.77/0.3) conflict opportunities with opposing vehicles during the hour the 250 vph volume level exists, or each vehicle will experience 2.36 conflict opportunities in every 100 miles of travel regardless of the direction of travel. Utilizing the above methodology, probable conflict opportunities per mile, per hour were developed over two-way volume levels ranging from 0 to 1,800 vph as shown in table 1. It was further assumed that these values would be reduced in direct proportion to the amount of available passing sight distance on the highway segment. Thus, where 50 percent passing sight distance is available, the conflict opportunities would be reduced from 5.91 to 2.95 conflicts per mile, per hour. This assumption is a conservative approach because where passing is severely restricted, passing conflicts may actually increase to compensate for the reduced opportunity to pass.

PASSING-ACCIDENT COSTS

The presence of an auxiliary passing lane is intended to reduce the number of catastrophic passing accidents that occur due to the presence of an opposing vehicle in the passing maneuver. Such accidents normally involve high-speed head-on, or run-off-the-road accident types. To identify the value of aggregate passing-accident costs, and, ultimately, the pro rata individual conflict costs, it was necessary to quantify the cost of passing-related accidents caused by the presence of an opposing vehicle. However, the lack of detailed data on passing accidents required that an approximate accident cost framework be developed using summary statistics from available data bases. Using data published by the Federal Highway Administration and the National Safety Council (*10, 14*), the distribution of accidents per year by severity on two-lane rural highways was estimated as

Fatal Accidents: 7,469

Injury Accidents: 148,591

PDO Accidents: 1,578,800

Total: 1,734,839

In a study conducted by the Franklin Institute Research Laboratories (*5*), it was concluded that approximately 10 percent of the accidents on the two-lane system are passing related. Therefore, the total number of passing-related accidents was estimated as 10 percent of the above value, or 173,484 per

TABLE 1 NUMBER OF ANNUAL PASSING CONFLICTS IN THE PRESENCE OF AN OPPOSING VEHICLE

HOURLY VOLUME	AVERAGE VOLUME (VPH)	HOURLY MILES* (MILLIONS)	PASSING CONFLICT RATE (#/MI/HR)	ANNUAL PASSING CONFLICTS (MILLIONS)
0 - 100	50	2041.1	.0605	123.5
100 - 200	150	333.7	1.437	479.5
200 - 300	250	130.2	5.91	765.8
300 - 400	350	16.7	14.37	239.9
400 - 500	450	19.3	27.27	526.3
500 - 600	550	5.4	44.60	240.8
600 - 700	650	2.1	66.29	139.2
700 - 800	750	3.2	92.03	294.5
800 - 900	850	1.1	122.0	133.7
900 - 1000	950	2.4	154.3	370.3
1000 - 1100	1050	0	190.1	0
1100 - 1200	1150	0.4	228.9	91.6
1200 - 1300	1250	0.5	270.1	135.0
1300 - 1400	1350	0	313.5	0
1400 - 1500	1450	0	358.9	0
1500 - 1600	1550	0.3	406.2	121.9
1600 - 1700	1650	0	455.1	0
1700 - 1800	1750	0.1	505.5	50.5
			TOTAL	3712.5

*Ref. 10

year. This aggregate number of passing-related accidents is consistent with NSC statistics, which indicate that 3.2 percent of all rural accidents (5,188,500), or 166,032 rural passing accidents, are caused by improper overtaking (*14*). Other research has estimated that 3.5 percent of all passing accidents involve a fatality, and 42 percent of all non-fatal accidents involve personal injury (*10*).

Not all of the above-mentioned accidents can be attributed to the presence of opposing vehicles because passing accidents on two-way rural roads may also occur at intersections (driveways), railroad crossings, narrow bridges, roadside developments, or other such sites. The results of other research indicate that 20 percent, 58 percent, and three percent of all passing-related accidents occurred at "special situation" locations in the states of North Carolina, Texas, and Utah respectively (*15*). These particular states were selected to permit a representation of geographical distributions to approximate the effects of flat, rolling, and mountainous terrains. The remaining non-special situation passing-related accidents, which constitute 80 percent, 42 percent, and 97 percent, respectively, of all rural, two-lane passing accidents, were therefore assumed to be high-speed passing maneuvers that could result in catastrophic accidents.

For this research, it was assumed that these remaining non-special situation passing accidents are passing accidents that occur in the presence of an opposing vehicle such that the

TABLE 2 ESTIMATES OF THE COST OF EACH CONFLICTING PASS DUE TO THE PRESENCE OF AN OPPOSING VEHICLE

	Low	Average	High
Estimated Total Passing Accident Costs per Year (Millions)	$1616.1	$2424.2	$3232.3
Estimated Total Conflicts Per Year (Millions)	3712	3712	3712
Estimated Cost per Conflict (two-way)	$ 0.44	$ 0.66	$ 0.88

presence of the opposition vehicle contributed to the occurrence of the accident. Because the Utah data were reportedly inaccurate, due to underreporting, only the Texas and North Carolina data were used to establish boundary conditions for opposing vehicle-related passing accidents. Values of 40 percent, 60 percent, and 80 percent were therefore used as estimates of low, average, and high opposing-vehicle passing-related accidents. The actual value will depend upon the general terrain, roadway characteristics, and other factors appropriate to a particular state or region within a state.

Using 1978 data, passing-accident costs were assumed to be $300,700 for a fatal accident, $15,800 for a personal-injury accident, and $750 for a property-damage accident (*16*). Combining these values with the estimated opposing vehicle-related, passing-accident frequency data, total nationwide passing-accident costs were estimated to range from $1.6 to $3.2 billion per year. These accident costs were divided by the number of annual passing conflicts in the presence of an opposing vehicle for volumes ranging from 0 to 1,800 vph, as listed in table 1. The resulting estimated proportional cost associated with each passing conflict opportunity is shown in table 2. A comparison of the estimated passing conflict cost over various average daily traffic (ADT) volumes is presented in figure 1. It may be noted that the 2,000 to 5,000 and the 5,000 to 10,000 ADT ranges appear to be generating costs far in excess of other ADT levels. This, in general, suggests that a substantial number of miles of rural two-way, two-lane mileage in the 2,000 to 10,000 ADT range should receive consideration for upgrading to freeway standards or being provided with auxiliary passing lanes to reduce conflict and accident costs.

PASSING LANE EFFECTIVENESS

A study by the California DOT reported on the accident reduction potential attributable to the construction of passing lanes on two-lane rural highways (*17*). This study examined 19 projects that reconstructed over 48 miles of rural roadway from their original two-lane cross-section to a three-lane cross-section composed of the original roadway plus a third lane for passing. It was found that auxiliary passing lanes can be expected to reduce fatal accidents by approximately 60 percent, personal injury accidents by approximately 20 percent, and property damage accidents by approximately 20 percent. Applying these effectiveness measures to the previously estimated nationwide passing-accident data, the estimated annual dollar savings that could be expected if auxiliary passing lanes

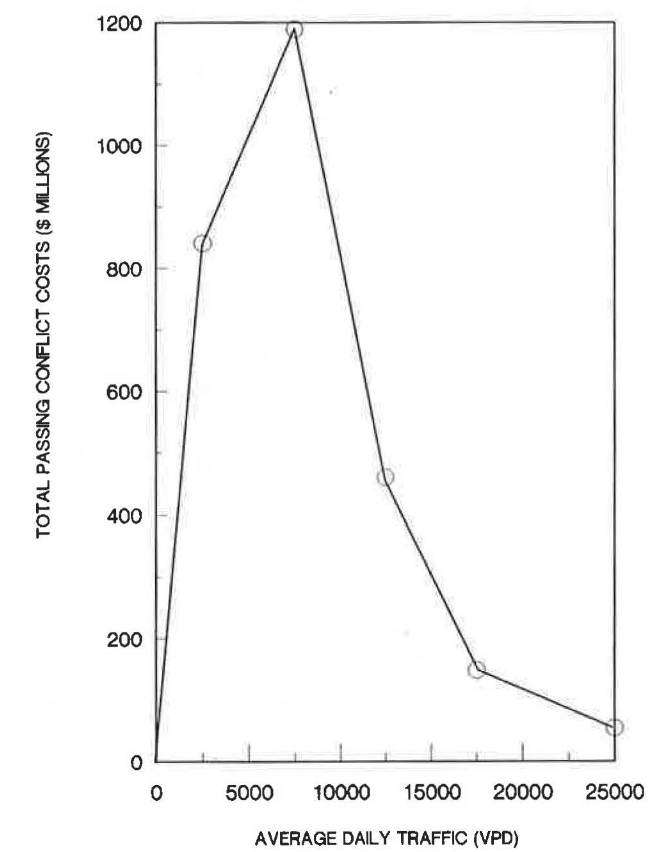


FIGURE 1 Comparison of passing conflict costs over various average daily traffic volumes.

TABLE 3 ESTIMATED PASSING ACCIDENT COST SAVINGS PRODUCED BY AUXILIARY PASSING LANES ANNUALLY

Accident Type	Cost Per Occurrence	Estimated Total Savings ($x10^6) Low	Average	High	% of Total
Fatality	$300,700	438.1	657.3	876.2	71
Injury	$ 15,800	171.3	256.9	324.6	28
PDO	$ 750	5.9	8.7	11.7	1
Total		615.3	922.9	1230.5	100

were constructed on all two-lane state primary highways are shown in table 3. A comparison of the total cost savings to the total passing-related accident costs indicated that the construction of auxiliary passing lanes may reduce by approximately 38 percent the total cost of passing-related accidents. Thus, for purposes of this research, it was assumed that an auxiliary passing lane would be 38 percent effective in reducing opposing vehicle-related passing accident costs.

To examine the benefits of reduced vehicle operating cost and travel time savings, it was necessary to simulate traffic flow conditions both with and without the presence of an auxiliary passing lane. The TWOWAF model was used for this purpose (*10, 19*). An experimental design was developed to generate simulation data that could be used to estimate the travel-time and vehicle-operating-cost savings associated with auxiliary passing lanes. Parameters that were assumed to be randomized and held constant include

1. Alignment. A flat, tangent alignment was assumed for the simulation modeling. The influence of horizontal and vertical curves was introduced by varying the percent of roadway with no-passing zones.
2. Sight Distance. A minimum of 1,000 feet was defined as available except where limited by no-passing zones.
3. Desired Speeds. A speed of 55 mph was assumed for autos with a standard deviation of 5.3 mph. However, because average truck speeds in the 10-year period preceding the imposition of the 55 mph speed limit were 6 mph below passenger speeds, it was assumed that trucks operate at speeds 7.5 mph below passenger car speeds (*20*). An examination of this speed reduction for trucks indicated that this 7.5 mph assumption reduced the speed of all vehicles approximately 3 mph and caused an increase in delay to all vehicles of approximately 10 percent compared to all vehicles operating at identical speeds. These overall reductions were judged to be consistent with the general effect of trucks on rural two-lane roadways. The assumed standard deviation for trucks speeds was also 5.3 mph.
4. Directional Distribution. For the purpose of developing average speed and delay models, A 50/50 split was assumed as the most common directional distribution on two-lane rural roads.
5. Traffic Composition. A traffic stream composed by 10 percent trucks was assumed.

Independent variables used in the simulation modeling were ADT volume, percent of the highway with permitted passing, and length of highway section being considered for auxiliary passing-lane treatment. ADT was varied from 2,000 to 9,500 vehicles per day. The percent passing was varied from zero to 100 percent with no-passing zones introduced in 528-foot segments. Section length was defined in terms of a replicated standard passing lane module consisting of one passing lane in each direction within a two-mile module, and varied from two miles to ten miles in total length. The selection of these lengths corresponds to the California study, which recommended alternating the direction of the passing lane each mile (*17*). Based on this recommendation, the assumed passing lane plan view is shown in figure 2.

The full experimental design would have required 880 cells to be tested. To reduce the computational requirements, the statistical technique of response surface methodology was applied (*22*). The TWOWAF simulation model was then used to develop the speed and delay values for both passenger cars and trucks. For the without-passing-lane configuration, data were generated for each flow direction and then averaged. Because the passing lane configuration could not be explicitly simulated by the TWOWAF model, an auxiliary passing lane was approximated by removing traffic volumes from the opposite direction, thus permitting passing only at specified one-mile intervals in one direction.

Stepwise regression analysis was used to develop the speed and delay relationships from the TWOWAF simulation data. The resulting delay models are listed below, where X_1 = one way volume (100 to 580 vph range), X_2 = section length (10,560 to 52,800 ft range), and X_3 = percent passing (0 to 100 percent range).

1. Without auxiliary passing lane:

Average passenger car delay (sec/mi)

$$= -0.475 + 0.020X_1 + 0.000139X_2 - 0.020X_3 \quad (6)$$

This model provided an R^2 of 96 percent with normal plots of residuals.

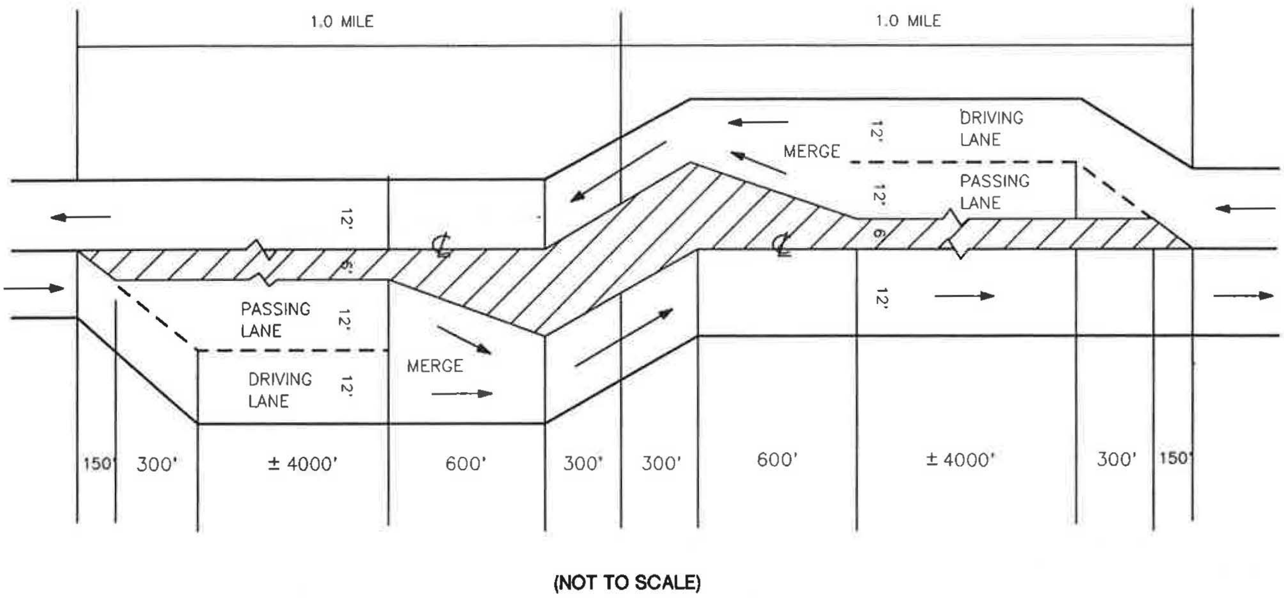


FIGURE 2 Typical passing lane horizontal alignment.

Average truck delay (sec/mi)

$$= -1.82 + 0.0095X_1 + 0.0001X_2 - 0.0078X_3 \quad (7)$$

This model provided an R^2 of 89 percent with normal plots of residuals.

2. With auxiliary passing lane:

Average passenger car delay (sec/mi)

$$= 0.250 + 0.017X_1 \quad (8)$$

This model provided an R^2 of 88 percent with normal plots of residuals.

Average truck delay (sec/mi)

$$= 0.0038 + 0.0083X_1 + 0.000029X_2 \quad (9)$$

This model provided an R^2 of 63 percent with normal plots of residuals.

Examination of the above delay models indicates that the traffic volume, percent passing, and section lengths are all significant variables for a two-lane roadway. However, with an auxiliary passing lane in place, delay is primarily dependent upon the traffic volume. These delay relationships may be expected, since delay should be a function of all three independent variables when an auxiliary lane does not exist. However, with the addition of an auxiliary passing lane, the effect of percent passing becomes insignificant because passing is normalized at 50 percent.

Vehicle operating costs vary as a function of travel speed and longitudinal grade. Because longitudinal grade was constrained to 0 percent, the only parameter assumed to affect running cost was the speed of the various vehicles with and without the presence of an auxiliary passing lane. Regression analysis was again used to develop the following speed models from the TWOWAF simulation data. The independent variables are as defined above.

1. Without auxiliary passing lane:

Average passenger car speed (ft/sec)

$$= 79.8 - 0.0189X_1 - 0.00013X_2 + 0.018X_3 \quad (10)$$

This model provided an R^2 of 96 percent with normal plots of residuals.

Average truck speed (ft/sec)

$$= 70.3 - 0.00798X_1 - 0.000088X_2 + 0.006X_3 \quad (11)$$

This model provided an R^2 of 96 percent with normal plots of residuals.

2. With auxiliary passing lane:

Average passenger car speed (ft/sec)

$$= 79.1 - 0.0174X_1 \quad (12)$$

This model provided an R^2 of 87 percent with normal plots of residuals.

Average truck speed (ft/sec) $= 68.3 - 0.0077X_1$ (13)

This model provided an R^2 of 66 percent with normal plots of residuals.

An examination of the speed models indicates that volume, percent passing, and section length are significant variables in the case of a two-lane roadway, while traffic volume is the only significant variable when an auxiliary passing lane is added. The models were used in conjunction with 1977 running-cost data (*22*) to estimate vehicle operating costs.

PASSING-LANE COSTS

To determine typical passing lane quantities and construction cost, it was assumed that most passing lanes would require some minor earthwork, 6 inches of aggregate base course, and 6 inches of asphalt surface course for the addition to the existing two lanes, and 1.5 inches of asphalt resurface over the entire length of the passing lane project. With this esti-

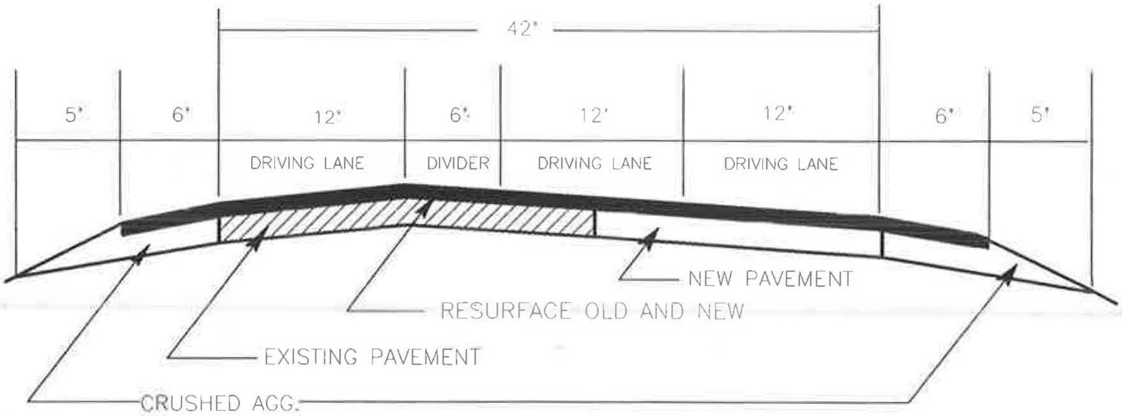


FIGURE 3 Typical APL cross section.

mate, figure 3 presents a typical cross-section of an auxiliary passing lane added to the outside of an existing two-lane roadway. It should be noted that the passing lane will vary from one side of the centerline to the other after each mile (thus the centerline location remains constant), and that, assuming a 42-foot surface width and 12-foot lanes, a six-foot median exists between opposing lanes. Using 1978 cost data, the initial cost of the typical auxiliary passing lane was estimated to range from \$250,000 to \$400,000 per mile. Maintenance cost savings attributable to the construction of an auxiliary passing lane plus overlay surface on the existing pavement was estimated at \$2,000 per mile, per year. The salvage value at the end of an assumed 20-year service life was estimated at \$35,000 per mile, which consists of the cost of right-of-way and one-half the cost of earthwork from the original estimate as suggested by AASHTO (*22*).

ECONOMIC ANALYSIS

The final task of the research was to incorporate the conflict, speed, delay, and cost relationship in an economic analysis model that would reveal the relative attractiveness of an auxiliary passing lane as traffic volumes vary over peak, off-peak, weekday, weekend, and monthly levels for highway sections of a given length and percent passing. By subtracting annual without-passing lane road-user costs from the with-passing lane user costs, an estimate of the total benefits of an auxiliary passing lane were determined. These benefits were then compared to the cost to construct and maintain an auxiliary passing lane after all costs and benefits were discounted to net present value. All cost data were adjusted to reflect 1978 conditions. The methodology used corresponds to that outlined in the 1977 AASHTO guidelines on economic analysis (*22*).

The results of the benefit-cost analyses were used to develop a break-even model that used two discount rates (four and eight percent), two construction costs (\$250,000 and \$400,000 per mile), and three conflict costs (\$0.22, \$0.33, and \$0.44 per conflict). Regression analysis was used to develop a break-even model which, for a given set of conditions, would indicate the minimum ADT at which an auxiliary passing lane would be economically justified. This would be that ADT associated with a benefit-cost ratio of 1.0. The resulting model is expressed as

$$\mathrm{ADT} = \exp[(17.0 - 0.369X_1 - 0.386 \ln X_2 + 0.138X_3 - 1.84X_4 + 0.00232X_5)/1.82] \quad (14)$$

where:

X_1 = section length
X_2 = length of roadway with permitted passing (%),
X_3 = discount rate (%),
X_4 = conflict cost (\$), and
X_5 = construction cost (\$1,000's).

An examination of the structure of the break-even model indicates that as the section length (number of replicated passing lanes constructed) increases, the ADT required to economically justify construction of the auxiliary passing lane section decreases, as it does when the percent passing and conflict cost are increased. However, when the discount rate or the cost of construction increases, the ADT at which the auxiliary passing lane is justified increases. Both of these observations conform to general expectations because more passing lanes (length), high percent passing available on the old road, and higher conflict costs should lower the ADT required to economically justify an auxiliary passing lane.

To simplify use of the break-even model, a nomograph was developed and is presented in figure 4. The nomograph is based on a 4 percent discount rate which has been recommended for safety projects (*16*) and includes values for passenger and truck delay costs (\$3.50 and \$10.00 per hour, respectively). To use the nomograph:

1. Estimate the per-mile construction cost of the auxiliary passing lane for the site as well as the cost of conflicts for the region or state. These estimates may be updated to current year dollar costs if it is assumed that any cost increases since 1978 are constant over all costs and all benefits. However, a better approach is to reduce current cost to 1978 cost levels.
2. Connect these estimated values to turn line 1.
3. Determine the extent to which passing is permitted on the existing road by comparing the directionally averaged length of no-passing zones to the total roadway length.
4. Connect turn line 1 and the percent passing to turn line 2.
5. Determine the length of roadway section that is to receive

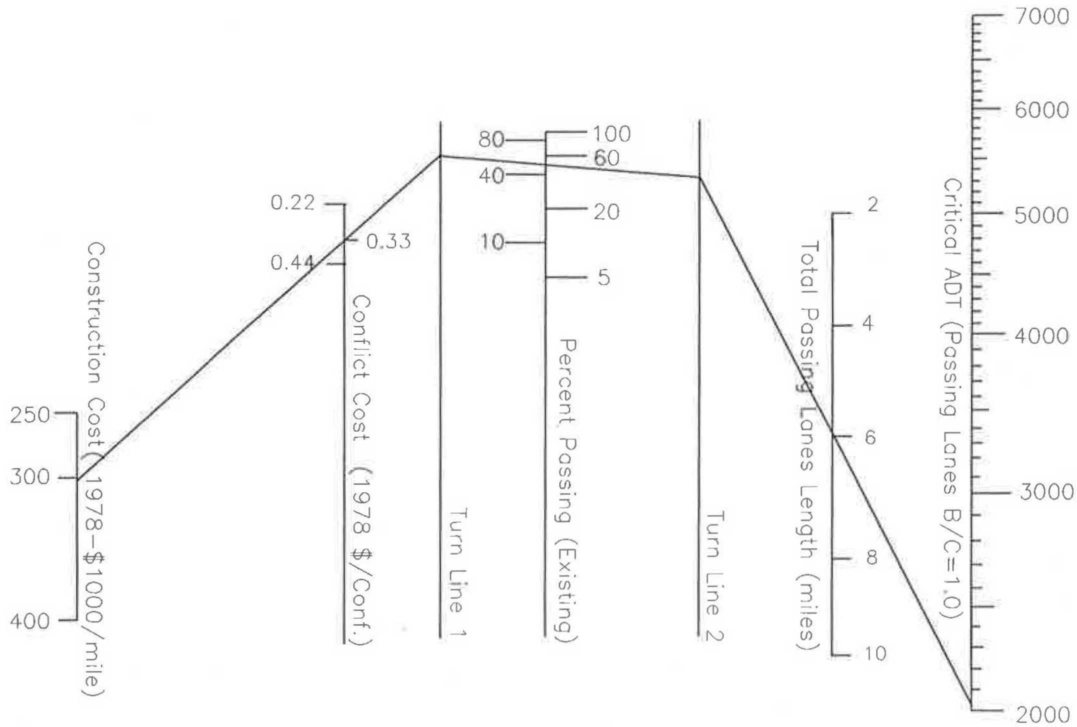


FIGURE 4 Economic analysis nomograph.

new passing lane construction, and connect the point on turn line 2 to the length to establish the ADT that must be exceeded to economically justify construction of the auxiliary passing lane. Conversely, the existing ADT at the site may be connected to the point on turn line 2 to determine the length of section for which the construction of auxiliary passing lanes is justified.

CONCLUSIONS

The auxiliary passing lane benefit-cost model and the nomograph for the critical ADT are based on a number of assumptions that constrain their general applicability.

The conflict-probability model estimates the number of passing conflicts that will occur on a two-lane roadway. The assumption of linear reduction to conflicts as the percent passing is reduced should be considered a limitation because, for some sites, a reduction in the percent passing may in fact stimulate the presence of conflict rather than reduce conflicts. However, since no research exists regarding an increase in accidents or conflicts as the percent passing is varied, the assumption of direct linearity appears reasonable. When figuring the cost per conflict, the relationship of intersection (special situation) passing accidents to all passing accidents was a determinant to the use of low, average, or high cost-per-conflict values, depending on the number of intersection-related passing accidents compared with all passing accidents. Care needs to be exercised in selecting an appropriate value for any case study application of the design warrant.

Similarly, with regard to the cost per conflict and the benefit to be derived from the construction of an auxiliary passing lane, the assumption of the accident reduction value of 38 percent of the original condition was based solely on California data. With further study of the safety benefits of other auxiliary passing lanes, this estimate of accident reduction potential may also vary, and may be increased to reflect the passing lane safety savings due to reduced passing accidents at special situation sites such as intersections and driveways.

The two-way traffic simulation model (TWOWAF) used to estimate speed and delay was capable of modeling a passing lane within the test section length only by eliminating traffic in the opposite direction. Thus, for a 6-mile segment, passing lanes were artificially introduced into alternating 1-mile lengths (1 mile in each direction) for the total 6-mile length. This was accomplished by restricting traffic flow in the opposing direction and permitting passing only at 1-mile intervals where passing is permitted. Future research should use a newer version of TWOWAF, which contains a passing-lane model capable of placing a specific size passing lane anywhere, and in either or both directions within the test section, and then developing several other general warrants where only one such lane, and not successive passing lanes, are used over varying length test sections.

Further limitations of the break-even model arise from the use of many assumed average values that were input to the TWOWAF traffic simulation model to generate travel speeds and delays for passenger and truck vehicles. Some of these parameters include vehicle composition, desired travel speeds and standard deviations of speed, available passing-sight distance and passing zone locations, as well as an assumed tangent roadway with a flat terrain, which inhibited truck speeds to 7.5 mph below passenger vehicle speeds. While it was necessary to normalize these and other roadway characteristics due to financial limitations placed on this research, a major revision of one or more of these assumed average conditions might cause the break-even model to overestimate or underestimate benefit-cost ratios and critical ADTs.

In summary, the model developed in this research was designed to assist engineers in evaluating the need for auxiliary passing lanes on two-lane highways. Where the critical ADT is determined to be substantially larger or substantially smaller than the ADT that exists at a site, many of the above limitations are expected to have only minor impact and may not affect the benefit-cost ratio or the critical ADT significantly. Where the critical ADT is reasonably close to the ADT that exists at the site in question, a detailed economic analysis should be undertaken using site-specific, TWOWAF-generated speed and delay data. A microcomputer program is available to provide detailed economic analysis of specific sites with specific input parameters.

REFERENCES

1. U.S. Department of Transportation, *Highway Statistics*, 1984.
2. U.S. Department of Transportation, *Fatal & Injury Accident Rates*, 1984.
3. U.S. Department of Transportation, *Fatal Accident Reporting System*, 1984.
4. R. S. Hostetter and E. L. Sequin. The Effects of Sight Distance and Controlled Impedance Distance on Passing Behavior. In *Highway Research Record 292*, HRB, National Research Council, Washington, D.C., 1969.
5. M. S. Janoff and A. Cassel. *Identification and Evaluation of Remedial Aid Systems for Passing Maneuvers on Two-Lane Rural Roads*, Volumes II, III and V. The Franklin Institute Research Laboratories, Washington, D.C., 1970.
6. Hardwood, St. John, and Warren. Operational and Safety-effectiveness of Passing Lanes on Two-lane Highways. In *Transportation Research Record 1026*, TRB, National Research Council, Washington, D.C., 1985.
7. E. G. Evans and T. B. Treadway. Economic Analysis of Truck Climbing Lanes on Two-lane Highways. In *Highway Research Record 245*, HRB, National Research Council, Washington, D.C., 1968.
8. D. G: McCallum. *The Economic Evaluation of Crawler Lanes*. Geometric Road Design Standards, Organization for Economic Cooperation and Development, Paris, 1977.
9. A. R. Kaub. A Design Warrant for Auxiliary Passing Lanes on Rural Two-lane Highways. Ph.D. thesis, University of Wisconsin-Madison, 1987.
10. Weaver, D. Graeme, and D. L. Woods. *Passing and No Passing Zones; Signs, Markings and Warrants*. Report FHWA-RD-79-5, Federal Highway Administration, Washington, D.C., September 1978.
11. W. R. Stockton, J. M. Mounce, and N. E. Walton, Guidelines for the Application of Selected Signs and Markings to Low Volume Rural Roads. In *Transportation Research Record 597*, TRB, National Research Council, Washington, D.C., 1976.
12. *Policy on Geometric Design of Highways and Streets*, AASHTO, 1984.
13. *Manual of Uniform Traffic Control Devices*, U.S. Department of Transportation, Washington, D.C., 1978.
14. *Accident Facts of 1978*. National Safety Council, Washington, D.C., 1979.
15. G. D. Weaver and D. L. Woods. *No Passing Zone Treatments for Special Geometrics and Traffic Operational Situations*. Report FHWA-RD-81-093, Federal Highway Administration, Washington, D.C., September 1981.
16. W. F. McFarland, et al. *Assessment of Techniques for Cost Effectiveness of Highway Accident Countermeasures*. Report FHWA-RD-79-53, Federal Highway Administration, Washington, D.C., January 1979.
17. E. A. Rinde. *Accident Rates vs. Shoulder Widths*. CALTRANS Report #Ca-DOT-TR-3147-1-77-01, 1977.
18. R. Goldblatt and E. Lieberman. *Review of Existing Two-Lane, Two-Way Rural Road Computer Simulation Models*. NCHRP Project 3-28-A, unpublished report, TRB, National Research Council, Washington, D.C., 1981.
19. R. Goldblatt and E. B. Lieberman. *Calibration and Validation of TWOWAF, Two-Lane, Two-Way Rural Road Computer Simulation Model*. NCHRP Project 3-28A, unpublished paper. TRB, National Research Council, Washington, D.C., December 1981.
20. *Highway Statistics, Summary to 1985*. U.S. Department of Transportation, Washington, D.C., 1986.
21. G. E. P. Box, W. G. Hunter, and J. S. Hunter. *Statistics for Experimenters*, J. Wiley & Sons, 1978.
22. *A Manual on User Benefit Analysis of Highway and Bus Transit Improvements*. AASHTO, Washington, D.C., 1977.

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

Uniform Delay Approach to Warrants for Climbing Lanes

K. M. Wolhuter and A. Polus

Current warrants for climbing lanes are discussed, and the consequences of their usage are explored. Data were obtained and analyzed to derive relationships among flow, gradient, and speed on South African roads. These relationships were used to calibrate TRARR, a simulation program developed by the Australian Road Research Board, and this, in turn, was used to establish relationships between delay and flow for various gradients. Actual delay is offered as an alternative warrant, it being pointed out that the Highway Capacity Manual offers delay as a criterion of Level of Service. The paper postulates that delay suffered would not be a function of the gradient on which it occurs if the climbing lane were the subject of economic analysis. Various isochronistic warrants are offered for consideration with the consequences of adoption of this approach being pointed out.

Most vehicles can maintain relatively high speeds on level terrain; consequently, the flow on these grades is characterized by low speed differentials and minimal turbulence, and high flow levels may occur. As soon as a gradient of any consequence is encountered, (in excess of about 3 percent) the situation changes dramatically. Speed differentials increase, leading to an increase in platooning of vehicles and reduction in the level of service. This phenomenon has long been recognized, and auxiliary lanes (variously known as climbing lanes, crawler lanes, truck lanes and, confusingly, passing lanes) have been provided to overcome this problem. In this paper, reference will be made throughout to climbing lanes. Passing lanes are generally found on flat gradients and are intended to increase the overall capacity of a road above that of a normal two-lane road, whereas climbing lanes, found on gradients, serve to match the capacity of the grade to that of the flatter sections of the road and eliminate excessive delay due to the low speeds of trucks.

The first problem confronting the designer in the provision of climbing lanes along a route is to determine at which points along the route climbing lanes can be installed to best advantage. This problem is invariably resolved by the use of warrants. Warrants may be described as surrogates for economic analysis and are often based on fairly arbitrary but easily measured parameters. A general economic analysis procedure is proposed by the British Department of Transport (*1*). Other than this, economic analysis has seldom, as far as can be established, been seriously attempted by practitioners.

The form and value of these warrants are legion. Warrants can be subdivided into five broad groups: truck speed reduction, speed differential between trucks and passenger cars, either of the above in association with a traffic volume, and, lastly, reduction in level of service. Reference to the literature reveals that, in each group, a range of values is encountered.

AASHTO (*2*) recommends the inclusion of climbing lanes where the critical length of grade, that distance which causes a reduction of 10 mph in the speed of a loaded truck, is exceeded. AASHO (*3*) used a truck speed reduction of 15 mph, whereas Glennon and Joyner (*4*) recommend the criterion of 10 mph, quoting increased accident risk in support of their contention. Polus et al. (*5*) proposed a truck speed reduction of 12 mph (20 km/h) as the speed warrant for climbing lanes, using the truck speed/gradient relationship quoted in the 1965 edition of "A Policy on Geometric Design of Rural Highways," and this suggests that the entry speed to a gradient be accepted as 40 mph (64 km/h). South Africa (*6*) also uses a truck speed reduction of 20 km/h, but assumes a truck speed of 80 km/h on level grades. Canada (*7*) considers that a truck speed reduction of 15 km/h warrants a climbing lane but measures the speed reduction from the 85th percentile or running speed. It is not clear whether this is the 85th percentile speed of trucks or of the whole traffic stream. Botswana (*8*) uses a truck speed reduction of 25 km/h.

The use of truck speed reduction as a warrant implies that passenger car speed is totally unaffected by gradient. Some authorities take cognizance of the fact that passenger car speed is, in fact, influenced by gradient and refer to the speed differential between cars and trucks as a warrant. For example, the *Transportation and Traffic Engineering Handbook* (*9*) refers to a speed differential of 10 mph between trucks and the mainline flow.

Very often, the speed reduction warrant is associated with a volume warrant. *The Highway Capacity Manual* (*10*) considers climbing lanes as an alleviating treatment when the following warrants are met:

- upgrade volumes exceed 200 vph,
- upgrade truck volumes exceed 20 vph, and
- a speed reduction of 10 mph or more is expected for the average truck.

Wolhuter also provides a volume warrant, based on the catch-up rate on various grades and with various percentages of trucks in the traffic stream, as illustrated in table 1.

The above warrants are intended for application to individual grades. The philosophy adopted by Australia (*11*), on the other hand, considers the need for climbing lanes based on examination of a considerable length of the road in question. The justification for climbing lanes is based on traffic

K. M. Wolhuter, National Institute for Transport and Road Research, Pretoria 0001, Republic of South Africa. A. Polus, Technion-Israel Institute of Technology, Haifa, Israel.

TABLE 1 VOLUME WARRANTS FOR CLIMBING LANES

Gradient (%)	Traffic volume in design hour	
	5 % trucks	10 % trucks
4	632	486
6	468	316
8	383	243
10	324	198

volume, the percentage of trucks in the traffic stream, and the availability of overtaking opportunities on the route. The speed reduction criterion is reduction to 40 km/h. Climbing lanes should span the full length of the grade, but partial climbing lanes may be considered when truck speeds fall below 40 km/h and a full lane is not justified because of low traffic volumes or high construction cost. On extreme grades, where truck speeds are reduced to 20 km/h or less, passing bays, typically less than 100 m long, can be considered when all the following conditions are met:

- long grades over 8 percent,
- high percentage of heavy vehicles,
- low overall traffic volumes, and
- high construction costs.

A separate class of warrants refers to level of service. Level of service is a descriptor of operational characteristics in a traffic stream, measured in terms of delay, speed, and ratio of volume to capacity. An important feature of this descriptor is that it is a representation of driver perception of the traffic environment and bears little or no relation to the cost of creating that environment or the cost of operating in it. The warrants suggested by the *Highway Capacity Manual* are—

- a reduction of two or more levels of service in moving from the approach segment to the grade and
- Level of Service E (LoS E) exists on the grade.

Polus et al. suggest that a climbing lane is warranted if the design hourly volume exceeds the specific grade service volume for a level of service one lower than that adopted for the design of a level section of the road.

Typically, the motivation given for selection of a particular warrant is based on qualitative arguments. An alternative approach to warrants for climbing lanes is presented in this paper.

BACKGROUND TO STUDY

Whereas estimation of the construction and maintenance costs of a climbing lane usually does not present any problem, derivation of the benefit accruing from this investment is more intractable. Economic analyses have often indicated that the benefit derived from time savings alone overshadows other benefits. However, to achieve a correct perspective, the overall benefit is briefly discussed below.

The benefit subdivides into benefit to the road user and benefit to the community as a whole as represented by the road authority. Benefit to the road user involves changes in—

- extent of delay, a time benefit;
- operating cost, an economic benefit;
- accident exposure, a safety benefit; and
- level of stress, a comfort benefit.

Benefit to the community derives from—

- higher levels of service, hence postponement of the obsolescence of a facility and
- reduced need to provide additional passing sight distance elsewhere, hence a potential reduction in construction cost.

Although sometimes described differently, it is clear that all these benefits have strong economic overtones, but, as stated above, the value accrued from time savings alone tends to overshadow the economic benefits derived by other means. For this reason, the attention of this paper is focused on the calculation of delay. The contention is that a specific climbing lane, warranted by time savings alone, could show a "profit" if the other factors were also taken into account; that is, such a warrant tends to be conservative.

Delay does not lend itself readily to direct measurement in the field, hence the use of alternative criteria, such as the percentage of time spent following. Seeing, however, that delay is simply the time added to a trip by travelling at a speed lower than desired, simulation offers a convenient technique for its determination.

MODUS OPERANDI EMPLOYED

Time mean speeds were measured at various sites, covering a range of gradients and under widely varying traffic flows, and classified according to vehicle type. These were used to calibrate a simulation model, TRARR, developed by the Australian Road Research Board, to local prevailing conditions.

Delay is a function of space mean speed. The simulation model was used to derive space mean speeds achieved by passenger cars on a range of gradients across a range of traffic flow. Gradients varied between 3.6% and 8.4%, and flow from 30 vehicles per hour (vph) to 1,500 vph. Space mean speeds achieved by vehicles traveling at headways of 10 s or longer are considered to be desired speeds, in other words, dictated by the hill)climbing capability of the individual vehicle or by the preference of the driver when the gradient is not sufficiently steep to govern vehicle performance. Increasing flow levels inevitably lead to a drop in space mean speed below that desired, and this reduction in speed is the basis for calculation of delay.

Data collection, calibration and calculation of delay are described in more detail in the following sections.

DATA COLLECTION

Data Acquisition System

The data were acquired for this analysis using the Traffic Engineering Logger (TEL) developed by the National Insti-

TABLE 2 DESCRIPTION OF VEHICLE CLASSES

Class		Length (m)	Description
Light	Short	<5.9	Passenger car
	Long	5.91 - 10.0	Passenger car with trailer
Heavy	Short	<10.0	Single-unit truck
	Medium	10.1 - 16.8	Tractor + semi-trailer
	Long	>16.8	Tractor + semi + trailer

tute for Transport and Road Research, South Africa. The TEL is a microprocessor-based system capable of collecting traffic data in one of three modes of operation. These are—

- roadside installation,
- vehicle mounted installation, and
- hand-held operation.

For this study, the roadside installation was employed, in which traffic data are collected from successive induction loops buried in the road surface.

The TEL differentiates among classes of vehicles on two bases. Cars, in spite of their lesser mass, show a greater disturbance of the magnetic field than do trucks because of their lower center of gravity. The length of vehicles is measured on the basis of their speed and corresponding time of occupation of the loops. Data acquired by this system include time of arrival (to nearest 0.1s), speed (to nearest 1 km/h), vehicle length (to nearest 0.1m), and class of vehicle, for each vehicle.

Table 2 lists the classes of vehicles among which the TEL can differentiate. For the purpose of this study, vehicles in the light category were grouped together as it was found that the percentage of cars in the traffic stream that were towing caravans (trailers) was very low. It was also found, as discussed later, that further aggregations could also be employed with advantage.

Data Acquired

Two sets of data were collected and subdivided as shown in table 3. The observation points were located sufficiently far along the grade for vehicle speeds to have stabilized to the gradient.

PRELIMINARY ANALYSIS OF DATA

Speed distributions for uninterrupted flow conditions were derived by considering only those speeds associated with headways of 10 s and longer and by aggregating them into 5 km/h intervals. Typical distributions for the various classes of vehicles are illustrated in figure 1 for the Ben Schoeman site.

The means and the standard deviations of the speed distributions were calculated for each of the sites as shown in table 4. The means are plotted as shown in figure 2, which illustrates what appears to be an inconsistency with expectations, as speeds at the four-lane sites are lower than those at the two-lane sites. These lower speeds are attributed principally to the fact that the speeds recorded on the freeway sections refer only to the slow lane. There is also a difference between the occurrence of headways longer than 10 s on two-lane roads and freeways, as illustrated in figure 3, and it is presumed that this would also account, even if only in part, for the difference. Further analysis established that the difference in speeds between two) and four-lane roads is statistically insignificant at the 5 percent confidence level. It is thus possible to ignore differences in cross-section in the study that follows.

It was also found that the difference in speeds between medium-heavy and long-heavy vehicles is statistically insignificant. Operators match the mass hauled to the capacity of the tractor, and the additional trailer serves to accommodate high-volume, low-mass loads. It is thus not surprising that there should be relatively little difference in performance

TABLE 3 DESCRIPTION OF DATA SETS

Site	Lanes	Gradient (%)	Distance along grade (m)	Sample size (veh)
Cornelia	2	3.62	1 600	9 232
Colenso	2	5.21	3 000	17 196
Long Tom	2	8.38	2 000	15 980
Rigel North	4	3.54	3 100	16 132
Rigel South	4	4.45	4 000	4 651
Ben Schoeman	4	4.97	1 400	20 945
Krugersdorp	4	6.44	1 800	21 254

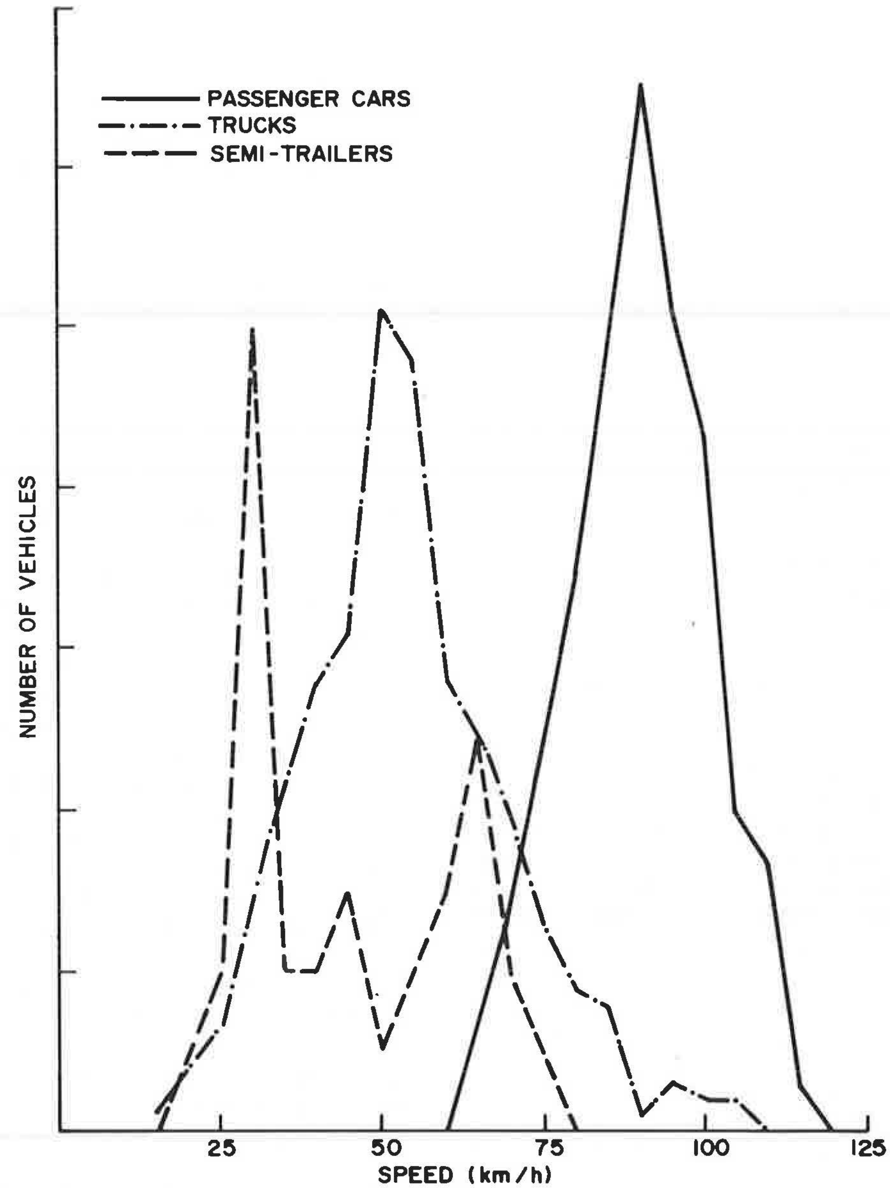


FIGURE 1 Distribution of speed on a gradient of 4.97% by class of vehicle.

between the two vehicle configurations. Consequently, only three classes of vehicles, passenger cars, single unit trucks, and semitrailers, are considered in this study. Average speeds derived for the semi-trailers are shown in table 5.

RELATIONSHIP BETWEEN GRADIENT AND SPEED

The speeds shown for the various vehicles on the observed gradients (as reflected in table 4) and the aggregated speeds for the semitrailers (as listed in table 5) represent what can be considered desired speeds for the purposes of this study. In short, they represent the limiting performance of the vehicle class in question or, alternatively, the speed preferences of the drivers where the performance of the vehicle does not dictate the selection of speed.

The following relationships between gradient and desired speed (or limit of vehicle performance) were derived by means of regression:

$$V_c = 123.32 - 6.99\,G\ (R^2 = 0.986)$$

$$V_t = 76.89 - 4.79\,G\ (R^2 = 0.994)$$

$$V_s = 69.13 - 5.33\,G\ (R^2 = 0.946)$$

where

V_c = passenger car speed (km/h)
V_t = truck speed (km/h)
V_s = semi-trailer speed (km/h)
G = gradient (%)

These relationships are plotted in figures 4 and 5 as dotted lines and represent actual performances by the various classes of vehicles as measured on South African roads. These are

TABLE 4 SPEED VERSUS GRADIENT FOR VARIOUS VEHICLE CLASSES

Num. of Lanes	Grade (%)	Vehicle class							
		Light		Heavy					
				Short		Medium		Long	
		Mean	S.D.	Mean	S.D.	Mean	S.D.	Mean	S.D.
2	3.62	100.6	15.6	60.6	15.6	51.8	14.3	49.8	15.4
2	5.21	89.2	16.0	52.3	18.4	37.6	13.8	34.6	12.9
2	8.38	64.5	12.9	37.7	11.8	32.4	15.9	19.0	10.2
4	3.54	95.7	14.3	60.2	16.1	52.1	16.8	50.9	16.8
4	4.45	92.5	14.4	55.1	14.4	47.4	16.2	48.2	15.9
4	4.97	86.7	15.8	52.2	16.7	40.4	15.3	42.3	16.8
4	6.44	78.2	15.1	44.9	17.3	32.7	12.3	26.4	9.3

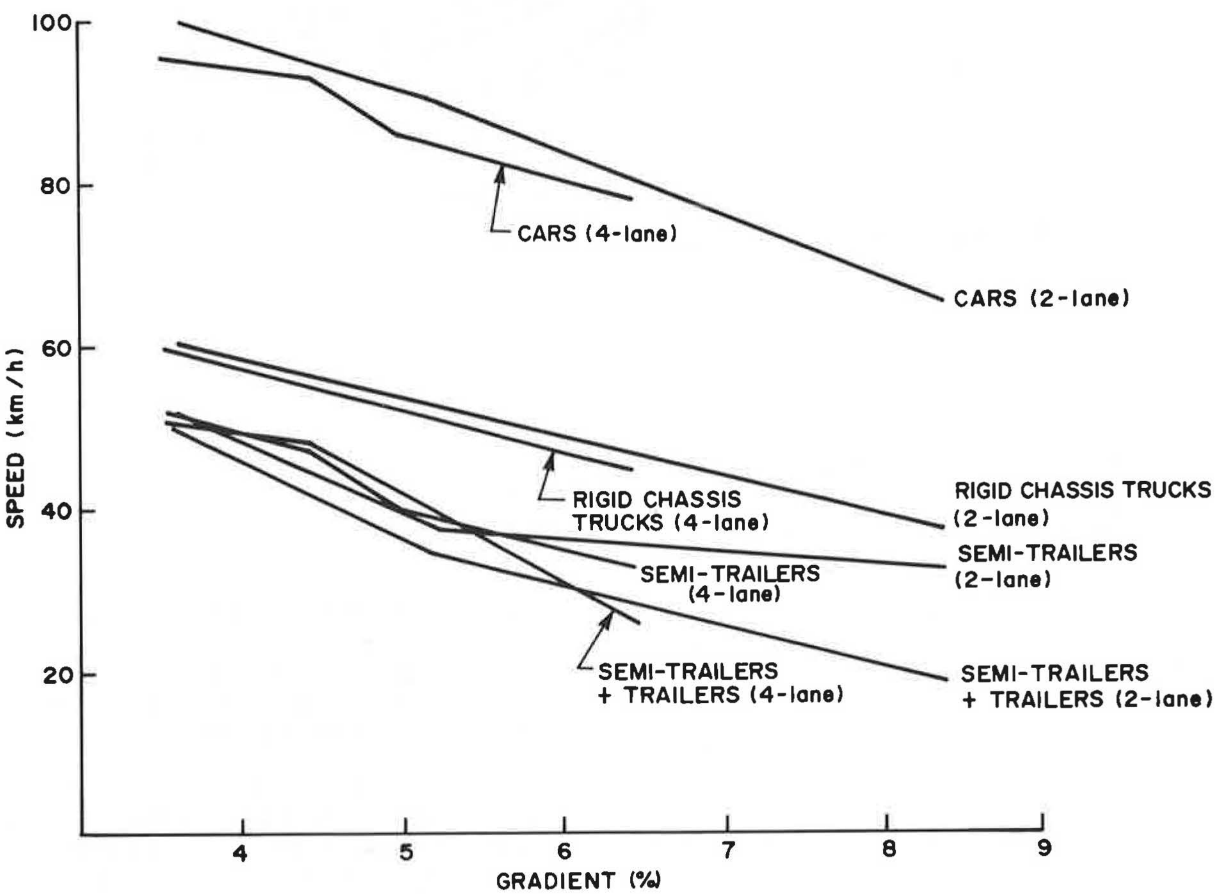


FIGURE 2 Observed mean speeds on gradients by class of vehicle.

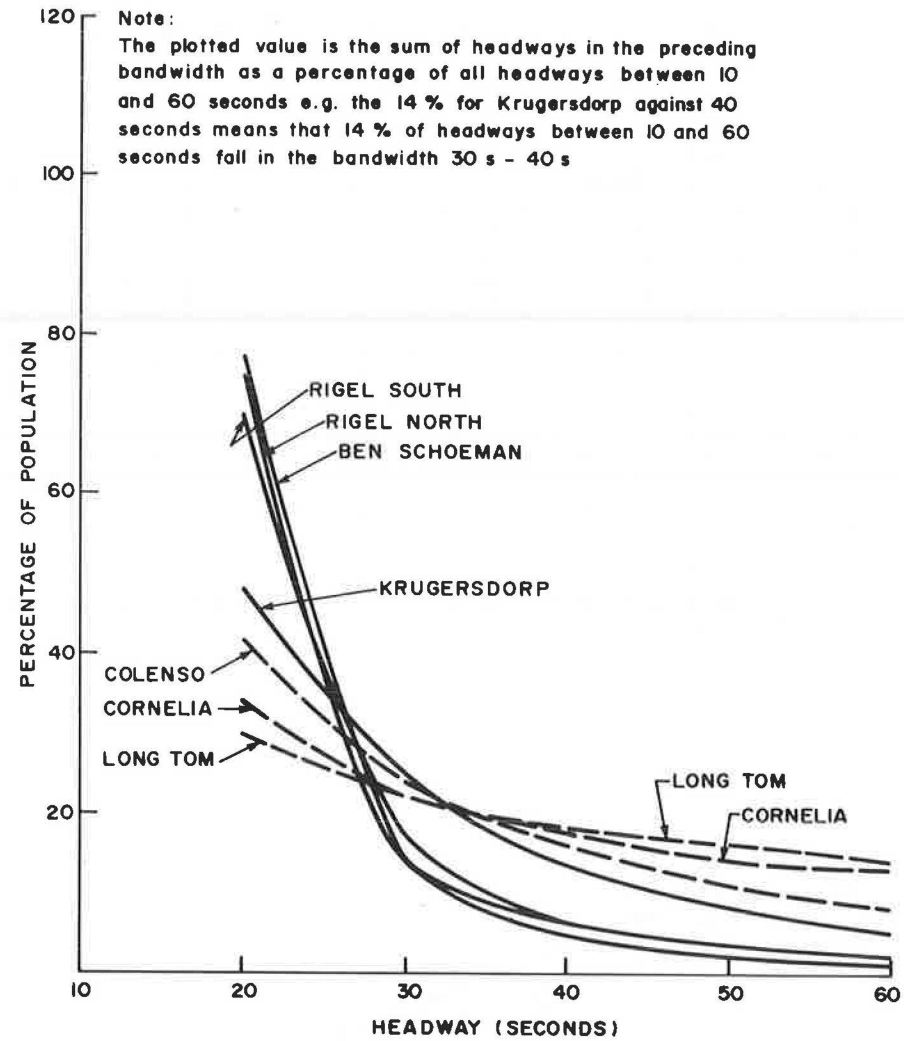


FIGURE 3 Distribution of observed headways greater than 20 s.

TABLE 5 AGGREGATED SPEED ON GRADIENTS FOR SEMITRAILERS

Gradient (%)	Speed (km/h)
3.54	52.60
3.62	51.44
4.45	47.49
4.97	40.83
5.21	36.66
6.44	31.93
8.38	28.00

speeds measured at a point, or spot speeds, whereas, for the purposes of study of delay, reference to speed will imply space mean speed.

RELATIONSHIP BETWEEN FLOW AND SPEED

The relationship between flow and speed on each grade was derived by a laborious process consisting of a number of computational steps, as described below:

1. The entire period of observation at each site, typically with a duration of 72 hours, was divided into successive 2-minute intervals, and the volume of vehicles in each class for each 2-minute interval was derived. A volume of between 1 and 50 vehicles is thus equivalent to a flow of between 30 and 1,500 vph.
2. The range of speeds, between 0 km/h intervals, and the

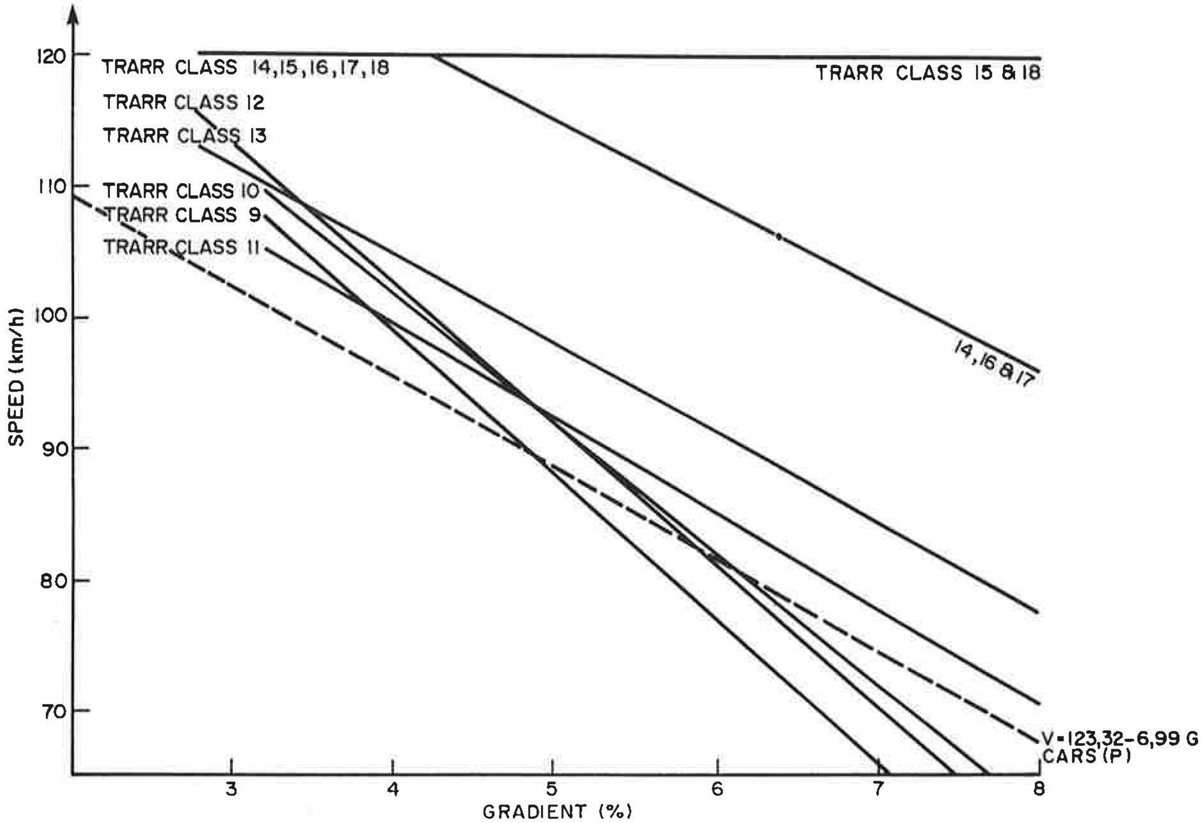


FIGURE 4 Comparison of observed mean speeds of passenger cars and TRARR Vehicle Classes 9 to 18.

number of vehicles of each class falling into each individual speed range calculated.

3. The histogram of speed so derived was then aggregated with the histograms of other two-minute intervals with the same flow.

4. Step 3 thus leads to the creation of a three-dimensional surface, with speed and flow as its base and number of vehicles as the vertical coordinate.

5. For convenience, and to apply a degree of smoothing, five successive flow levels were aggregated, representing steps of 150 vph between the various flow levels, or a total of ten points between zero and 1,500 vph.

6. Finally, the mean speed for each class of vehicle and for each aggregated flow level was calculated.

7. The process described above was repeated for each of the seven sites.

It was found that the relationship can be expressed as

$$V_{ca} = 128.38 - 6.89\,G - 0.008\,Q\ (R^2 = 0.87)$$

where

V_{ca} = actual speed for passenger cars (km/h)
G = gradient (%)
Q = flow along upgrade (vph)

Calibration of Model

The simulation model, TRARR (Traffic on Rural Roads), was obtained from the Australian Road Research Board and calibrated in two stages.

This model employs eighteen different classes of vehicles, and, as a first step, runs were carried out with each of the eighteen classes of vehicles on each of the seven gradients. The conditions of the runs were that the position of the observation points in the simulation corresponded with the points at which data were actually gathered on the various gradients, and that flows were selected to represent headways of 10 s or longer.

The desired speeds for each of the classes of vehicle on the various gradients as derived from the simulation were also regressed to secure relationships between speed and gradient, and these are shown as solid lines in figures 4 and 5. Visual inspection suggested that the best correspondence could be obtained by using—

- for cars TRARR Class 11
- for trucks TRARR Class 7
- semi-trailers TRARR Class 6

The next stage involved running traffic streams containing these classes of vehicles, in the percentages observed in the field, on the various gradients. The delays of particular interest in this study are those suffered by passenger vehicles. Further analysis was carried out to derive a relationship between average passenger car speed versus gradient and flow for the chosen classes of vehicles in the simulation model. The relationship found is—

$$V_{cs} = 131.660 - 6.538\,G - 0.017\,Q\ (R^2 = 0.95)$$

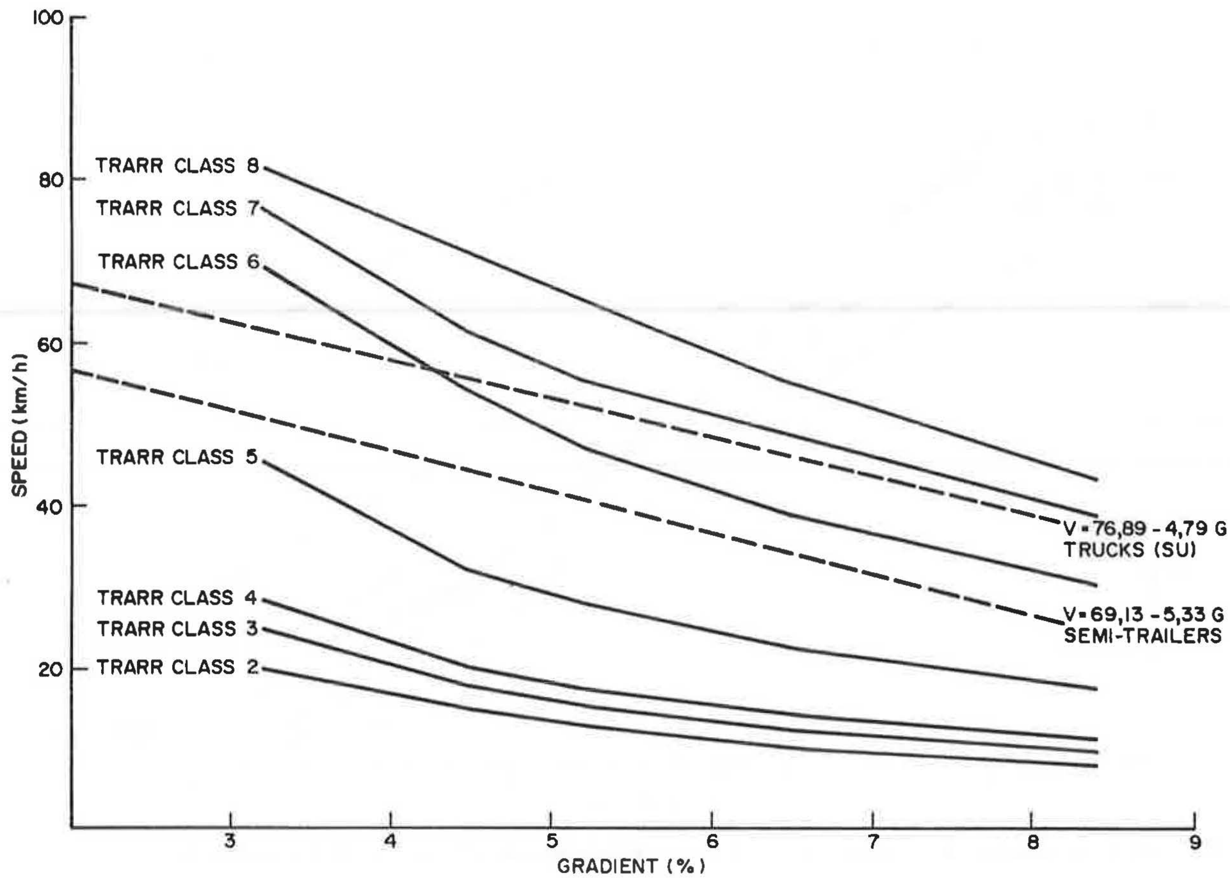


FIGURE 5 Comparison of observed mean speeds of trucks and semitrailers and TRARR Vehicle Classes 2 to 8.

where

V_{cs} = simulated speed for passenger cars (km/h)
G = gradient (%)
Q = flow along upgrade (vph)

This relationship compares reasonably well with that found for the field data.

DELAY

As suggested earlier, delay is that period of time added to a trip by a reduction of space mean speed to a value less than the desired. Space mean speed is calculated as the quotient of distance and mean journey time, and the speeds compared are those attained on the various grades at various flow levels against speeds attained on the same grades at very low flow levels.

Delay is thus calculated as:

$$T_d = 3600\ (1/V_a - 1/V_d)$$

where

T_d = delay/km/passenger car(s)
V_a = speed achieved by passenger cars at varying flow levels (km/h) = $131.660 - 6.538\ G - 0.017\ Q$
V_d = speed achieved by passenger cars at headways of 10 s or longer (km/h) = $131.660 - 6.538\ G$

G and Q have the same meaning as before.

Ultimately, the product of delay per passenger car and the passenger car flow provides the total delay per kilometer experienced per hour, assuming that the flow rate remains constant for the entire hour.

There could be fluctuations in flow within the hour, which would introduce an underestimation in the calculated delay. For example, a flow of 750 vph could either be absolutely uniform or a volume of 749 vehicles in 30 minutes followed by a vehicle with a headway of 30 minutes. Using the above relationships, the total delay to passenger cars (on a 5 percent gradient with an assumed 15 percent trucks in the traffic stream) would amount to 57.15 min/km in the case of the uniform flow; whereas, in the second case, the total delay would be 134.14 min/km. It is suggested that the likelihood of such an extreme fluctuation is remote. However, if a flow of 750 vph represents a flow of 600 vph for 30 minutes followed by a flow of 900 vph for a further 30 minutes, there is still an underestimation of delay, although the error reduces from 76.99 min/km (134.14 − 57.15) to 3.01 min/km, an error of 5.27 percent. Clearly, further research is required to establish typical ranges of variation and to introduce appropriate corrections into the calculation of delay.

Using the relationships derived above, delay, suffered per kilometer by an individual passenger car, was calculated for a range of gradients, 3 to 9 percent, and flows along the upgrade varying from zero to 1,500 vph. These are plotted as shown in figure 6.

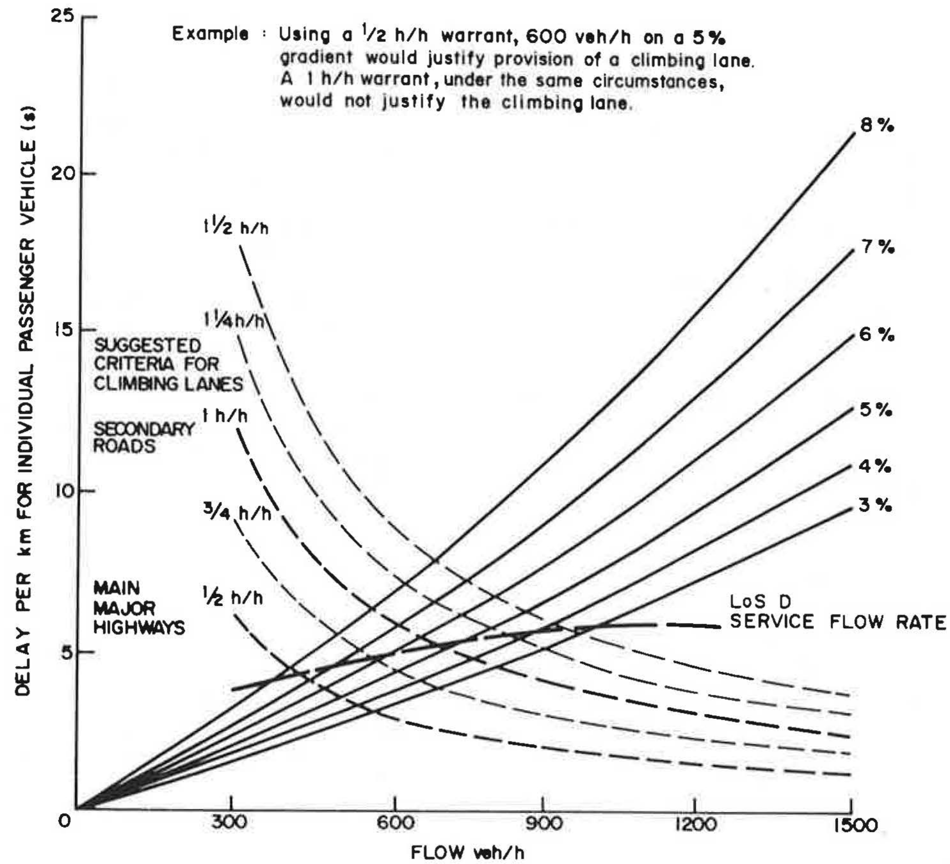


FIGURE 6 Relationship between flow, gradient, and delay.

DELAY CONSEQUENCES OF EXISTING WARRANTS

On level sections, rural roads typically operate in the range of LoS A or LoS B, except when close to urban areas. Seasonal fluctuations on recreational routes could produce the same result. *The Highway Capacity Manual* suggests, as one of the warrants for climbing lanes, a reduction by two levels of service, such as from LoS B to LoS D. The contention is that most drivers would be prepared to accept that operating conditions on a steep upgrade need not be comparable to those on level sections, suggesting that a reduction would be acceptable. However, a reduction through two levels, such as from LoS B to LoS E, would not be acceptable, and could thus lead to a reduction in safety being generated by impatience and ill-considered overtaking maneuvers. Such a reduction could also lead to a considerable increase in delay.

By way of illustration, a curve representing service flow rates for LoS D for various gradients is also shown on figure 6. The plotted values are based on the assumption of 15 percent trucks in the traffic stream and a grade 2.5-km (1.5-mi) long. This distance allows for a substantial length of gradient over which vehicle speeds are no longer influenced by preceding gradients.

It can be observed that the level of service warrant indicates a reasonably constant delay per vehicle regardless of the gradient. Because of the higher flows that can be accommodated on the flatter slopes before LoS D occurs, the overall delay to the traffic stream required to warrant a climbing lane is considerable.

AN ALTERNATIVE WARRANT FOR CLIMBING LANES

Also presented in figure 6 are five lines that represent isochronistic warrants for climbing lanes. These lines are based on the assumption that the total hourly delay for a given section, 1 km in this example, should remain constant regardless of the gradient. Thus, lines for ½, ¾, 1, 1¼, and 1½ hours of total delay per hour are presented. These are simply hyperbolae of the form:

$$W = Q * D * Pp / 3600$$

where

- W = constant, equal to selected warranting total delay = ½, ¾, 1, 1¼ or 1½ (h/h/km)
- Q = flow (vph)
- D = delay per individual passenger car(s)
- Pp = percentage passenger cars in stream

A climbing lane will be justified to the right of each line and not justified to its left.

The selection of a particular criterion (½, ¾, etc.) is left to the individual agency and ought to be determined beforehand, based on general and economic design policies. It is suggested, for example, that on major highways an agency may prefer a higher standard by opting for the ½ h/h criterion and on secondary roads accept the 1 h/h criterion.

A decision to adopt delay as a warrant in preference to those currently in use has consequences that may not be over-looked. The most obvious of these is that the flow at which a climbing lane is warranted shows a dramatic decrease on the flatter grades and a similar increase on steeper grades. The ¾ h/h warrant demonstrates a break-even point with the current LoS warrant at 600 pcph and a 5 percent gradient. Experience indicates that the majority of gradients on any route are likely to be less steep than 5 percent, and flows in excess of 600 pcph are not uncommon. It is therefore reasonable to expect that, if the values in the above example are adopted, delay would illustrate a need for more climbing lanes than current warrants would suggest. At low volumes there would be some reduction in the number of climbing lanes called for on steeper grades. Construction costs, in the more rugged terrain that these gradients imply, could be substantially reduced.

A further consequence of delay as a warrant is that levels of service on steeper gradients may decrease, theoretically to beyond capacity. There is thus a logical cut-off point, in terms of flow and gradient, beyond which delay becomes meaningless and, therefore, a level of service criterion may have to be employed. This cut-off point, as well as the exact criterion of total delay, is still to be established. The basic concept, however, of a diminishing level of service with increasing gradient is seen as matching driver expectations which anticipate worsening of conditions with increasing steepness of grades. This expectation can be used to advantage to provide climbing lanes where drivers do not expect unnecessary delay.

ACKNOWLEDGMENTS

This paper is presented with the permission of the Divisional Director, Division of Road and Transport Technology, Council for Scientific and Industrial Research, Pretoria.

REFERENCES

1. *Highway Link Design.* TD 9/81. U.K. Department of Transport, 1981.
2. *A Policy on Geometric Design of Highways and Streets.* AASHTO, Washington, D.C., 1984.
3. *A Policy on the Geometric Design of Rural Highways.* AASHO, Washington, D.C., 1965.
4. J. C. Glennon and C. A. Joyner. Re-evaluation of Truck Climbing Characteristics for use in Geometric Design. *Journal of Texas Transportation Institute*, 1969.
5. A. Polus, J. Craus, and I. Greenberg. Applying the Level-of-Service Concept to Climbing Lanes. In *Transportation Research Record 806*, TRB, National Research Council, Washington, D.C., 1981.
6. K. M. Wolhuter. *Geometric Design of Rural Roads.* Technical Recommendations for Highways 17. National Institute for Transport and Road Research, CSIR, Pretoria, South Africa, 1985.
7. *Geometric Design Standards for Canadian Roads and Streets.* Roads and Transportation Association of Canada, Ottawa, Ontario, 1976.
8. *Botswana Road Design Manual.* Botswana Ministry of Works, Gaberone, 1982.
9. *Transportation and Traffic Engineering Handbook.* Institute of Transportation Engineers; Prentice-Hall, N.J. 1976.
10. *Special Report 209: Highway Capacity Manual.* TRB, National Research Council, Washington, D.C., 1985.
11. *Guide to the Geometric Design of Rural Roads.* National Association of Australian State Road Authorities, Canberra, 1985.

Publication of this paper sponsored by Committee on Geometric Design.

Possible Design Procedure To Promote Design Consistency in Highway Geometric Design on Two-Lane Rural Roads

RUEDIGER LAMM, ELIAS M. CHOUEIRI, JOHN C. HAYWARD, AND ANAND PALURI

European design guidelines explicitly address horizontal design consistency for two-lane, rural roads in an attempt to promote smooth operating speed profiles and, in turn, safe operation. U.S. practice qualitatively advocates consistent alignment but provides little objective guidance to assure that consistency is achieved. This paper presents a procedure for measuring the consistency of horizontal design as defined by operating speed and accidents expected. Operating speeds and accident rates can be predicted for various lane widths based on degree of curve and posted recommended speeds, as derived from measurement of 261 sites in New York state. Guidelines for changes in operating speeds and acceptable accident rates for good, fair, and poor designs are suggested, and various nomographs are developed to evaluate roadway sections based on design parameters. In addition, an example application is provided to illustrate the case of fair design practices. It is concluded that such a procedure could readily be adapted by the design community in prescribing improvements to existing facilities or in fine tuning new highway design.

Abrupt changes in operating speed because of horizontal alignment are a leading cause of accidents on two-lane, rural roads, according to many experts (*1–6*). State and Federal agencies spend approximately 2 billion dollars annually to resurface, restore, and rehabilitate these roadways, exclusive of major reconstruction required to refine roadway geometrics (*7*). It seems that in an improvement program of this magnitude a convenient method for locating alignment inconsistencies, which may cause abrupt operating speed changes, would be beneficial. Such a mechanism would enable the engineering agency to provide cost-effective horizontal alignment modifications consistent with the resurfacing, restoration, and rehabilitation (RRR) program and thereby enhance traffic safety on two-lane, rural highways. An objective method of identifying hazardous elements that require abrupt operating speed changes would enable the agency to make geometric revisions at the same time that other deficiencies are being remedied.

REVIEW

An international review of existing design guidelines (*8–14*) has shown that European countries directly or indirectly address three design issues in their guidelines much more explicitly than U.S. agencies (*2, 15–19*). French, German, Swedish, and Swiss designers are provided with geometric criteria which direct them toward—

- achieving consistency in horizontal alignment,
- harmonizing design speed and operating speed, and
- providing adequate driving dynamic safety.

The objective of this research was to explore whether these guidelines could be adopted for U.S. practice in new design, major reconstruction, and, especially, RRR projects.

Prior studies by the authors (*15–21*) were relied on to develop the proposed methodology. Research that evaluated the impact of design parameters (degree of curve, length of curve, superelevation rate, lane width, shoulder width, sight distance, gradient, and posted recommended speed) and traffic volume on 261 two-lane, rural highway sections in New York state demonstrated that the most successful parameters in explaining the variability in operating speeds and accident rates were degree of curve and posted recommended speed limits. The relationship of operating speed and degree of curve is quantified by the following regression model (*15–18*) between operating speed and various design and traffic volume parameters:

$$V85 = 34.700 - 1.005DC + 2.081LW + 0.174SW + 0.0004AADT \qquad R^2 = 0.842 \qquad SEE = 2.814$$

where

- $V85$ = Estimate of operating speed, expressed by the 85th-percentile speed (mph) of passenger cars under free-flow conditions,
- DC = Degree of curve (deg./100 ft.) (range: 0° to 27° for investigations on operating speeds, and 1° to 27° for investigations on accident rates, since the accident situation on tangents is clearly affected by variables other than those on curves),
- LW = Lane width (ft.),
- SW = Shoulder width (ft.),
- $AADT$ = Average Annual Daily Traffic [range: 400 to 5,000 vehicles per day (vpd)],
- R^2 = Coefficient of determination, and
- SEE = Standard error of estimate (mph).

R. Lamm, and A. Paluri, Clarkson University, Potsdam, N.Y. 13676. E. M. Choueiri, N. Country Community College, Saranac Lake, N.Y. 12982. J. C. Hayward, Michael Baker Jr., Inc., Beaver, Pa. 15009.

The independent variables in the above equation were selected by the step-wise regression technique in the order: *DC*, *LW*, *AADT*, and *SW*. For instance, *DC* had the highest correlation with the dependent variable (*V*85); thus, it was the first variable included in the equation, and so forth.

Design parameters, sight distance, length of curve, and gradient (up to 5 percent) were not included in the regression model because the regression coefficients associated with these parameters were not significantly different from zero at the 95 percent level of confidence. Superelevation rate and posted recommended speed were withheld from the regression analysis because they are highly correlated with degree of curve.

However, in comparing the above equation with the following equation, which only includes the design parameter degree of curve, note from the coefficients of determination (R^2) that the influence of *LW*, *SW* and *AADT* in the above equation explains only an additional of about 5.5 percent of the variation in the expected operating speeds.

$$V85 = 58.656 - 1.135\,DC \qquad R^2 = 0.787 \qquad SEE = 3.259 \tag{1}$$

Note that this regression equation has an R^2 value of 0.787 and its standard error for estimating the observed operating speed is 3.259 mph. It is clear including the additional design and traffic volume parameters adds little to the predictive capability of the model.

Similar relationships were established between operating speeds and posted recommended speeds, and accident rates and degrees of curve; table 1 shows some of the results. Note that (1) the models are valid for road sections with grades up to 5 percent, and low and intermediate traffic volumes (between 400 and 5,000 vpd); and (2) a cross-validation of the models on a new sample of 61 rural, two-lane, curved sections determined that they can be used, with a marked degree of confidence, for prediction purposes. It is likely that grades over 5 percent and *AADT* volumes greater than 5,000 vpd will measurably influence operating speeds and accident rates on two-lane, rural highways; however, because of a lack of data (less than 20 percent of the two-lane, rural highway network in New York is made up of sections where grades are greater than 5 percent and traffic volume exceeds 5,000 vehicles per day), these effects were not analyzed.

TABLE 1 PREDICTIVE REGRESSION EQUATIONS OF OPERATING SPEEDS AND ACCIDENT RATES FOR DIFFERENT LANE WIDTHS (*16–18*)

All lanes

$V85 = 58.656 - 1.135DC;\ R^2 = 0.787$ (1)

$V85 = 25.314 + 0.554RS;\ R^2 = 0.719$ (2)

$ACCR = -0.880 + 1.410DC;\ R^2 = 0.434$ (3)

10-ft lanes

$V85 = 55.646 - 1.019DC;\ R^2 = 0.753$ (1a)

$V85 = 27.173 + 0.459RS;\ R^2 = 0.556$ (2a)

$ACCR = -1.023 + 1.513DC;\ R^2 = 0.300$ (3a)

11-ft lanes

$V85 = 58.310 - 1.052DC;\ R^2 = 0.746$ (1b)

$V85 = 29.190 + 0.479RS;\ R^2 = 0.744$ (2b)

$ACCR = -0.257 + 1.375DC;\ R^2 = 0.462$ (3b)

12-ft lanes

$V85 = 59.746 - 0.998DC;\ R^2 = 0.824$ (1c)

$V85 = 26.544 + 0.562RS;\ R^2 = 0.835$ (2c)

$ACCR = -0.546 + 1.075DC;\ R^2 = 0.726$ (3c)

where

V85 = Estimate of the operating speed, expressed by the 85th-percentile speed for passenger cars (mph),

DC = Degree of curve (degree/100 ft), range: 0° to 27°,

R^2 = Coefficient of determination,

ACCR = Estimate of accident rate for all vehicle types (acc./10^6 vehicle-miles), range: 1° to 27°,

RS = Posted recommended speed in the curve or curved section (mph).

TABLE 2 T-TEST RESULTS OF ACCIDENT RATES FOR DIFFERENT DEGREE OF CURVE CLASSES (*16–18*)

Degree of Curve Classes	Mean Accident Rate	t calc.		t crit.	Significance	Remarks
tangent (0°)	1.87					Considered as
		4.00	>	1.96	Yes	
1° - 5°	3.66					---- Good Design
		7.03	>	1.96	Yes	
> 5° - 10°	8.05					---- Fair Design
		6.06	>	1.99	Yes	
> 10° - 15°	17.55					---- Poor Design
		3.44	>	1.99	Yes	
> 15° - 26.9°	26.41					---- Poor Design

In addition, the research studies (*15–21*) determined that

1. No statistically significant difference exists between operating speeds on dry and wet pavements, as long as visibility is not affected decisively.
2. The gap between operating speeds of passenger cars and trucks increases with increasing degree of curve, but not in a manner that could create critical driving maneuvers on gradients up to 5 percent.
3. Accident rates increase with increasing degree of curve, despite the presence of stringent traffic warning devices at curved sites.
4. Vehicle acceleration and deceleration end or begin about 700 to 750 feet from the end of an observed curved road section.
5. Consistency in horizontal alignment, as reflected by a smooth operating speed profiles, can be achieved by examining the degree of curve.
6. For evaluating horizontal design consistency or inconsistency, the following changes in degrees of curve and their subsequent impact on changes in operating speeds, based largely on mean accident rates (see table 2), provide a reasonable (and quantifiable) classification system for differentiating good design and poor design:

Case 1 (good design):

Range of change in degree of curve: $\Delta DC \leq 5°$.
Range of change in operating speed: $\Delta V85 \leq 6$ mph (10 km/h).

For these road sections, consistency in horizontal alignment exists, and the horizontal alignment does not create inconsistencies in vehicle operating speeds.

Case 2 (fair design):

Range of change in degree of curve: $5° < \Delta DC \leq 10°$.
Range of change in operating speed: 6 mph $< \Delta V85 \leq 12$ mph (20 km/h).

These road sections have at least minor inconsistencies in geometric design.

Case 3 (poor design):

Range of change in degree of curve: $\Delta DC > 10°$.
Range of change in operating speed: $\Delta V85 > 12$ mph (20 km/h).

These road sections have strong inconsistencies in horizontal geometric design combined with breaks in the speed profile that may lead to critical driving maneuvers.

As shown in table 2, the results indicate significant increases (at the 95 percent level of confidence) in the mean accident rates among the different degree of curve classes compare. In other words, higher accident rates can be expected with higher degree of curve classes, despite stringent traffic warning devices often installed at the curve sites.

The results of table 2 indicate that gentle curvilinear horizontal alignments consisting of tangents or transition curves combined with curves up to 5° showed the lowest average accident risk. These observations agree well with the findings of some European guidelines (*8, 10, 11*) and the statements of AASHTO 1984 (14, pp. 248f) concerning "General Controls for Horizontal Alignment."

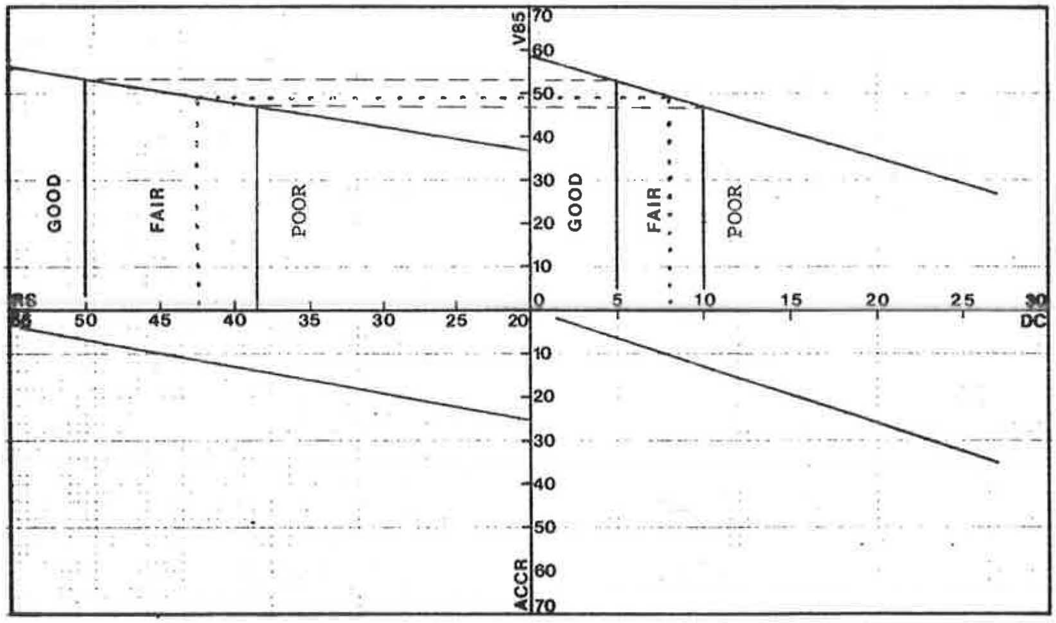
For horizontal alignments with changes in degrees of curve between 5° and 10° between successive design elements (defined as fair designs), the average accident rate in table 2 is twice as high as for those between 1° and 5°. For changes between 10° and 15° of curve (defined as poor designs), the accident rate is four times the rate associated with degrees of curve between 1° and 5°. For greater changes in degree of curve, the average accident rate is even higher. This confirms that changes in degree of curve between successive design elements that exceed 10° should be interpreted as poor designs while those in the range between 5° and 10° can still be judged as fair designs.

NOMOGRAMS FOR EVALUATING OPERATING SPEEDS AND ACCIDENT RATES

The regression models for all lanes combined, formulated in table 1, are depicted in figure 1. From the resulting nomogram, the designer is able to roughly predict operating speeds (85th-percentile speeds) and accident rates on curves or curved sections of two-lane, rural highways from beforehand knowledge of the degree-of-curve or posted-recommended-speed parameters.

On the other hand, the regression models for the individual lane widths formulated in table 1 are depicted in figure 2. As the figure shows, operating speeds decrease with increasing degree of curve, for different lane widths, in a nearly parallel manner.

With respect to the relationship "accident rate vs. degree of curve" figures 1 and 2 reveal that accident rates increase



DC = Degree of curve (degree/100 feet),
V85 = Estimate of operating speed, expressed by the 85th-percentile speed (mph) (range up to 27°),
ACCR = Estimate of accident rate (acc./10^6 vehicle-miles)(range: 1° - 27°),
RS = Estimate of recommended speed (mph).

FIGURE 1 Nomogram for evaluating operating speeds and accident rates in curves or curved sections as related to degree of curve and posted recommended speed for all lane widths (*17*).

with increasing degree of curve, despite the presence of posted advisory speeds at curved sites (see figure 1). Furthermore, as figure 2 reveals, for degrees of curve $\leq 5°$, there appear to be non-significant differences in accident rates between the individual lane widths. For higher degrees of curve, the gap between accident rates on 12-foot and 11/10-foot lanes becomes wider and wider.

For all lanes combined, one can expect, as figure 1 reveals, an accident rate of about six accidents per million vehicle miles (mvm) for a 5° of curve, and an accident rate of about thirteen accidents per mvm for a 10° curve. That means that the accident risk on sections with a change in degree of curve of $\Delta DC > 10°$, as compared to sections with a change in degree of curve of $\Delta DC > 5°$ is at least twice as high. For higher degrees of curve, these comparisons are even more unfavorable. Similar results are obvious from figure 2, too, when comparing the accident rates for individual lane widths. Note that the differences between 12-foot and 11-foot lane widths are, more or less, more pronounced than those between 11-foot and 10-foot lane widths.

These relationships between roadway geometry, operating speeds and accidents in conjunction with the classification system form the basis for a design methodology. From geometric definition, the designer may predict operating speeds. Wide variations in operating speeds are shown to be further indicators of accidents. Reasonable judgments can then be applied to discriminate good, fair, and poor design on the basis of safety indicators but using only design information.

TUNING OF RADII-SEQUENCES

For an easy illustration of the following design procedure, the recommended boundaries for good, fair, and poor designs, as related to degree of curve, were converted to radii of curve. For instance, figure 3 shows the tuning of radii-sequences for succeeding curves, in the same or in the opposite direction, for different design cases. As figure 3 demonstrates, a radius of R = 500 feet can be combined, for example, in the case of—

- good designs: with a range of radii between $\sim 350 < 500 < 900$ feet and
- fair designs: with a range of radii between $\sim 270 < 500 < 3{,}500$ feet.

Regarding a sequence tangent-to-curve, the boundaries of good designs ($DC \leq 5°$) correspond to radii of curve ($R \geq$ 1,200 ft); thus, curves with radii $R \geq 1{,}200$ feet should follow an "Independent Tangent" in order to not create inconsistencies in vehicle operating speeds. The boundaries of fair designs ($5° < DC \leq 10°$) correspond to radii of curve (1,200 ft $> R \geq 600$ ft); radii within this range should follow an "Independent Tangent" in the sequence tangent-to-curve for fair design practices. These values agree well with the minimum radii for design speeds of 60 mph (good design) and of about 45 mph (fair design) for a superelevation rate of 8 percent in table III-6 (*14*).

By applying figure 3, the designer could immediately decide

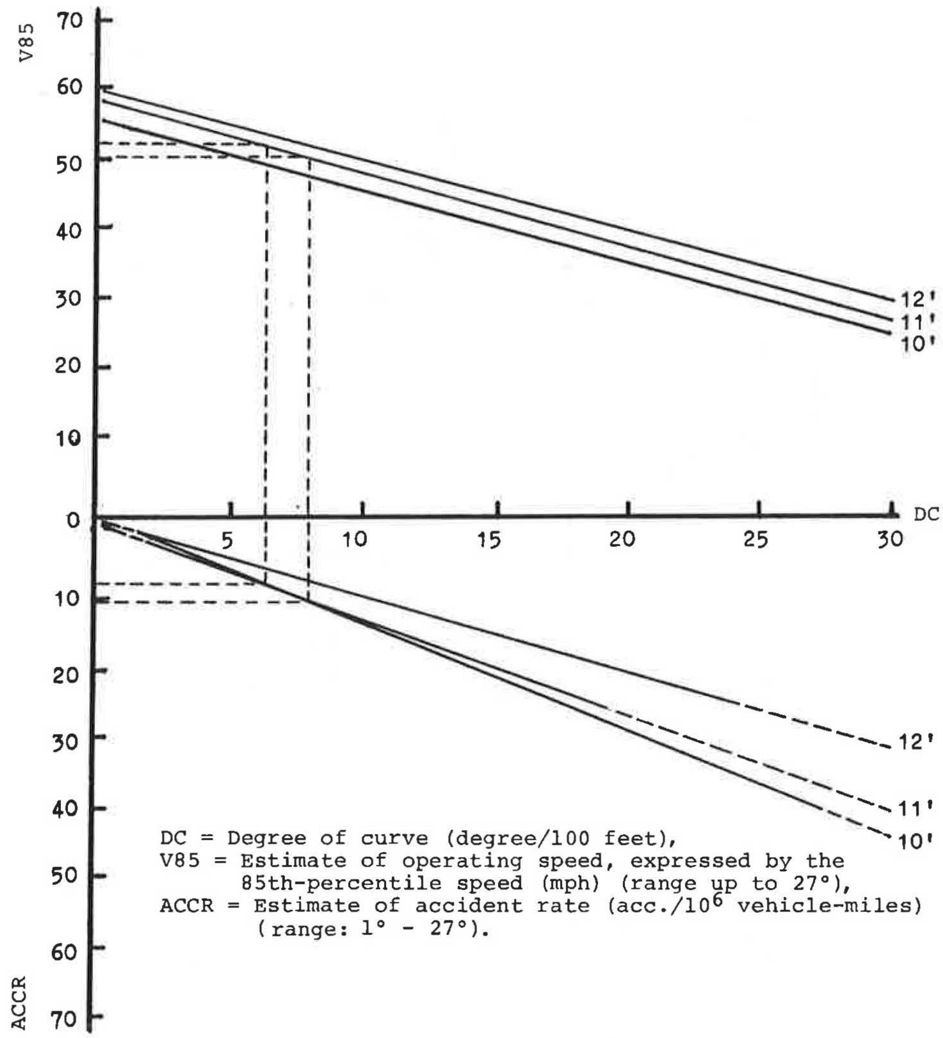


FIGURE 2 Nomogram for evaluating operating speeds and accident rates as related to degree of curve for individual lane widths.

whether or not certain radii of succeeding curves fall into the range of good, fair or poor design practices. For example, combining a radius of 1,000 feet—

- with 300-foot radius would be a poor design,
- with a 500-foot radius would be a fair design, and
- with a 700-foot radius would be a good design.

EVALUATION OF TANGENTS IN THE DESIGN PROCESS

Lamm et al. (companion paper in this Record) have established boundaries for tangent-lengths that are to be regarded as "independent" or "non-independent" design elements. For independent tangents, the sequence "tangent-to-curve" controls the design process, while for non-independent tangents, it is the sequence "curve-to-curve" that controls the design process.

Table 3 shows maximum allowable lengths of tangents that are regarded as non-independent design elements. The values with an asterisk represent lengths of tangents on which 85th-percentile speeds of 58 mph can be reached, as determined by Lamm et al. in a companion paper in this Record.

When dealing with tangent lengths, the following three cases must be distinguished.

Case 1

The existing tangent length is smaller than the maximum allowable one in table 3 that corresponds to the nearest 85th-percentile speed of the curve with the higher degree of curve. From this it follows that the tangent is to be regarded as non-independent (companion paper in this Record by Lamm et al.). Changes in degree of curve and operating speeds must be related to any two successive curves since the tangent in-between can be assumed to be negligible in the design process; that is, the sequence curve-to-curve controls the design process in this case.

Case 2

The existing tangent length is at least twice as long as the values listed in the last column of table 3, again related to the

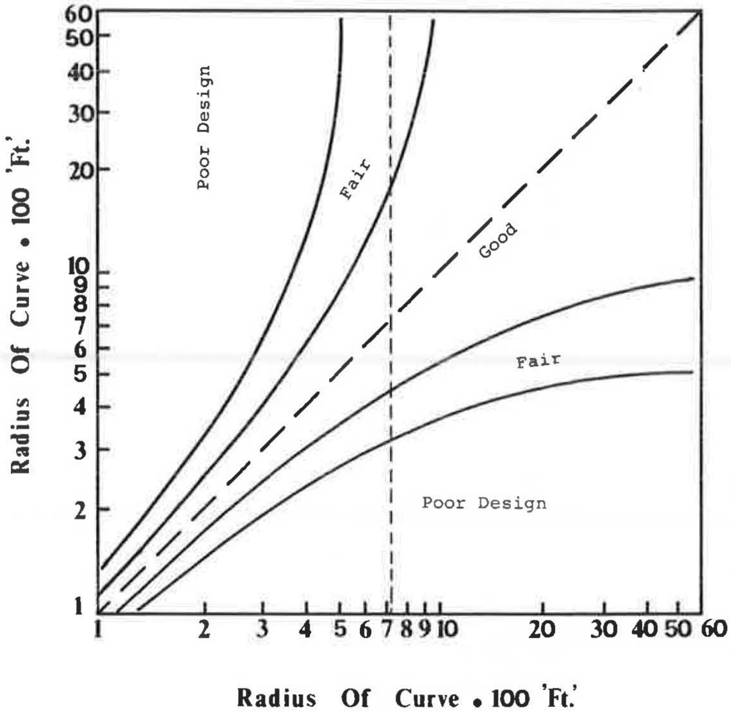


FIGURE 3 Tuning of radii-sequences of succeeding curves for good and fair design practices.

TABLE 3 RELATIONSHIP BETWEEN TANGENT LENGTHS AND 85TH-PERCENTILE SPEED CHANGES FOR SEQUENCES: TANGENTS TO CURVES

V85 in Curve	V85 in Tangent				
	34	40	46	52	58
22	250	425	625	850	1100*
28		325	500	725	1000*
34			375	600	850*
40				425	675*
≥46					475*

▭ Maximum allowable Lengths of Tangents, regarded as "Non-Independent Design Elements", (ft)

V85 = 85th-Percentile speed in curve or tangent (mph)

*For these values the highest operating speed in tangents V85 = 58 mph can be expected.

nearest 85th-percentile speed of the curve with the higher degree of curve. In this case, it can be assumed, without any calculations, that the tangent is independent (Lamm et al.), and that operating speeds of 56 to 60 mph are good estimates, depending on the individual lane widths (see equations (1a) through (1c) of table 1). In other words, the sequence tangent-to-curve controls the design process.

Case 3

The existing tangent length lies somewhere between Case 1 and Case 2. The operating speed in the independent tangent can be estimated from figure 4 and equations (2) through (4), as derived by Lamm et al.; in other words, the sequence tangent-to-curve controls the design process for both directions of travel.

PROCESS FOR EVALUATING HORIZONTAL DESIGN CONSISTENCY WITH EXAMPLE APPLICATIONS

Primarily at lower design speeds, the changing alignment may cause variations in operating speeds, which may, in turn, increase the accident risk by substantial amounts. Therefore, one of the important tasks in modern rehabilitation of the two-lane, rural road network in the United States is to ensure design consistency and to detect critical inconsistencies in horizontal alignment, especially with regard to RRR projects.

In what follows, the various steps of the design procedure are presented:

(a) Assess the road section where new designs, major reconstructions, or redesigns, such as in the case of RRR projects, may be considered.

(b) Determine for this road section the degree of curve of each curve within the section and the existing tangent length.

(c) Determine the expected 85th-percentile speed for each curve, in accordance with degree of curve, by applying figure 1 for a rough estimate or figure 2 for a more accurate estimate depending on the lane width. Compare equations (1a) through (1c) also.

(d) Conclude whether or not each tangent is an independent design element. For independent tangents, the tangent-to-curve sequence is of prime importance in the design process. For non-independent tangents, it is the sequence curve-to-curve. For independent tangents, determine the corresponding operating speeds according to Case 2 or Case 3 of the previous section.

(e) In accordance with the results of step (c) and step (d), calculate the change in degree of curve (ΔDC), and the change in operating speeds ($\Delta V85$) for the independent tangent-to-curve or curve-to-curve sequence.

Good Design Practices

(f_1) Determine all road sections where changes in degree of curve and changes in operating speeds correspond to the boundaries of good design practices:

Range of change in degree of curve: $\Delta DC \leq 5°$.

Range of change in operating speed: $\Delta V85 \leq 6$ mph (10 km/h).

Result

For these road sections, consistency in horizontal alignment exists and the horizontal alignment does not adversely detract from the expected operating speed profiles. Thus, RRR improvements can be made in most cases without considering traffic warning devices or horizontal alignment redesign. The majority of existing state routes in the United States exhibit these characteristics.

Note

The radii of successive curves should fall into the range of good design practices as shown in figure 3. For a tangent-to-curve sequence, at least curves with radii ($R \geq 1{,}200$ ft) should follow an independent tangent.

Rough estimates of expected accident rates may be made possible from figures 1 and 2 or equations (3a) through (3c) in table 1, depending on the lane width.

Fair Design Practices

(f_2) Determine all road sections where changes in degree of curve and changes in operating speeds correspond to the boundaries of fair design practices:

Range of change in degree of curve: $5° < \Delta DC \leq 10°$.

Range of change in operating speed: $6 \text{ mph} \leq \Delta V85 \leq 12$ mph.

Result

These road sections exhibit at least minor inconsistencies in geometric design. Normally, correcting the existing alignment is not necessary since low cost projects such as traffic warning devices may, to a certain extent, be successful in correcting these defects. For instance, RRR improvements can be installed which consider appropriate recommended speeds (see figure 1), unless a safety problem has been documented. One should note that despite traffic warning devices, road sections with changes in degree of curve that fall into the range (5° to 10°) have average accident rates that are about twice as high as those falling into the range of good design (see table 2).

Note

The radii of successive curves should fall into the range of fair design practices, as shown in figure 3. For a tangent-to-curve sequence at least curves with radii ($1{,}200 \text{ ft} > R \geq 600$ ft), equipped with posted recommended speeds (see figure 1) and arrow designations, should follow an independent tangent.

Rough estimates of expected accident rates may be made from figures 1 and 2 or equations (3a) through (3c) in table 1, depending on the lane width.

To achieve a high level of driving safety, superelevation rates and stopping-sight distances should be related to the expected operating speeds wherever possible.

Example Related to Figure 5

Step (a)
State of New York,
County No. 3604,
Route Number SR34 (mile markers 3094–3115),
Lane Width 11 feet.

Step (b)

Section	*Design Element*	*Degree of Curve*	*Length*
A–B	Tangent	0	0.2 mi ~ 1,060 ft
B–C	Curve	6.4	0.2 mi ~ 1,060 ft
C–D	Tangent	0	0.1 mi ~ 530 ft
D–E	Curve	8.0	0.1 mi ~ 530 ft
E–F	Tangent	0	1.5 mi ~ 7,920 ft

Step (c)

Expected 85th-Percentile Speed	*From Figure or Equation*	*Measured 85th-Percentile Speed*
Curve BC	52 mph	Figure 2 or Eqn. (1b)
Curve DE	50 mph	53 mph

Step (d)

Tangent AB (1,060 feet) In accordance with Case 2, see the section on "Evaluation of Tangents." The expected operating speed in the following curve is 52 mph; the nearest value in table 3 is 46 mph; two times the value of the last column of table 3 is 950 feet < 1060 feet. That means that tangent AB is independent. It follows that $V85 = 58$ mph, according to figure 2 or equation (1b) for a lane width of 11 feet.

Tangent CD (530 feet) In accordance with Case 3, see the section on "Evaluation of Tangents." The expected operating speed in the curve with the higher degree of curve is 50 mph; the nearest value in table 3 is 46 mph; the maximum length of tangent regarded as non-independent is 475 feet < 530 feet. That means that tangent CD is independent. Thus, the operating speed in the tangent can be estimated, according to Figure 4 (from the companion paper in this Record), as follows:

$$X = \frac{(52 + 50) \cdot (52 - 50)}{2 \cdot 1.302} \approx 80 \text{ ft} \tag{2}$$

$$TL - X = 530 - 80 = 450 \text{ ft}$$

$$\Delta V85_T = \frac{-2 \cdot (52) \pm \sqrt{4 \cdot (52)^2 + 5.208 \cdot (450)}}{2} \tag{3}$$

This implies the operating speed in Tangent CD is

$$V85T = 52 + 5 = 57 \text{ mph} \tag{4}$$

Tangent EF (7,920 feet) Independent, $V85 = 58$ mph.

Step (e)

Sequence	*Change in Degree of Curve ΔDC*	*Change in Operating Speed ΔV85*
Tangent AB to Curve BC	|0–6.4| = 6.4	|58–52| = 6 mph
Curve BC to Tangent CD	|6.4–0| = 6.4	|52–57| = 5 mph
Tangent CD to Curve DE	|0–8.0| = 8.0	|57–50| – 7 mph
Curve DE to Tangent EF	|8.0–0| = 8.0	|50–58| = 8 mph

Step (f_2)

For the existing alignment of figure 5, step (e) reveals that changes in degree of curve, and changes in operating speeds between tangent AB and curve BC in the direction AF, between tangent CD and curve DE in the direction AF, and between tangent EF and curve DE in the direction FA fall into the range of fair design.

Note that the degrees of curve of 6.4° and 8.0° correspond to radii of 900 feet and 720 feet. Since these radii lie between 600 feet and 1,200 feet, curve BC and curve DE, combined with independent tangents, fall into the range of fair design. This can be determined from figure 3, too, when radii of 720 feet and 900 feet are combined with a tangent (R is greater than or equal to 6,000 feet), according to the scale. The existing recommended speed of 45 mph combined with arrow designations, see figure 5, agrees well with the value of about 45 mph that can be determined from figure 1 for curve DE with a degree of curve of 8°. The expected accident rate can be determined from figure 2 or calculated from equation (3b). The observed accident rate was calculated from the following equation:

$$ACCR = \frac{(\text{No. Acc.}) \cdot (10^6)}{(365) \cdot (\text{No. Years}) \cdot (L) \cdot (AADT)} \tag{5}$$

where

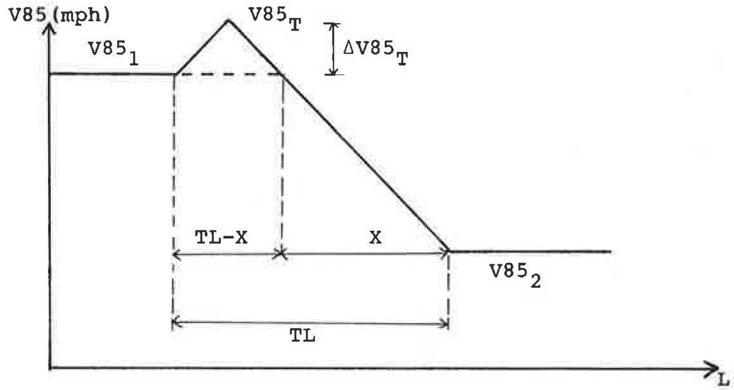
$ACCR$ = number of accidents per 1 million vehicle miles,
No. Acc. = number of accidents per years investigated,
No. Years = number of years investigated,
L = length of curve or curved section in miles, and
$AADT$ = Average Annual Daily Traffic (vehicles in both directions).

This implies that

	Expected Accident Rate	*Observed Accident Rate*
Curve BC	8.5	6.9
Curve DE	10.7	9.1

The expected accident rates agree, relatively well, with the observed ones and the mean accident rate for fair design of table 2.

Thus, one can conclude that the horizontal alignment of figure 5 corresponds to fair design practices and does not necessarily need improvements in geometric design. But it should not be forgotten that at least minor inconsistencies in a)



Legend:

TL = Tangent length, greater than the maximum allowable lengths for "Non-Independent Tangents" of Table 3 (ft),

X = Acceleration or deceleration distance between curve 1 and curve 2 (ft),

$V85_1, V85_2$ = Operating speeds in curves (mph),

$V85_T$ = Operation speed in tangent (mph),

$\Delta V85_T$ = Difference between the operating speed in the curve with the <u>lower</u> degree of curve and the operating speed in the tangent (mph).

$$X = \frac{(V85_1 + V85_2) \cdot (V85_1 - V85_2)}{2.604} \qquad (4)$$

$$V85_T^{*} = V85_1 + \Delta V85_T \qquad (5)$$

$$\Delta V85_T = \frac{-2 \cdot (V85_1) \pm \sqrt{4(V85_1)^2 + 5.208(TL-X)}}{2} \qquad (6)$$

*Note that for determining $V85_T$ always the operating speed of the curve with the lower degree of curve has to be selected.

FIGURE 4 Example for estimating the operating speed in an independent tangent.

horizontal alignment do exist. For instance, despite the presence of traffic warning devices (recommended speeds of 45 mph and arrow signs), the accident rates on the observed curved sites are about twice as high as the mean accident rate for good design, as shown in table 2.

To achieve a high level of driving dynamic safety, it is recommended to increase the existing superelevation rates from six percent to nine percent during the next resurfacing project, as shown for curve DE by the following calculation:

$$e_{max} = \frac{DC \cdot V85^2}{85{,}660} - f_R$$

$$e_{max} = \frac{8 \cdot 50^2}{85{,}660} - 0.14 = 0.09$$

where fR = side friction factor obtained from table III-6 (*14*); fR = 0.14 (estimated for an operating speed of 50 mph).

Similar calculations have to be performed if stopping sight distances are insufficient.

Poor Design Practices

(*f*3) Determine all road sections where changes in degree of curve and changes in operating speeds correspond to the boundaries of poor design practices:

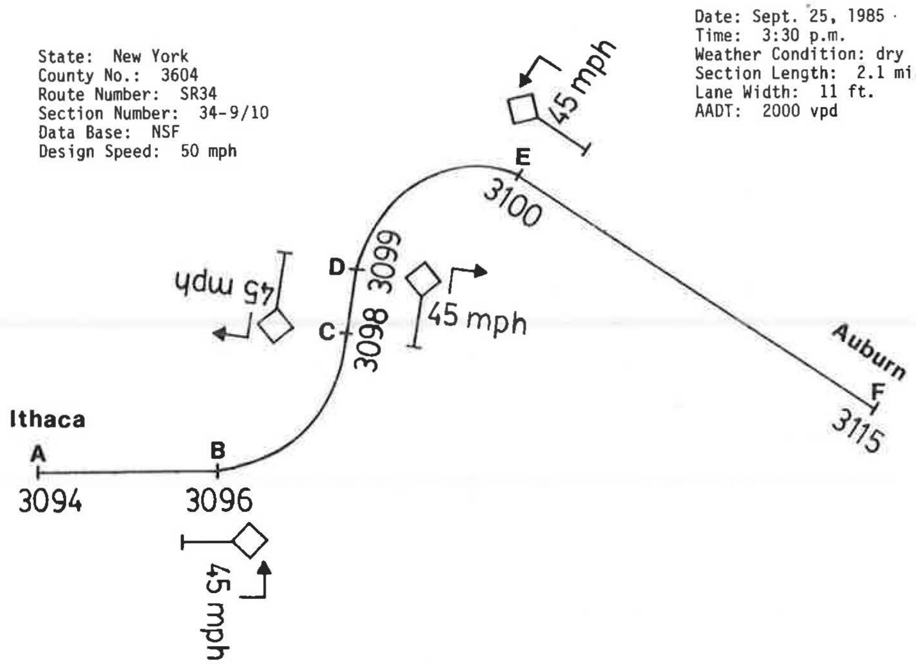
Range of change in degree of curve: $\Delta DC > 10°$.

Range of change in operating speed: $\Delta V85 > 12$ mph (20 km/h).

Result

These road sections represent strong inconsistencies in horizontal geometric design, combined with those breaks in the speed profile that may lead to critical driving maneuvers.

Despite stringent recommended speeds combined with arrow designations and chevrons (see figure 1), road sections with changes in degree of curve that fall into this range (10° to 15°) have about four times an average accident rate as those that



	Curve BC	Curve DE
Degree of Curve	6.4°	8.0°
Superelevation Rate	6%	6%
Measured 85th-percentile Speed* (Average for both directions)	--	53 mph
Stopping Sight Distance	0.1 mi.	0.1 mi.
Grade	0%	±0.5%
Recommended Speed	45 mph (Arrow Signs)	45 mph (Arrow Signs)
Number of Accidents++	3	2

*Not required for the design procedure, here only presented for comparison reasons.

++For an investigated period of three years (Jan. 1982 to Dec. 1984).

FIGURE 5 Case study of State Route 34 in the State of New York.

fall into the range of good design, and about twice as high as those that fall into the range of fair design (see table 2). Normally, for example, for RRR projects, high cost projects such as redesigns of at least hazardous road sections should be recommended, unless there was no documented safety problem.

Note

- Ranges of radii of successive curves that would represent poor design practices are shown in figure 3. For a tangent-to-curve sequence, curves with radii (R 〈 600 feet) should not be allowed to follow an independent tangent.
- Rough estimates of expected accident rates may be made possible from figures 1 and 2 or equations (3a) through (3c) in table 1, depending on the lane width.

PROCESS FOR EVALUATING DESIGN SPEED AND OPERATING SPEED DIFFERENCES

All reviewed highway geometric design guidelines (*8–14*) indicate that the design speed should be constant along longer roadway sections. Furthermore, the design speed (V_d) and the 85th-percentile speed ($V85$) must be well balanced to insure a fine tuning between road characteristic, driving behavior, and driving dynamics. Experiences (*1, 5, 6*) have shown that the design speed is sometimes lower than driver expectations and judgement of what the logical speed should be, especially on independent tangents. Therefore, harmonizing design speed and operating speed is another important goal that should be considered in rehabilitation of two-lane, rural highways.

To achieve this goal, it is recommended that the designer refer to step (c) and step (d) of the previous section and determine the expected 85th-percentile speed of every independent tangent or curve in the observed road section.

The 85th-percentile speed ($V85$) of every independent tangent, curve, or curved section must be tuned with the existing or selected design speed (V_d) in the following manner:

1. $V85 - V_d \leq 6$ mph (10 km/h) (good designs); no adaptations or corrections are necessary.
2. 6 mph $< V85 - V_d \leq 12$ mph (fair design); superelevation rates in curves or curved sections and stopping sight distances must be related to the expected 85th-percentile speed. Thus, it is inferred that the driving dynamic safety demand will not exceed the driving dynamic safety supply under wet pavement conditions, compare step (f_2).
3. $V85 - V_d > 12$ mph (20 km/h) (poor design). The 85th-

percentile speed should not be allowed to exceed the design speed by more than 12 mph (20 km/h). If such a difference occurs, normally the design speed should be increased. For example, redesigns of hazardous road sections are recommended, unless there was no documented safety problem.

With regard to a well-balanced design one should strive for a uniform design speed within an observed road section of substantial length, especially between independent tangents and curves. This conclusion is well expressed in the AASHTO Design Guide (*14*), as follows:

> In horizontal alignment, predicted on a given design speed, consistent alignment always should be sought. Sharp curves should not be introduced at the end of long tangents. Sudden changes from areas of flat curvature to areas of sharp curvature should be avoided. Where sharp curvature must be introduced it should be approached, where possible, by successively sharper curves from the generally flat curvature.

This can be done by applying the ranges of good designs, or, if necessary, of fair designs in figure 3.

In an example related to figure 5, where the design speed is 50 mph,

Design Speed: 50 mph

Section	$V85 - V_d$ *(mph)*	ΔV *(mph)*
Tangent AB	58 − 50	8
Curve BC	52 − 50	2
Tangent CD	57 − 50	7
Curve DE	50 − 50	0
Tangent EF	58 − 50	8

The results are inconsistent. At least for three design elements the differences between operating speeds and design speeds correspond to fair design practices 6 mph $< V85 - V_d \leq$ 12 mph. That means that minor inconsistencies in horizontal alignment do exist. However, correcting the existing alignment is not necessary since a documented safety problem related to fair designs does not exist (compare step (f_2)). Superelevation rates on curves have to be adjusted to the expected operating speeds to achieve a high level of driving dynamic safety, as it was shown in step (f_2).

CONCLUSION

By applying this procedure, the highway engineer can easily control good and fair designs and can detect poor horizontal designs during RRR project planning. The procedure has been illustrated using existing alignments to verify its validity, but such a technique could be applied as well for new designs, major reconstructions, or redesigns. From knowledge of degree of curve of each curve, and the existing transition length (length of transition curves or length of tangent) between two curves, the highway engineer can evaluate the horizontal alignment during the design stages according to the discussed design procedure.

For example, where the design analysis reveals road sections of fair or even poor designs, these sections can be corrected by changing the design element sequences in question. Such changes may be an independent tangent-to-curve sequence, or a curve-to-curve sequence, according to design-element sequences for good design practices shown in figure 3.

The impact of tuning the alignment in this way would result, in general, in more curvilinear alignments. Furthermore, the designer can predict expected operating speeds and accident rates on curved sections by applying the nomograms of figures 1 and 2. However, because of the low coefficients of determination for the accident rate related regression equations, caution should be exercised when using the equations for prediction purposes. In addition, the designer can predict appropriate recommended speeds by using figure 1 in cases where fair designs have to be maintained, or even newly introduced, such as when poor designs can only be improved to fair designs because of terrain or other constraints.

Finally, the design speed concept can be applied in the future in a more appropriate way by harmonizing design speeds and expected operating speeds for the selected design element sequences, already during the design stages.

It is felt that routine use of a procedure such as this by design agencies could lead to more cost effective and safe geometry for new designs, major reconstruction, and, especially, RRR projects. It is hoped that such procedures will be adopted and will ultimately become a part of national and state guidelines.

Note, the prediction equations formulated and cross-validated in this study are based on data from a limited geographic area of the state of New York and may only be appropriate for investigations within that state or region. Some caution should be exercised in extrapolating the design procedure to other areas with differing laws, law enforcement, driver behavior, terrain, weather, and traffic control devices. The models are quite possibly applicable in wider areas (and that is certainly desirable), but testing will be required to determine their suitability in other geographical areas.

ACKNOWLEDGMENT

This study was sponsored by the New York State Governor's Traffic Safety Committee, Albany, New York and by the National Science Foundation. The senior author wishes to thank the members of the TRB Geometric Design Committee for their encouragement and their valuable discussions while the research on consistency of highway geometric design was being conducted. Special thanks go to Prem Goyal for his assistance in preparing the example applications.

REFERENCES

References with an asterisk (*) have been translated from their original language into English for easy reference in the United States. The authors apologize for any inaccuracies that may have resulted as a consequence of the process of translation.

1. J. E. Leisch and J. P. Leisch. New Concepts in Design Speed Application. In *Transportation Research Record 631*, TRB, National Research Council, Wash., D.C., 1977.
2. J. Hayward, R. Lamm, and A. Lyng. *Survey of Current Geometric and Pavement Design Practices in Europe: Geometric Design*. International Road Federation, July, 1985.
3. E. U. Hiersche, R. Lamm, K. Dieterle, and A. Nikpour.

Effects of Highway Improvement Measures Designed in Conformity with the RAL-L on Traffic Safety on Two-Lane Highways. *Forschung Strassenbau und Strassenverkehrstechnik*, Vol. 431, 1984.

4. J. C. Hayward. Highway Alignment and Superelevation: Some Design-Speed Misconceptions. In *Transportation Research Report 757*, TRB, National Research Council, Wash., D.C., 1980.
5. G. Koeppel and H. Bock. Operating Speed as a Function of Curvature Change Rate. *Forschung Strassenbau und Strassenverkehrstechnik*, Vol. 269, 1979.
6. R. Lamm. *Driving Dynamics and Design Characteristics—A Contribution for Highway Design under Special Consideration of Operating Speeds*. Publications of the Institute of Highway and Railroad Design and Construction, University of Karlsruhe, Vol. 11, Federal Republic of Germany, 1973.
7. J. A. Cirillo. Safety Aspects of RRR Projects. *Public Roads*, Vol. 48, No. 3, 1984, pp. 103–107.
8. *Guidelines for the Design of Roads (RAS-L-1)*. German Road and Transportation Research Association, Committee 2.3, Geometric Design Standards, Edition 1984.
9. R. Lamm and J. Cargin. Translation of the *Guidelines for the Design of Roads (RAS-L-1)*. Federal Republic of Germany, and the Swiss Norm SN 640080a, *Highway Design, Fundamentals, Speed as a Design Element*, 1981, as discussed by K. Dietrich, M. Rotach, and E. Boppart, in *Road Design*, ETH Zuerich, Institute for Traffic Planning and Transport and Technique, Edition 1983, prepared at Clarkson University for the Safety and Design Division, Federal Highway Administration, U.S. Department of Transportation, May 1985.
10. *Highway Design, Fundamentals, Speed as a Design Element*. Swiss Association of Road Specialists (VSS), Swiss Norm SN 640080a, Edition 1981.
11. *Standard Specifications for Geometric Design of Rural Roads*. National Swedish Road Administration, Sweden, Edition 1982.
12. *Instruction sur les Conditions Techniques D'Aménagement de Routes Nationales*. Ministere de l'Equipement et du Logement, France, Edition 1975.
13. *Highway Link Design, Geometric Alignment Standards*. Departmental Standard TD9/81, Department of Transport, Great Britain, 1981.
14. *A Policy on Geometric Design of Highways and Streets*. AASHTO, Washington, D.C., 1984.
15. R. Lamm and E. M. Choueiri. Recommendations for Evaluating Horizontal Design Consistency Based on Investigations in the State of New York. In *Transportation Research Record 1122*, TRB, National Research Council, Washington, D.C., 1987.
16. E. M. Choueiri. *Statistical Analysis of Operating Speeds and Accident Rates on Two-Lane Rural State Routes*. Ph.D. Dissertation, Clarkson University, January 1987.
17. R. Lamm and E. M. Choueiri. *A Design Procedure to Determine Critical Dissimilarities in Horizontal Alignment and Enhance Traffic Safety by Appropriate Low-Cost or High-Cost Projects*. Final Report for the National Science Foundation, Washington, D.C., March 1987.
18. E. M Choueiri and R. Lamm. *Operating Speeds and Accident Rates on Two-Lane Rural Highway Curved Sections—Investigations about Consistency and Inconsistency in Horizontal Alignment*. Part I of the Research Contract "Rural Roads Speed Inconsistencies Design Methods," State University of New York Research Foundation, Albany, New York, July 1987.
19. R. Lamm, J. C. Hayward, and J. Cargin. Comparison of Different Procedures for Evaluating Speed Consistency. In *Transportation Research Record 1100*, TRB, National Research Council, Washington, D.C., 1986.
20. R. Lamm and E. M. Choueiri. Relationship Between Design, Driving Behavior and Accident Risk on Curves. *Proceedings of the Fortieth Annual Ohio Transportation Engineering Conference*, Ohio State University, in Cooperation with the Ohio Department of Transportation, December 1982, pp. 87–100.
21. R. Lamm and J. Cargin. Identifying Operating Speed Inconsistencies on Two-Lane Rural Roads. *Proceedings of the Thirty-Ninth Annual Ohio Transportation Engineering Conference*, Ohio State University, in Cooperation with the Ohio Department of Transportation, December 1985, pp. 13–22.

Publication of this paper sponsored by Committee on Geometric Design.

Tangent as an Independent Design Element

RUEDIGER LAMM, ELIAS M. CHOUEIRI, AND JOHN C. HAYWARD

Reviews of design guidelines for rural roads in Germany, France, and Switzerland reveal that highway designers adhere to controls on maximum and minimum lengths of tangents between successive curves. Minimum tangent lengths are prescribed to promote operating speed consistency, and maximum lengths are suggested to combat driver fatigue. Current U.S. practice does not set maximum or minimum lengths of tangents; instead current AASHTO policy favors long tangent sections for passing purposes on two-lane, rural roads. This paper presents a recommended strategy for U.S. highway designers to consider tangent lengths explicitly in rural highway design. The proposed approach uses recommended operating speed differences between successive horizontal geometric elements (curves and tangents) and acceleration or deceleration profiles derived from car-following tests to establish limits. Recommendations are also provided for transition lengths (tangent length) between successive curved roadway sections for (a) tangents that should be regarded as "non-independent" design elements; that is, the sequence "curve-to-curve" is the most important element of the design process and (b) tangents that should be regarded as "independent" design elements; that is, the sequence "tangent-to-curve" is the most important element of the design process.

In the highway geometric design process, tangents and horizontal curves with or without transition curves are regarded as design elements. Most of the reviewed highway geometric design guidelines (*1–7*) give recommendations for maximum or minimum tangent lengths.

For example, in the Federal Republic of Germany (*2, 3*) tangent lengths between curves are limited by the design speed. The maximum length in meters of tangent sections between two curves may not exceed twenty times the design speed of that roadway. In this way long tangents are controlled and a curvilinear environment is encouraged.

Minimum tangent lengths must be at least six times the design speed. For a typical design speed of 100 km/h (~60 mph) this would correspond to a maximum tangent length of 2,000 meters (6,500 feet) and a minimum tangent length of 600 meters (2,000 feet).

To avoid driver fatigue, it is recommended in France (*6*) that tangent sections be limited to a maximum of 40 to 60 percent of long roadway sections with maximum single tangent lengths between 2,000 and 3,000 meters (6,500 to 10,000 feet).

Swiss highway officials (*3, 4*) also limit tangent lengths to limit driver fatigue. Designs that permit more than one minute of driving on a straight section are not permitted. Minimum tangent lengths are related to "project speeds," which roughly translate to American practice as "theoretical operating speeds." For example, for a project speed of 100 km/h (~60 mph) a minimum tangent length of 150 meters (500 feet) would be permitted.

In the 1984 AASHTO Policy on Geometric Design of Highways and Streets (*1*), specific values for maximum or minimum tangent lengths are not specified. But the following statement is listed under General Controls for Horizontal Alignment: "Although the aesthetic qualities of curving alignment are important, passing necessitates long tangents on two-lane highways with passing sight distance on as great a percentage of the length of highway as feasible." This statement clearly supports the application of long tangents, especially for the design of two-lane, rural highways.

The only method developed to evaluate acceleration or deceleration movements between sequences of curve-to-curve or tangent-to-curve was found in the geometric design guidelines of Switzerland (*3, 4*). The Swiss have developed a formula for calculating transition lengths (tangent length), that is, the distance required for acceleration or deceleration of a vehicle as it approaches or leaves a curve based on the project speed difference between two curves or between a tangent and a curve. Unallowable ranges, or those that should be avoided for these transition lengths, are also tabulated (*8*).

BACKGROUND AND OBJECTIVE

In several publications and research reports (*8–15*) the authors recommend the following boundaries for changes in degree of curve and operating speed between successive design elements for good, fair and poor design practices. With the exception of some very good designs, the existing American design for low-volume, two-lane rural roads consists of sequences of curves and tangents where the transitions are rarely equipped with transition curves.

- Good design is present where successive changes in degree of curve are limited to 5°, and changes in operating speeds are limited to 6 mph (10 km/h) between successive design elements. The horizontal alignment operates well.
- Fair designs exist where changes of 5–10° in degree of curve are present, and changes of 6 mph to 12 mph (20 km/h) in operating speeds between successive design elements are permitted. Normally, low-cost projects such as traffic warning devices are warranted unless there is a documented safety problem.
- Poor designs show changes of more than 10° in degree

R. Lamm, Clarkson University, Potsdam, N.Y. 13676. E. M. Choueiri, N. Country Community College, Saranue Lake, N.Y. 12982. J. C. Hayward, Michael Baker Jr., Inc., Beaver, Pa. 15009.

of curve and differences of more than 12 mph (20 km/h) in operating speeds. Normally, high-cost projects such as redesign of at least hazardous road sections are recommended, unless there is no documented safety problem.

Furthermore, the following prediction equation was developed in references (13–15) for the relationship between expected operating speed and degree of curve, including all investigated lane widths from 10 feet to 12 feet.

$$V85 = 58.656 - 1.135DC;\ R^2 = 0.787 \quad (1)$$

where

$V85$ = Estimate of operating speed, expressed by the 85th-percentile speed for passenger cars (mph),
DC = Degree of curve (degree/100 ft), range: 0° to 27°, and
R^2 = Coefficient of determination.

(The above equation is valid for road sections with grades less than or equal to 5 percent and annual average daily traffic (ADT) values between 400 and 5,000 vehicles per day.)

To illustrate the application of equation (1), the following operating speeds could be expected in a sequence from a tangent to a curve with a degree of curve of 15° or vice versa:

Tangent: $DC = 0° \rightarrow \rightarrow V85 \sim 58$ mph

Curve: $DC = 15° \rightarrow \rightarrow V85 \sim 41$ mph

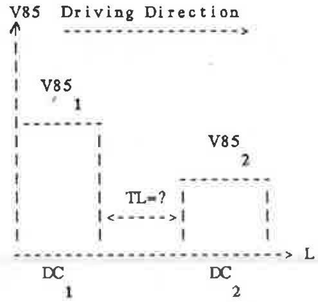
The speed change from the tangent to the curve is $\Delta V85$ = 17 mph. This value is far beyond the maximum allowable change in operating speeds, even for fair design practices defined above where $\Delta V85 \leq 12$ mph.

However, this statement would be true only for a relatively long tangent. The tangent must be long enough that a driver can reach the top 85th-percentile speed of 58 mph expressed by equation (1) for $DC = 0°$. For shorter tangents between succeeding curves, it would be expected that the average driver in a typical vehicle would not be able to accelerate or decelerate in such a way that the boundaries for good design practices ($\Delta V85 \leq 6$ mph) or even for fair design practices ($\Delta V85 \leq 12$ mph) may be exceeded. In those cases, operating speed changes would be related to the two successive curves, and the relatively short tangent between could be neglected in the design process for evaluating horizontal design consistency or inconsistency and for harmonizing design speed and operating speed. Therefore, the task of this research is to provide recommendations for transition lengths (tangent lengths) between successive curves for

- Tangents that should be regarded as non-independent design elements and the sequence curve-to-curve controls the design process, and
- Tangents that should be regarded as independent design elements and the sequence tangent-to-curve controls the design process.

ACCELERATION AND DECELERATION RATES

The transition length (TL) is that road section where the operating speed is changing between two design elements with the operating speeds $V85_1$ and $V85_2$ as assumed in the following sketch (*3, 4*). The transition length is given by



$$TL = (v_1^2 - v_2^2) / 2a = \frac{v_1 + v_2}{2} \cdot \frac{v_1 - v_2}{a} = \frac{\overline{V85} \cdot \Delta V85}{0.465a} \quad (2)$$

where

$\overline{V85}$ = average 85th-percentile speed between successive curves (mph),
$\Delta V85$ = difference between the 85th-percentile speeds (mph),
TL = transition length (tangent length) (ft), and
a = acceleration/deceleration rate (ft/sec^2).

When the degrees of curve of two successive design elements are known, the expected 85th-percentile speeds can be determined by equation (1). To evaluate the transition lengths from equation (2), acceleration or deceleration rates between successive design elements must be known.

To determine an estimate of the coefficient a in equation (2), typical accelerations and decelerations were studied between tangents and specific curved sections of two-lane rural highways (*13–15*). Because of financial and time constraints, acceleration and deceleration movements from tangents-to-curves or curves-to-tangents were made at curves where speeds of 30 mph (three sections), 35 mph (two sections), and 40 mph (one section) were recommended. The study sites were located in St. Lawrence County in New York.

The optimal procedure required that the speeds of individual vehicles be recorded. To accomplish this, an investigation car (the "follow car"), a car observed in the field (the "test car"), and a tape recorder on which to place any relevant information were used. Note that two persons, a driver and an observer, were required in the "follow car" to allow observation of the situation while speed data were being recorded.

Measurements of travel speeds were made at particular points along the routes. The measurement points were uniform in characteristics:

- Sections were horizontal (longitudinal grades less than 1.5%).
- Intersections and places where an influence on traffic flow might be expected through changes in the highway surroundings were not present in the sections.
- Cross sections were representative with regard to the width of the roadway. Three sections with 10-ft lane width and three sections 11-ft lane width were selected.
- Sight conditions at measuring points were adequate.
- Points of measurements were equipped so as not to be recognizable as such by drivers but obvious enough to be seen by the observers in the follow car.

In all cases, eleven spots (from the beginning of the curve into the tangent section) marked with driveway reflectors were set up along the routes investigated on both sides of the roadway. The distance between two spots was 250 feet; thus, the measurement sections were about ½-mi long. On the average, the recommended speed plates were located about 500 feet (0.1 mile) from the curves in the deceleration direction, while in the acceleration direction at this spot the normal speed limit of 55 mph was posted.

The car speeds were measured during off-peak periods of the week, during dry conditions, and in daylight. The traffic flows were light, and cars were capable of attaining the speeds they desired under the conditions of the site; in other words, a car was selected for speed survey if it had sufficient headway to be considered travelling at its own free speed.

With regard to the analysis process, the observer in the follow car observing the cars crossing his field of view had to select the cars to be sampled. Once a car was spotted under free-flow conditions, an initial acceleration by the driver of the follow car was made in order to catch up and adjust his speed to that of the test vehicle. Then, at each of the study spots along the highway, the observer in the follow vehicle would record the speed of the test vehicle by reading the speed from the speedometer of the follow car. Other relevant information, such as the sex and approximate age of the driver of the test vehicle and the type and mark of the test vehicle, were also recorded, but their effect was not considered in this study. A distance of at least one mile was necessary for the follow car to accelerate and adjust its speed to that of the test car.

All conflicts in which evasive action was taken, such as turning maneuvers into driveways before the end of the speed measurements, were recorded, but those measurements were not considered in the analysis.

Normally the speeds of at least twenty passenger cars (test cars) were recorded on the tape recorder at each of the eleven test points along the routes investigated from the tangent to the curve (deceleration) and from the curve to the tangent (acceleration). The data on the tape recorder was later analyzed, and the 85th-percentile speed at each of the set-up test spots was determined.

Regression equations relating the 85th-percentile speeds to distances travelled are as follows:

Acceleration:

Recommended Speed in Curve: 30 mph

$$V85 = 37.0 + 0.05DT - 0.00002DT^2 \tag{3a}$$

Recommended Speed in Curve: 35 mph

$$V85 = 42.0 + 0.04DT - 0.00002DT^2 \tag{3b}$$

Recommended Speed in Curve: 40 mph

$$V85 = 47.0 + 0.04DT - 0.00002DT^2 \tag{3c}$$

Deceleration:

Recommended Speed in Curve: 30 mph

$$V85 = 33.0 + 0.05DT - 0.00002DT^2 \tag{4a}$$

Recommended Speed in Curve: 35 mph

$$V85 = 38.0 + 0.04DT - 0.00002DT^2 \tag{4b}$$

Recommended Speed in Curve: 40 mph

$$V85 = 43.0 + 0.04DT - 0.00002DT^2 \tag{4c}$$

where

$V85$ = estimate of 85th-percentile speed (mph), and
DT = distance travelled (feet).

The above equations are plotted in figures 1 and 2. The acceleration and deceleration processes are clearly indicated to end or begin at about 700 to 750 feet from the end of the observed curved sections. This means that any reaction from

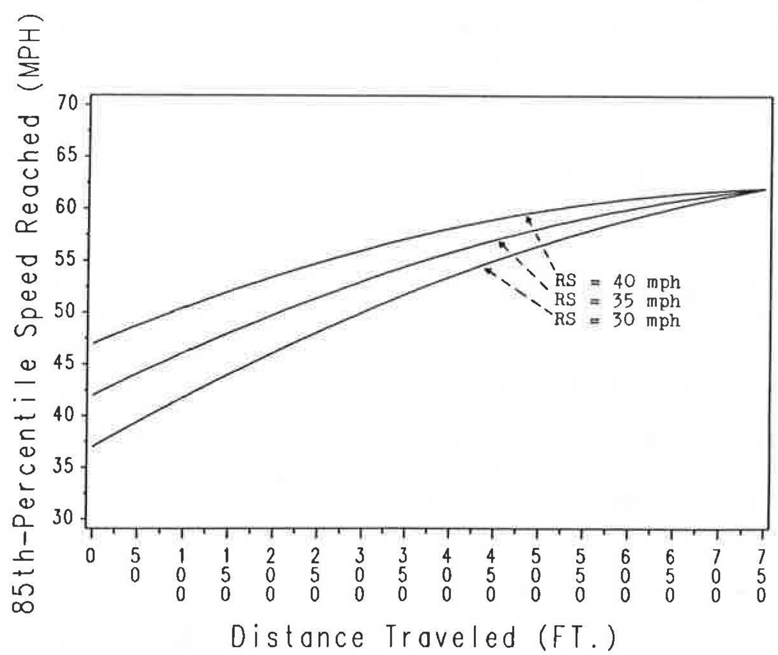


FIGURE 1 85th-Percentile speed vs. distance traveled; passenger cars (acceleration).

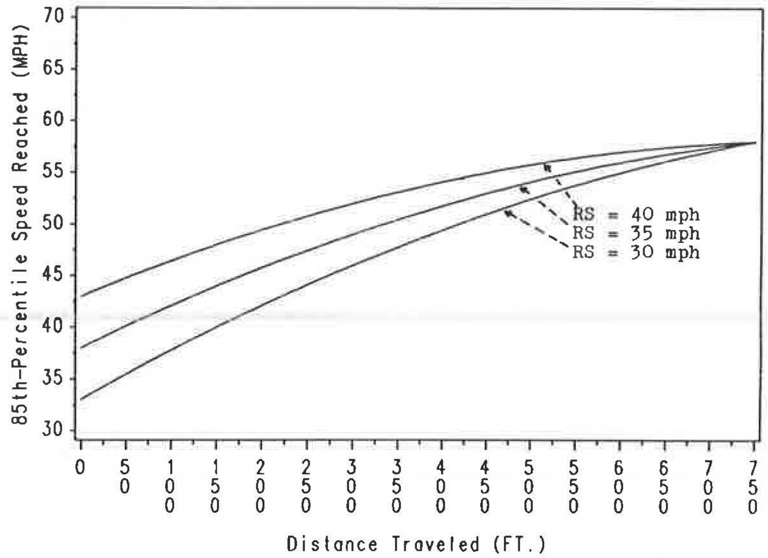


FIGURE 2 85th-Percentile speed vs. distance traveled; passenger cars (deceleration).

the driver in the deceleration direction begins nearly 200 to 250 feet from the recommended speed plates, which are normally posted 500 feet in front of a curve or a curved section. Another finding is that the operating speeds at the beginning of a curve in the deceleration direction are nearly 4 to 5 mph lower (figure 2), than those at the end of the curve in the acceleration direction (figure 1).

Related to the distance of 750 feet, the average deceleration and acceleration rates ranged between 2.8 and 2.9 ft/sec^2 for the tested six road sections consisting of tangents (length of at least ½ mile) followed by curves with recommended speeds between 30 and 40 mph. Since the differences between deceleration and acceleration rates are more or less negligible, an average acceleration or deceleration rate of 2.8 ft/sec^2 was selected for the following analysis. This value agrees well with the deceleration and acceleration rate of 0.8 m/sec^2 (2.64 ft/sec^2) on which the design of transition lengths in the Swiss Standard (*3, 4*) is based. Furthermore, this value agrees well with the values in the AASHTO design guide (*1*), table III-4, where average acceleration rates of about 2.1 ft/sec^2 for passing maneuvers in the speed groups 30 to 40 mph and 40 to 50 mph are tabulated.

DETERMINATION OF NECESSARY TRANSITION LENGTHS (TANGENT LENGTHS)

For traffic safety reasons driving behavior during the deceleration process is a particularly important factor.

As previously outlined, operating speed differences $\Delta V85$ between two successive design elements greater than 6 mph should be avoided for good designs and greater than 12 mph for fair designs. An illustration of the above conclusion is given in figure 3.

With an average acceleration or deceleration rate of a = 2.8 ft/sec^2 the transition length in equation (2) now reads:

$$TL = \frac{\overline{V85} \cdot \Delta V85}{1.302} \tag{5}$$

where

$\overline{V85}$ = average 85th-percentile speed between successive curves (mph),

$\Delta V85$ = difference between the 85th-percentile speeds (mph), and

TL = transition length (tangent length)(in feet).

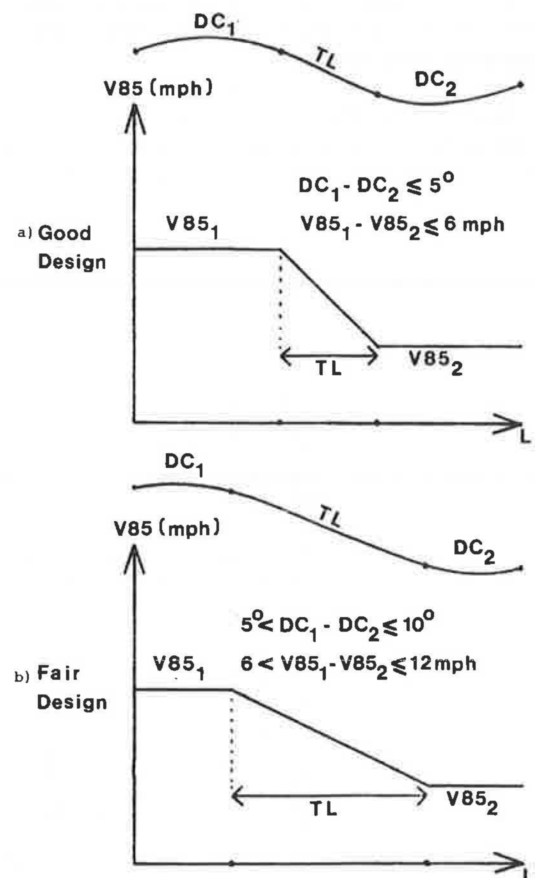


FIGURE 3 Transition length between successive design elements.

TABLE 1 NECESSARY TRANSITION LENGTHS (TANGENT LENGTHS) FOR GOOD AND FAIR DESIGN PRACTICES

$V85_1$ resp. $V85_2$							
22							
28	115*						
34	260	145*					
40	430	315	170*				
46	625	510	370	200*			
52	850	735	595	425	225*		
58	1105	990	850	675	480	255*	
	22	28	34	40	46	52	58
	$V85_2$ (mph) resp. $V85_1$						

* good designs

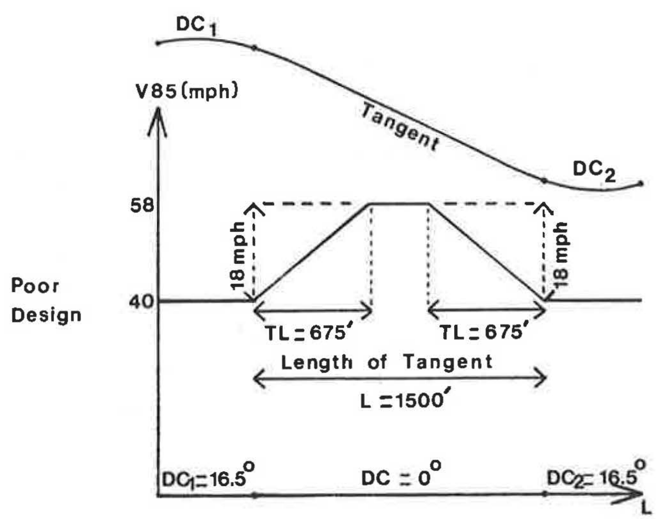
|__| fair designs

Based on the above equation, necessary transition lengths for good and fair design practices are shown in table 1. The values with an asterisk represent good design practices, meaning a driver is able to decelerate or accelerate within the range of operating speed changes of up to 6 mph. The values within boxes represent fair design practices, meaning a driver is able to decelerate or accelerate within the range of operating speed changes of up to 12 mph (see figure 3).

Thus, from the viewpoint of reasonable changes in degree of curve and the corresponding changes in operating speeds, the transition lengths (mostly expressed by tangents) in table 1 should represent maximum boundaries for good and for fair design practices.

In all the other cases (see, for example, the unmarked values in table 1), a driver is able to exceed the recommended operating speed changes, which may result in critical driving maneuvers, especially during the deceleration process.

An illustration of the above statement is given in figure 4 for a sequence of two curves ($DC = 16.5°$) joined by a relatively long tangent ($DC = 0°$, $L = 1{,}500$ ft). The 85th-percentile speeds can be determined from equation 1. As can be seen from figure 4, a driver is able to accelerate within the tangent from an operating speed of 40 mph in the curve to the highest operating speed of 58 mph in the tangent, for which, according to table 1, a transition length of 675 ft is needed. In this example the maximum allowable operating speed change even for fair designs of $\Delta V85 \leq 12$ mph has thus been exceeded, a clear indication of poor design practices.



$\Delta DC = 16.5° > 10°$ (poor design)
$\Delta V85 = 18$ mph > 12 mph (poor design)

FIGURE 4 Example of poor design practices.

RECOMMENDATIONS FOR TANGENTS

The majority of transitions between curves on the two-lane, rural highway network in the United States consist of tangents, with the exception of very good designs where transition curves are applied and operating speed changes exceeding the boundaries for good design or even fair design normally do not exist.

With regard to tangents between succeeding curves the following criteria must be distinguished:

1. The transition lengths (tangent lengths) given in table 1 represent maximum boundaries to allow non-critical deceleration or acceleration movements between successive curves for good or fair designs. In order not to be too conservative, tangent lengths between two successive curves, which fall in the range of fair design practices (table 1) may be considered as non-independent design elements. That means, changes in degrees of curve and operating speeds between two successive curves may be calculated directly without regarding the tan-

gent in-between as an independent design element. By this assumption the most critical case for fair design practices, especially during a deceleration process ($\Delta V85 = 12$ mph, see figure 3b) is covered. In all the other cases ($\Delta V85 < 12$ mph) the tangent lengths are not sufficient for the average driver to decelerate or accelerate in such a way that the assumed boundaries of operating speed changes for fair or even good designs are exceeded.

Note that the values of the transition lengths for fair design practices (table 1) agree well with the lengths of superelevation runoffs provided in table III-14 (*1*) in case of a reversal in alignment, for example, for a maximum superelevation rate of 8 percent.

2. Tangent lengths between successive curves that exceed the values of fair design (table 1) should be regarded as independent design elements. In these cases a driver is able to accelerate or decelerate in such a way that even the maximum allowable operating speed changes for fair designs ($\Delta V85 \leq 12$ mph) may be exceeded; that means, critical driving maneuvers already have originated. Therefore, in case of a relatively long tangent between two successive curves, changes in degrees of curve and operating speeds on this section must be calculated by regarding the tangent in between as an independent design element (see, for example, figure 4).

DESIGN PROCEDURE WITH EXAMPLE APPLICATIONS

The results of table 1 are rounded in table 2, where the values within boxes represent the maximum allowable lengths of tangents regarded as non-independent design elements, as outlined in the previous section. The values with an asterisk represent lengths of tangents for which, related to the speed changes of table 2, 85th-percentile speeds of 58 mph can be reached. As the research of the authors (*11, 15*) has revealed, on long tangents an 85th-percentile speed value of 58 mph is a good estimate for a degree of curve $DC = 0°$, see equation (1). Thus, the maximum operating speed in tangents will be confined in what follows to this value.

To evaluate a tangent between two successive curves as independent and to estimate the expected operating speed in the tangent ($V85_T$), the following procedure is recommended:

(1) Assess the tangent length (TL) between the two successive curves (these may be in the field, as in the case of RRR projects, or in the design stages for new designs, major reconstructions, or redesigns).

(2) Determine for the degree of curve 1 (DC_1) and the degree of curve 2 (DC_2) the corresponding 85th-percentile speeds ($V85_1$ and $V85_2$) by applying equation (1).

(3) Compare the existing tangent length between the two successive curves with the maximum allowable tangent length (from table 2) that corresponds to the nearest 85th-percentile speed of the curve with the higher degree of curve.

(4) Conclude that if the existing tangent length is smaller than the maximum allowable one, then the tangent is to be regarded as non-independent. That means changes in degree of curve and operating speed will be especially related to the two successive curves since the tangent can be assumed to be negligible. Note that the requirements for sufficient lengths of superelevation runoffs should be fulfilled.

TABLE 2 RELATIONSHIP BETWEEN TANGENT LENGTHS AND 85TH-PERCENTILE SPEED CHANGES FOR SEQUENCES: TANGENTS TO CURVES

V85 in Curve	V85 in Tangent				
	34	40	46	52	58
22	$\boxed{250}$	425	625	850	1100*
28		$\boxed{325}$	500	725	1000*
34			$\boxed{375}$	600	850*
40				$\boxed{425}$	675*
≥46					$\boxed{475}$*

$\boxed{}$ Maximum allowable Lengths of Tangents, regarded as "Non-Independent Design Elements", (ft)

V85 = 85th-Percentile speed in curve or tangent (mph)

*For these values the highest operating speed in tangents V85 = 58 mph can be expected.

Example

$TL = 300$ ft,
$DC_1 = 3° \rightarrow \rightarrow \rightarrow V85_1 = 55$ mph,
$DC_2 = 9° \rightarrow \rightarrow \rightarrow V85_2 = 48$ mph, see equation (1).

The 85th-percentile speed in table 2 that is closest to 48 mph in the curve with the higher degree of curve is 46 mph. (This simplification was done for an easier application of table 2.) For 46 mph the maximum length of tangents regarded as non-independent is 475 feet. Since $TL = 300$ feet < 475 feet, the tangent has to be evaluated as non-independent design element, and no individual operating speed ($V85_T$) is to be assigned to the tangent.

Thus, only the sequence curve-to-curve with the corresponding operating speeds ($V85_1$ and $V85_2$) plays an important role in the design process for evaluating horizontal design consistency or inconsistency, since the tangent in between can be assumed to be negligible. For the example discussed a change in degree of curve and operating speed

$\Delta DC = |3° - 9°| = 6°$, and

$\Delta V85 = |55 - 48 \text{ mph}| = 7$ mph

can be expected on the above road section. In conformity with the recommended boundaries for good, fair, and poor design practices, the existing horizontal alignment thus corresponds to fair designs ($\Delta V85 > 6$ mph).

(5) Conclude that if the existing tangent length between successive curves is greater than the maximum allowable (table 2), then the tangent is to be regarded as an independent design element. That means, changes in degree of curve and operating speed are to be especially related to the sequence tangent-to-curve. The 85th-percentile speed in the tangent ($V85_T$) can be estimated as outlined in the following examples, see figure 5.

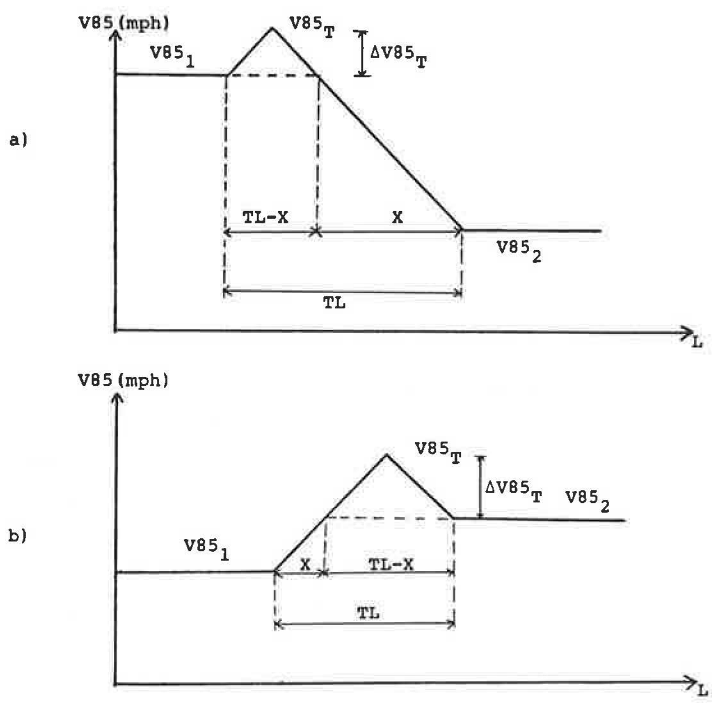
Example Related to Figure 5a

$TL = 0.20$ mi $\sim$ 1,050 ft,

$DC_1 = 6° \rightarrow \rightarrow \rightarrow V85_1 = 52$ mph,

$DC_2 = 22.4° \rightarrow \rightarrow \rightarrow V85_2 = 33$ mph, see equation (1).

The 85th-percentile speed in table 2 that is closest to 33 mph in the curve with the higher degree of curve is 34 mph. For 34 mph the maximum length of tangents regarded as non-independent is 375 feet.



Legend:

TL = Tangent length, greater than the maximum allowable lengths for "Non-Independent Tangents" of Table 2 (ft),
X = Acceleration or deceleration distance between curve 1 and curve 2 (ft),
$V85_1$, $V85_2$ = Operating speeds in curves (mph),
$V85_T$ = Operation speed in tangent (mph),
$\Delta V85_T$ = Difference between the operating speed in the curve with the lower degree of curve and the operating speed in the tangent (mph).

FIGURE 5 Typical examples for estimating operating speed in independent tangents.

Since $TL = 1050$ feet > 375 feet the tangent has to be evaluated as an independent design element. Thus, the sequence tangent-to-curve plays an important role in the design process for evaluating horizontal design consistency or inconsistency for both directions of travel on this road section. The 85th-percentile speed in the tangent ($V85T$) can be estimated as shown below (see figure 5a).

Equation (5) is used to calculate the acceleration or deceleration distance (X) between curve 1 and curve 2. This implies

$$X = \frac{\overline{V85} \cdot \Delta V85}{1.302} \tag{5a}$$

$$X = \frac{42.5 \cdot 19}{1.302} = 620 \text{ ft}$$

Then, the remaining tangent length is

$$TL - X = 1050 - 620 = 430 \text{ feet}$$

along which a driver is able to perform additional acceleration or deceleration maneuvers. (Exceptional case: $DC_1 = DC_2 \rightarrow\rightarrow\rightarrow V85_1 = V85_2$ $X = 0$; perform the calculations in the same way with $X = 0$). By transforming equation (5), in order to calculate the difference $\Delta V85_T$ between the operating speed in the curve with the lower degree of curve ($V85_1$) and the estimated operating speed in the tangent ($V85_T$), the formula now becomes (see figure 5a):

$$\frac{[V85_1 + (V85_1 + \Delta V85_T)] \cdot \Delta V85_T}{2 \cdot 1.302} = \frac{(TL - X)}{2}$$

or

$$\Delta V85_T = \frac{-2 \cdot (V85_1) \pm \sqrt{4(V85_1)^2 + 5.208(TL - X)}}{2} \tag{6}$$

It follows that

$$\Delta V85_T = \frac{-2 \cdot (52) \pm \sqrt{4(52)^2 + 5.208(430)}}{2} \approx 5 \text{ mph}$$

Thus, the operating speed in the independent tangent for evaluating the sequences tangent-to-curve in both directions of travel becomes $V85_T = V85_1 + \%V85_T = 52 + 5 = 57$ mph.

For the discussed example the following changes in degrees of curve and operating speeds can be expected between

tangent to curve 1:
$\Delta DC = |0° - 6°| = 6°$,
$\Delta V85 = |57 - 52 \text{ mph}| = 5$ mph, and
tangent to curve 2:
$\Delta DC = |0° - 22.4°| = 22.4°$,
$\Delta V85 = |57 - 33 \text{ mph}| = 24$ mph.

The changes in operating speeds reveal that the sequence independent tangent-to-curve 1 corresponds to good design practices ($\Delta V85 < 6$ mph), while the sequence independent tangent-to-curve 2 corresponds to poor design practices ($\Delta V85 > 12$ mph). In the event the calculated 85th-percentile speed in the independent tangent exceeds the value of 58 mph, it is recommended that the 85th-percentile speed in the examined tangent be confined to this value. As previously mentioned, 58 mph is a good estimate for the 85th-percentile speed in long tangents for the nationwide speed limit of 55 mph on two-lane, rural roads.

Example Related to Figure 5b

$TL = 0.15$ mi $= 790$ ft,

$DC_1 = 27° \rightarrow\rightarrow\rightarrow V85 = 28$ mph,

$DC_2 = 22.4° \rightarrow\rightarrow\rightarrow V85 = 33$ mph, see equation (1).

The 85th-percentile speed in table 2 that is closest to 28 mph in the curve with the higher degree of curve corresponds exactly to 28 mph. For 28 mph the maximum length of tangents that is regarded as non-independent is 325 feet. Since $TL = 790$ feet < 325 feet, the tangent has to be evaluated as an independent design element.

The 85th-percentile speed in the tangent, related to figure 5b, can be estimated in the same way as discussed in the previous example.

According to equation (5a), the acceleration or deceleration distance between curve 1 and curve 2 is as follows:

$$X = \frac{30.5 \cdot 5}{1.302} = 117 \text{ ft.}$$

Therefore, the remaining tangent length becomes

$$TL - X = 790 - 117 = 673 \text{ feet}$$

According to equation (6), the difference between the operating speed in the curve with the lower degree of curve and the operating speed in the tangent now becomes

$$\Delta V85_T = \frac{-2(33) \pm \sqrt{4(33)^2 + 5.208(673)}}{2} \approx 11 \text{ mph}$$

Note that for the example of figure 5b curve 2 is the curve with the lower degree of curve.

It follows that the operating speed in the independent tangent is

$$V85_T = V85_2 + \Delta V85_T = 33 + 11 = 44 \text{ mph.}$$

For the discussed example the following changes in degrees of curve and operating speeds can be expected between

tangent to curve 1:
$\Delta DC = |0° - 27°| = 27°$,
$\Delta V85 = |44 - 28 \text{ mph}| = 16$ mph, and

tangent to curve 2:
$\Delta DC = |0° - 22.4°| = 22.4°$,
$\Delta V85 = |44 - 33 \text{ mph}| = 11$ mph.

The changes in operating speeds reveal that the sequence independent tangent-to-curve 1 corresponds to poor design practices ($\Delta V85 > 12$ mph), while the sequence independent tangent to curve 2 can be still evaluated as fair design ($\Delta V85 < 12$ mph).

(6) The calculations of step (5) must not be performed on long tangents between two successive curves. The length of those tangents must be at least twice as high as the values listed in the last column of table 2, related to the nearest 85th-percentile speed of the curve with the higher degree

of curve. In these cases, it can be assumed without any further calculation that the tangents are independent, and that an operating speed of 58 mph is a good estimate on those long tangents.

A typical example for such a case is shown in figure 4.

Example Related to Figure 4

$TL = 1500$ ft,

$DC_1 = 16.5° \rightarrow\rightarrow\rightarrow V85_1 = 40$ mph,

$DC_2 = 16.5° \rightarrow\rightarrow\rightarrow V85_2 = 40$ mph, see equation 1.

The 85th-percentile speed in table 2 that is closest to 40 mph is exactly 40 mph. To accelerate or decelerate from 40 mph to the highest operating speed of 58 mph in the tangent a distance of 675 feet is needed (compare corresponding value in the last column of table 2): $2 \cdot 675 = 1{,}350$ feet $< 1{,}500$ feet. Thus, it can be concluded that the tangent is independent and an operating speed of $V85_T = 58$ mph is a good estimate in the long tangent. For the example the following change in degree of curve and operating speed can be expected for this road section:

$\Delta DC = | 0° - 16.5° | = 16.5°$,

$\Delta V85 = | 58 - 40 \text{ mph} | = 18$ mph.

It follows that the existing horizontal alignment corresponds to poor design practices since $\Delta V85 > 12$ mph.

CONCLUSION

Several countries have limitations on maximum and minimum tangent lengths between curves. The procedure presented above is a rational method to set tangent guidelines for U.S. practice and to provide recommendations for transition lengths (tangent lengths) between successive curved roadway sections for

- tangents that should be regarded as non-independent design elements; that is, the sequence curve-to-curve is the most important element of the design process, and
- tangents that should be regarded as independent design elements; that is, the sequence tangent-to-curve is the most important element of the design process.

The method can be used for new design as well as evaluating in-place roadways in need of safety upgrades.

ACKNOWLEDGMENTS

This study was sponsored by the New York State Governor's Traffic Safety Committee, Albany, New York and by the National Science Foundation. The senior author wishes to thank the members of the TRB Geometric Design Committee for their encouragement and their valuable discussions. Special thanks go to Anand Paluri for his support in elaborating this publication.

REFERENCES

1. *A Policy on Geometric Design of Highways and Streets.* AASHTO, Washington, D.C., 1984.
2. *Guidelines for the Design of Roads,* (RAS-L-1). Committee on Geometric Design Standards, German Road and Transportation Research Association, Edition 1984.
3. R. Lamm and J. G. Cargin. *Translation of the Guidelines for the Design of Roads* (RAS-L-1), Federal Republic of Germany, and the Swiss Norm SN 640080a, *Highway Design, Fundamentals, Speed as a Design Element,* 1981, as discussed by K. Dietrich, M. Rotach, and E. Boppart in *Road Design,* ETH Zuerich, Institute for Traffic Planning and Transport and Technique, Edition 1983, Federal Highway Administration, Washington, D.C., May 1985.
4. *Highway Design, Fundamentals, Speed as a Design Element.* Swiss Association of Road Specialists (VSS), Swiss Norm SN 640080a, Edition 1981.
5. *Standard Specifications for Geometric Design of Rural Roads.* National Swedish Road Administration, Sweden, Edition 1982.
6. *Instruction sur les Conditions Techniques D'Aménagement de Routes Nationales.* Ministère de l'Equipement et du Logement, France, Edition 1975.
7. *Highway Link Design, Geometric Alignment Standards.* Departmental Standard TD9/81, Department of Transport, Great Britain, 1981.
8. R. Lamm, J. C. Hayward, and J. G. Cargin. Comparison of Different Procedures for Evaluating Speed Consistency. In *Transportation Research Record 1100,* TRB, National Research Council, Washington, D.C., 1986.
9. R. Lamm and J. G. Cargin. Identifying Operating Speed Inconsistencies on Two-Lane Rural Roads. *Proceedings of the Thirty-Ninth Annual Ohio Transportation Engineering Conference,* conducted by the Department of Civil Engineering, The Ohio State University in Cooperation with The Ohio Department of Transportation, December 1985, pp. 13–22.
10. J. Hayward, R. Lamm, and A. Lyng. *Survey of Current Geometric and Pavement Design Practices in Europe, Part: Geometric Practices,* International Road Federation, Washington, D.C., July 1985.
11. R. Lamm and E. M. Choueiri. Relationship Between Design, Driving Behavior, and Accident Risk on Curves. *Proceedings of the Fortieth Annual Ohio Transportation Engineering Conference,* conducted by the Department of Civil Engineering, The Ohio State University in Cooperation with the Ohio Department of Transportation, December 1986, pp. 87–100.
12. R. Lamm and E. M. Choueiri. Recommendations for Evaluating Horizontal Design Consistency Based on Investigations in the State of New York. In *Transportation Research Record 1122,* TRB, National Research Council, Washington, D.C., 1987, pp. 68–78.
13. E. M. Choueiri. *Statistical Analysis of Operating Speeds and Accident Rates on Two-Lane Rural State Routes.* Ph.D. Dissertation, Clarkson University, January, 1987.
14. R. Lamm, R. and E. M. Choueiri. *A Design Procedure to Determine Critical Dissimilarities in Horizontal Alignment and Enhance Traffic Safety by Appropriate Low-Cost or High-Cost Projects.* Report for the National Science Foundation, Washington, D.C., March 1987.
15. E. M. Choueiri and R. Lamm. *Operating Speeds and Accident Rates on Two-Lane Rural Highway Curved Sections—Investigations about Consistency and Inconsistency in Horizontal Alignment.* Part I of the Research Contract "Rural Roads Speed Inconsistencies Design Methods," State University of New York Research Foundation, Albany, New York, July 1987.

Publication of this paper sponsored by Committee on Geometric Design.

New and Improved Model of Passing Sight Distance on Two-Lane Highways

JOHN C. GLENNON

A mathematical model is derived for describing the critical nature of the passing maneuver on two-lane highways. This model is based on the hypothesis that a critical position exists during the passing maneuver where the passing sight distance requirements to either complete or abort the pass are equal. At this point, the decision to complete the pass will provide the same head-on clearance to an opposing vehicle as will the decision to abort the pass. Current highway practice in both designing and marking for passing sight distance uses a model that assumes that once a driver starts a pass, he must continue until the pass is completed. In other words, the model assumes that the driver has no opportunity to abort the pass. Because this hypothesis is unrealistic, the model derived here is recommended for determining new passing sight distance requirements for both designing and marking passing zones. Suggested values are given for these requirements. A brief analysis is also presented of the sensitivity of passing sight distance requirements to vehicle length. This analysis shows that the effect of truck length is not as dramatic as previously reported in the literature.

Although significant advances have been made since 1971 in understanding the critical aspects of the passing maneuver on two-lane highways, the highway community still clings to false and archaic principles. Actually in the current practice for both the design and the marking of passing zones, these zones are neither designed nor marked directly. Current marking practice in the 1978 *Manual of Uniform Traffic Control Devices* (MUTCD) (*1*), for example, is concerned with no-passing zones, and passing zones merely happen where no-passing zones are not warranted. In highway design, the current practice is stated in the 1984 *Policy on Geometric Design of Streets and Highways* (*2*) by the American Association of State Highway and Transportation Officials (AASHTO). In AASHTO policy, which has remained unchanged since 1954, the design of passing sight distance (PSD) only considers the percentage of highway that has PSD, regardless of whether that PSD forms passing zones of adequate length.

Another inconsistency exists in that, although the AASHTO design and MUTCD marking practices are based on the same hypothetical model, they use completely different criteria to exercise that model. Whereas the current AASHTO practice assumes a 10-mph speed differential between passing and impeding cars for all design speeds, the MUTCD practice comes from the 1940 AASHO policy (*3*), which used speed differentials ranging from 10 mph at a 30-mph design speed to 25 mph at a 70-mph design speed.

John C. Glennon, Chartered, 8340 Mission Road, Suite B-12, Prairie Village, Kans. 66206.

Besides the inconsistencies already discussed, the basic hypothesis underlying both current PSD design and PSD marking practices is flawed. Although this hypothesis correctly considers the opposing vehicle and the final head-on separation distance as integral components of the critical passing maneuver, it determines overly long PSD requirements by assuming that the passing driver has no opportunity to abort the maneuver.

This paper first addresses the development of a more appropriate model for PSD requirements. With this model developed, the paper then focuses both on the application of the model to proper highway design and marking practices and also on the sensitivities of PSD requirements to vehicle length.

RESEARCH SINCE 1971

In 1971, Weaver and Glennon (*4*) and Van Valkenberg and Michael (*5*) independently recognized that the AASHTO model (*2–3, 6–7*) for PSD fails to address the critical nature of the passing maneuver. These studies also both recognized that a safe passing maneuver not only requires continuously varying amounts of PSD (depending on the lesser of the needs for completing or aborting the maneuver), but also has a relative position between the passing and impeding vehicles where the ability to complete the pass is equal to the ability to abort the pass. Weaver and Glennon called this the critical position, and Van Valkenberg and Michael called it the point of no return. Neither study, however, attempted to mathematically define this critical position.

In 1976, Harwood and Glennon (*8*) attempted to better explain the state-of-the-art concerning PSD requirements. This paper contributed further definition of the critical position as that point where the PSD needed to complete the pass is equal to the PSD needed to abort the pass. As shown in figure 1, the pass starts with a minimal PSD needed to abort, the PSD increases through the maneuver until the PSD needed for either completing or aborting the maneuver is equal, and then the PSD decreases through the remainder of the maneuver based on the temporal needs for pass completion.

Lieberman (*9*), in 1982, added further insight by developing a mathematical time-distance model that identified the critical position and the critical PSD as a function of design speed. however, he incorrectly concluded that AASHTO requirements for PSD were inadequate by calculating his PSD requirements as the sum of both the critical PSD and the distance needed for the passing vehicle to get from the initial trailing position to the critical position. His model also ignored the direct effects of vehicle length and the elapsed time for

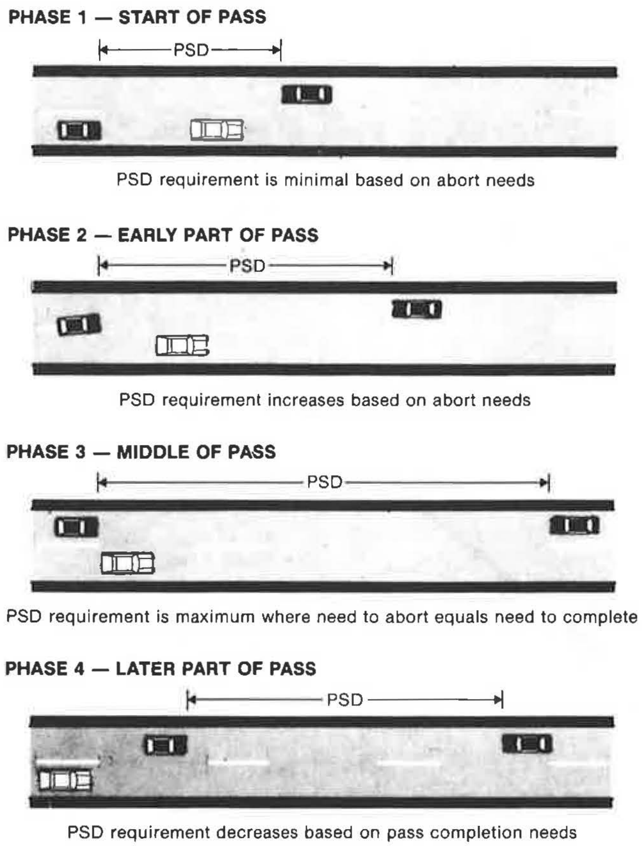


FIGURE 1 Four phases of a passing maneuver.

perception-reaction in the abort maneuver. Regardless of these shortcomings, the Lieberman formulation was conceptually correct and, as such, provided the inspiration for the model developed in this paper.

Saito (*10*), in 1983, re-emphasized the importance of the abort maneuver in determining PSD requirements. To that date, his modeling came closest to determining true PSD needs. However, he looked only at the needs of the abort maneuver and ignored the trade-offs between the completed and abort maneuvers. In other words, rather than calculating the critical position, he assumed that position was where the passing vehicle is immediately behind the impeding vehicle. As indicated later, this assumption gives PSD requirements that are not too different from those found by using a critical position calculated as a function of design speed.

DERIVATION OF A CRITICAL PASSING MODEL

Figure 2 shows time-space diagrams for both the completed passing maneuver and the aborted passing maneuver from the critical position where the PSD needed for safe completion equals the PSD needed for safe abortion. If an opposing vehicle appears before the passing vehicle reaches the critical position, the PSD needed to abort the pass is less than the PSD needed at the critical position. Likewise, if an opposing vehicle appears after the passing vehicle reaches the critical position, the PSD needed to complete the pass is less than the PSD needed at the critical position. Therefore, the maximum or critical PSD is that needed at the critical position.

The proposed model assumes that the opposing vehicle travels at the design speed, that the passing vehicle accelerates to the design speed at or before the critical position and continues at that speed unless the pass is aborted, and that the impeding vehicle travels at a constant speed at some increment less than the design speed.

Since the initial part of the pass is of no consequence in determining the critical sight distance, *Sc*, figure 2 starts the passing vehicle at the critical position and equates the two possible maneuvers in time and space. The sub-model for the completed pass assumes that each vehicle maintains a constant speed and that at the end of the pass there is an acceptable clearance, *C*, between passing and opposing vehicles and an acceptable gap, *G*, between passing and impeding vehicles. For the aborted pass, the impeding and opposing vehicles maintain their constant speeds, but the passing vehicle after a one-second driver perception-reaction time decelerates at rate, *d*, until it achieves an acceptable gap, *G*, behind the impeding vehicle and an acceptable head-on clearance, *C*. [Note that the one-second perception-reaction time is also a part of the completed pass time, but can be ignored in this part of the analysis because it does not affect any of the key time-distance parameters.]

To develop a usable model for the critical PSD requires simultaneous solutions of equations for both the completed and aborted passes, knowing by definition that their critical positions and critical sight distances are equal. The following sections illustrate the development of this model.

Equate Critical Positions

The critical position for the completed pass is shown on figure 2A as:

$$\Delta_c + vt_1 = L_p + G + (v - m)t_1$$

or

$$\Delta_c = L_p + G - mt_1 \tag{1}$$

The critical position for the aborted pass is shown on figure 2B as:

$$\Delta_c' + v + vt_2 - \frac{dt_2^2}{2} = (v - m) + (v - m)t_2 - G - L_i$$

or

$$\Delta_c' \frac{dt_2^2}{2} - m(t_2 + 1) - G - L_I \tag{2}$$

Since by definition $\Delta_c = \Delta_c$, Equations 1 and 2 can be solved simultaneously for t_1, as follows:

$$t_1 = t_2 + 1 - \frac{dt_2^2}{2m} + \frac{(2G + L_i + L_p)}{m} \tag{3}$$

Equate Critical Sight Distances

The critical PSD for each maneuver is taken directly from figure 2 as the total distance between passing and opposing vehicles when the passing vehicle is in the critical position.

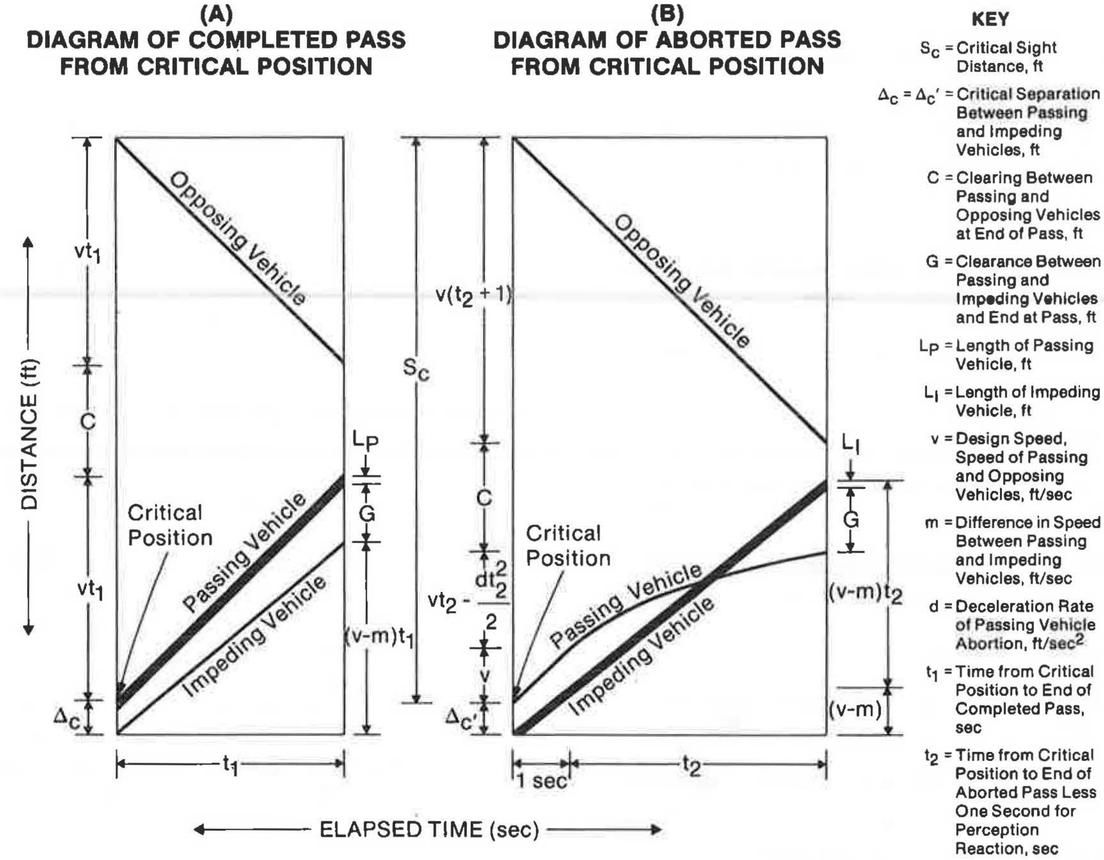


FIGURE 2 Time-space diagrams for the critical passing maneuver.

Equating these distances and solving for t_1 gives:

$$2vt_1 + C = v + vt_2 - \frac{dt_2^2}{2} + C + v(t_2 + 1)$$

$$t_1 = t_2 + 1 - \frac{dt_2^2}{4v} \tag{4}$$

Solve Time Relationships

By simultaneous solution of Equations 3 and 4, t_2 can be isolated as a function of definable parameters as follows:

$$t_2 + 1 - \frac{dt_2^2}{4v} = t_2 + 1 - \frac{dt_2^2}{2m} + \frac{(2G + L_I + L_p)}{m}$$

or

$$t_2 = \sqrt{\frac{4v(2G + L_I + L_p)}{d(2v - m)}} \tag{5}$$

since

$$t_1 = t_2 + 1 - \frac{dt_2^2}{4v}$$

then

$$t_1 = 1 + \sqrt{\frac{4v(2G + L_I + L_p)}{d(2v - m)}} - \frac{(2G + L_I + L_p)}{2v - m} \tag{6}$$

Solve the Critical Position

Equations 1 and 6 can be solved simultaneously to derive an expression for the critical position as a function of design speed, v, speed difference, m, desired gap, G, deceleration rate, d, and lengths of vehicles, L_I and L_p, as follows:

$$\Delta_c = L_p + G - mt_1$$

or

$$\Delta_c = L_p + G - m + m\left[\frac{(2G + L_I + L_p)}{2v - m} - \sqrt{\frac{4v(2G + L_I + L_p)}{d(2v - m)}}\right] \tag{7}$$

Assuming a minimum acceptable headway of one second for G, then $G = m$ and Equation 7 is revised as follows:

$$\Delta_c = L_p + m\left[\frac{(2m + L_I + L_p)}{2v - m} - \sqrt{\frac{4v(2m + L_I + L_p)}{d(2v - m)}}\right] \tag{8}$$

[Note that the same relationship is found if, in Figure 2, the passing vehicle is assumed to be behind the impeding vehicle at the critical position.]

Solve the Critical Passing Sight Distance

Using Figure 2 and Equation 1, the passing sight distance, S_c, can be solved for any design speed as a function of the critical position, Δ_c, speed differential, m, and length of passing vehicle, Lp, as follows:

$$S_c = 2vt_1 + C$$

and

$$t_1 = \frac{L_p + G - \Delta_c}{m}$$

therefore

$$S_c = \frac{C + 2v(L_p + G - \Delta_c)}{m}$$

Having already assumed $G = m$ and also assuming a minimum acceptable head-on clearance of one second, then $C = 2v$. Therefore:

$$S_c = 2v + \frac{2v(L_p + m - \Delta_c)}{m}$$

or

$$S_c = 2v\left[2 + \frac{L_p - \Delta_c}{m}\right] \tag{9}$$

PASSING SIGHT DISTANCE REQUIREMENTS

Now that a usable model has been developed for the critical PSD, the question remains how to apply it to the design and marking of a passing zone. Obviously, S_c defines the minimum PSD required for any part of the passing zone where a passing vehicle can reach the critical position. As a worst-case scenario, it seems appropriate to provide S_c at the end of a passing zone, assuming that it is reasonable to expect the critical situation at this point. It is not reasonable, however, to expect that the passing vehicle will be in the critical position at the beginning of a passing zone. Actually the PSD requirement at the beginning of the zone is something less than S_c; however, because passing operations vary widely by speed differentials, opposing vehicle speeds, and vehicle lengths, an added safety factor would be incorporated by starting the passing zone where S_c first becomes available.

Recognizing that the assumptions used to develop the critical passing model may be subject to some interpretation and adjustment, this section provides recommendations for PSD requirements based on the following additional assumptions:

1. The AASHTO use of passenger cars for the passing and impeding vehicles are appropriate criteria.
2. The length of the average passenger car is 16 feet.
3. A reasonably safe deceleration rate in the abort maneuver is 8 ft/sec^2.
4. Based on the Weaver and Glennon study (*4*), the following table of critical (15th percentile) speed differentials is appropriate:

Design Speed (mph)	*Speed Differential (mph)*
30	12
40	11
50	10
60	9
70	8

Substituting Assumptions 1 through 3 into Equations 8 and 9, the critical passing model is reduced to relationships that are a function of the design speed and the speed differential as follows:

$$S_c = 2v\left[2 + \frac{16 - \Delta_c}{m}\right]$$

where

$$\Delta_c = 16 + m\left[\frac{(2m + 32)}{2v - m} - \sqrt{\frac{v(2m + 32)}{2(2v - m)}}\right]$$

Using these equations and solving for the design relationships found under Assumption 4 above, table 1 shows the derived PSD requirements. In comparing these recommendations with current AASHTO and MUTCD requirements, they are found to be considerably less than the AASHTO requirements, but very close to the MUTCD requirements (even though the MUTCD requirements were derived with a completely different set of models and criteria.)

Although this paper does not analyze the requirements for passing zone length, previous studies (*4, 11*) have shown that very short zones, such as the 400-ft default length allowed by the MUTCD, are not appropriate for safe highway operations. Therefore, the recommendations of Weaver and Glennon (*4*) for minimum passing zone length, based on 85th percentile passing vehicle distances, should be implemented unless another rationale is shown to be more appropriate. These passing zone lengths are also shown in table 1.

TRUCK LENGTH CONSIDERATIONS

Several authors (*9, 12–14*) have expressed alarm at the supposed inadequacy of PSD requirements (most particular AASHTO requirements) for passes involving trucks in general and longer trucks, in particular. These studies were dramatized by Donaldson (*15*) as follows:

> The recent research of Lieberman demonstrates the thorough inadequacy of the AASHTO sight distance formulae for the successful execution of the passing maneuver . . . . Lieberman has shown that significantly longer sight distances are needed when the impeding vehicle is a truck . . . . The research of Gericke and Walton demonstrates that the AASHTO sight distance formulae for geometric design are inadequate for any vehicle and especially inadequate for cars passing trucks . . . . Saito shows that successful aborts are impossible under most high-speed conditions on the basis of current MUTCD standards . . . . If one extrapolates his kinematic model, it shows substantial increases in the lengths of time and distances for successful aborts of cars attempting to pass longer trucks . . . . The passenger car/truck relationship in the passing maneuver is highly dangerous on many thousands of our rural arterial and collector routes that have inadequate sight distance but which are marked to permit passing maneuvers that cannot be accomplished by most of the vehicles making the attempts.

The flaw in the remarks quoted above is that none of the studies cited by Donaldson were based on a correct analysis of passing sight distance requirements. Of the sources cited, Lieberman (*9*) failed to correctly apply his own insights on the definition of the critical sight distance, Saito (*10*) ignored the trade-offs between completed and aborted passes, and Gericke and Walton (*12*) used the [incorrect] AASHTO model to derive their results, as did Fancher (*13*) and Khasnabis (*14*).

Table 2 shows the sensitivity of the derived PSD requirements to vehicle length. As can be seen, the PSD requirements increase as a function of vehicle length but not as dramatically as previously stated in the literature.

Whether a truck should be considered as a design vehicle

TABLE 1 DERIVED PASSING SIGHT DISTANCE REQUIREMENTS

Design Speed (mph)	Critical Position: Front of passing vehicle relative to front of impeding vehicle (ft)	Maximum Abort Position: Front of passing vehicle relative to front of impeding vehicle (ft)	Minimum Length of Passing Zone (Ref. 4)	PSD Requirement (ft)
40	-43	-10	600	670
50	-38	-10	900	830
60	-32	-8	1200	990
70	-25	-5	1500	1140

TABLE 2 DERIVED PASSING SIGHT DISTANCE REQUIREMENTS AS A FUNCTION OF PASSED VEHICLE LENGTH

Design Speed (mph)	Rounded PSD Requirements for Various Passed Vehicle Lengths (ft)*			
	Passenger Car	55-ft. Truck	65-ft. Truck	110-ft. Truck
40	670	760	780	850
50	830	960	980	1080
60	990	1150	1180	1320
70	1140	1320	1380	1550

* Uses passenger car for passing vehicle

for PSD is a moot point, considering, first, that the vehicle length is really only critical for an end-zone pass and, second, that passing drivers have adaptive behavior that considers not only their position in the zone but the vehicle length to be passed.

CONCLUSIONS

The current AASHTO (*2*) model for passing sight distance requirements ignores the possibility of an aborted maneuver and thereby determines overly long distances. This paper derives a more appropriate model that considers the trade-offs between aborted and completed passes. The passing sight distance requirements derived with this model are considerably less than the AASHTO requirements but are surprisingly close to those presented in the MUTCD (*1*). Application of the derived model also shows that the effect of truck length is not as dramatic as previously reported in the literature.

The derived model should be used to revise both the AASHTO and MUTCD practices so that a correct and consistent basis is used for both the design and marking of passing zones. In doing so, the assumption of a one-second, head-on clearance; a one-second gap; an 8-ft/sec^2 deceleration; and a 15th-percentile speed differential should all be questioned. However, because the critical condition addresses only the infrequent pass at the end of a zone, care should be exercised in being overly conservative in selecting these values. For example, the one-second, head-on clearance and one-second gap seem short but may be reasonable considering the rarity of a [small] 15th-percentile speed differential and a [relatively low] 8-ft/sec^2 abort deceleration.

REFERENCES

1. *Manual on Uniform Traffic Control Devices*. Federal Highway Administration, Washington, D.C., 1978.
2. *A Policy on Geometric Design of Highways and Streets*. AASHTO, Washington, D.C., 1984.
3. *A Policy on Criteria for Marking and Signing No-Passing Zones on Two and Three Lane Roads*. AASHTO, Washington, D.C., 1940.
4. G. D. Weaver and J. C. Glennon. *Passing Performance Measurements Related to Sight Distance Design*. Research Report 134-6, Texas Transportation Institute, July 1971.
5. G. W. Van Valkenberg and H. L. Michael. Criteria for No-Passing Zones. *Highway Research Record 366*, HRB, National Research Council, Washington, D.C., 1971.
6. *A Policy on Geometric Design of Rural Highways*. AASHTO, Washington, D.C., 1954.
7. *A Policy on Geometric Design of Rural Highways*. AASHTO, Washington, D.C., 1965.
8. D. W. Harwood and J. C. Glennon. Framework for Design and Operation of Passing Zones on Two-Lane Highways. In *Transportation Research Record 601*, TRB, National Research Council, Washington, D.C., 1977.
9. E. B. Lieberman. Model for Calculating Safe Passing Distances on Two-Lane Rural Roads. In *Transportation Research Record 869*, TRB, National Research Council, Washington, D.C., 1982.
10. M. Saito. Evaluation of the Adequacy of the MUTCD Minimum Passing Sight Distance Requirement for Aborting the Passing Maneuver. *ITE Journal*, January 1984.
11. J. R. Jones. *An Evaluation of the Safety and Utilization of Short Passing Sections*. Master's thesis, Texas A & M University, 1970.
12. O. F. Gericke and C. M. Walton. Effect of Truck Size and Weight on Rural Roadway Geometric Design (and Redesign) Principles and Practices. In *Transportation Research Record 806*, TRB, National Research Council, Washington, D.C., 1981.
13. P. S. Fancher. Sight Distance Problems Related to Large Trucks. In *Transportation Research Record 1052*, TRB, National Research Council, Washington, D.C., 1986.
14. S. Khasnabis. Operational and Safety Problems of Trucks in No-Passing Zones on Two-Lane Highways, In *Transportation Research Record 1052*, TRB, National Research Council, Washington, D.C., 1986.
15. G. A. Donaldson. Large Truck Safety and the Geometric Design of Two-Lane, Two-Way Roads. *ITE Journal*, September 1985.

Publication of this paper sponsored by the Committee on Geometric Design.

Abridgment

Development of Limiting Velocity Models for the Highway Performance Monitoring System

GARY E. ELKINS AND JEREMY SEMRAU

A study was performed for the Federal Highway Administration to increase the efficiency of vehicle speed models for the Highway Performance Monitoring System analytical process. Probabilistic and deterministic models developed by the World Bank were adapted for conditions in the United States. These models estimate vehicle average travel speed as a function of relevant road and traffic characteristics. This is done by evaluating a set of constraining speed models that consider the influence of vertical grades, horizontal curves, roughness, traffic congestion, and highway type. These models were adapted to conditions in the United States using engineering judgment and limited available data. Although further research is needed to refine these models, the models produce reasonable results and are recommended for use in planning models as a basis for computation of road user costs. More research is needed in this general area from the engineering community. Input from experts in vehicle mechanics, dynamics, and human factors would be particularly helpful in determining driver reactions and behavior and further developing speed prediction models as a function of road characteristics and vehicle class.

The Federal Highway Administration (FHWA) uses a set of approximately 92,000 annually monitored sample pavement sections across the United States to assess the condition of the nation's highways and road network. This system is called the Highway Performance Monitoring System (HPMS). One part of the HPMS analysis package is used for planning purposes to study the impacts of different funding scenarios on highway users. A complex, time-consuming, computer algorithm is presently used to estimate vehicle speeds on each sample section from which travel time, fuel consumption, and vehicle operating cost impacts are computed. This paper summarizes the development of an efficient speed prediction model for use in the HPMS analytical process (*1*) and discusses problems encountered in development of the model.

LIMITING VELOCITY MODELS

Limiting velocity models developed by the World Bank were chosen for adaptation to United States conditions (*2*). Using the results of past studies and engineering judgement, the following limiting velocity models were formulated. One model relates a vehicle speed to the minimum of five constraining speeds:

$$Vss = \text{Min}\,(VDRIVE, VBRAKE, VCURVE, VROUGH, VDESIR) \quad (1)$$

where:

Vss = steady state speed,
$VDRIVE$ = maximum possible driving speed,
$VBRAKE$ = maximum allowable braking speed on downgrades,
$VCURVE$ = maximum allowable speed on horizontal curves,
$VROUGH$ = maximum allowable ride severity speed
$VDESIR$ = desired speed.

This model is called the Minimum Limiting Velocity Model (MLVM).

The second model treats each constraining speed as a random variable. This model, called the Probabilistic Limiting Velocity Model (PLVM), is:

$$Vss = \exp\,(S^2/2)/[(1/VDRIVE)^{1/B} + (1/VCURVE)^{1/B} + (1/VBRAKE)^{1/B} + (1/VROUGH)^{1/B} + (1/VDESIR)^{1/B}]^B \quad (2)$$

where:

S^2 = variance associated with unmeasured vehicle, road, and speed measurement characteristics,
B = a constant parameter for each vehicle class.

The PLVM has several interesting features. When two or more speeds become equally dominant, the probabilistic speed drops below the deterministic speed by a larger amount. Also, as more speeds begin to lower the probabilistic speed, they do so at a diminishing rate. Thus, the stochastic nature of driver perception is modeled such that as the driver reacts to a greater number of speed constraints, he or she will drive slower than the minimum of the constraining speeds.

MODEL PARAMETERS FOR UNITED STATES CONDITIONS

The parameters of the limiting velocity models that were examined for adjustment to U.S. conditions are:

G. E. Elkins, Texas Research and Development Foundation, 2602 Dellana Lane, Austin, Tex. 78746. J. Semrau, Department of Civil Engineering, University of Texas at Austin, Austin, Tex. 78712.

- The constraining speeds for each class, including *VDRIVE*, *VBRAKE*, *VCURVE*, *VROUGH*, and *VDESIR*;
- The exponential parameter *B* for each vehicle class;
- The variance term S^2, which represents the errors associated with speed predictions for each vehicle class.

Ideally, calibration of these models for United States conditions should be based on direct field measurements. To best model the effects of various road characteristics, actual sites that have only one dominating characteristic must be selected. Vehicle spot speeds should be measured and the model fitted to the data. Unfortunately, no suitable data representative of conditions in the United States could be found.

Maximum Possible Driving Speed (*VDRIVE*)

The maximum possible driving speed is the speed a vehicle travels when all the available driving power is used. *VDRIVE* becomes a constraint on the speeds of vehicles on positive vertical grades. The force balance and power relationship used to find *VDRIVE* is

$$Av(VDRIVE)^3 + mg(GR + CR)VDRIVE = 375\ HPDRIVE \quad (3)$$

where

VDRIVE = maximum possible driving speed, mph;
HPDRIVE = maximum available driving power, horsepower;
m = vehicle mass;
g = acceleration due to gravity;
CD = aerodynamic drag coefficient;
Av = frontal area of vehicle;
GR = vertical gradient;
CR = rolling resistance coefficient

The terms *HPDRIVE*, *m*, *CD*, *Av*, and *CR* are vehicle

TABLE 1 1985 U.S. VEHICLE FLEET MODELING PARAMETERS

Vehicle Class	Frontal Area Sq. Ft.	Vehicle Weight lb.	Drag Coef.	Rolling Resistance(1) CR = RC1 + RC2 (v)^n RC1	RC2	HPDRIVE Hp
Small Auto	22.8	2,720	0.42	.0125	6.5E-07	93
Medium Auto	25.9	3,780	0.45	.0125	6.5E-07	140
Large Auto	28.7	4,560	0.49	.0125	6.5E-07	200
Pickup	30.8	5,000	0.59	.016	6.5E-07	175
2A-SU Truck	36.9	12,000	0.70	.0076	9.0E-05	230
3A-SU Truck	55	35,000	0.70	.0076	9.0E-05	275
2S-2 Semi	90	50,000	0.80	.0076	2.0E-05	325
3S-2 Semi	90	62,500	0.80	.0076	2.0E-05	325

(1)NOTE: v = speed, mph
n = 2 for autos and pickups
n = 1 for trucks and semi

dependent. The values of these parameters developed for the U.S. vehicle fleet are shown in table 1. All terms but *HPDRIVE* are from automotive industry literature. *HPDRIVE* is the available horsepower after accounting for internal power losses. To find this parameter, *HPDRIVE* was back-calculated from Equation 3 by inserting the observed top speeds of various vehicles on 0-% grades. For automobiles it was found that the calculated *HPDRIVE* approximated the SAE net braking horsepower, which includes the effects of internal power losses. This quantity is recommended as *HPDRIVE* for automobiles.

A different approach to the estimate of *HPDRIVE* was required for trucks with diesel engines. The best source of information found for U.S. trucks was a 1970 study by the Western Highway Institute (WHI) (*3*). The WHI recommended multiplying the rated horsepower of a diesel engine by a factor varying from .78 to .85, depending on the number of axles, gear range, and engine size. This equation is suspect since the truck population has changed significantly since 1970. Although the relationship is used in this study, these results should be reviewed in future studies to determine its suitability.

Maximum Allowable Braking Speed (*VBRAKE*)

On steep downgrades, a maximum constraining speed, or braking crawl speed, has been observed (*4, 5*). The braking crawl speed is believed to be related to vehicle braking capability resulting through use of the retardation power of the engine (downshifting) and the brakes. In general, only large vehicles have been observed to slow down on steep down grades. Limiting crawl speeds on downgrades are not generally found on grades less than 4% or shorter than 3,000 feet.

Although large trucks may have braking speeds, little information examining this effect was found. *The 1985 Highway Capacity Manual* (*6*) indicates that very few studies have been performed to analyze the impact of heavy vehicles on traffic flow on downgrades. Due to the lack of information on this behavior, this term is not included in this model.

Maximum Allowable Curve Speed (*VCURVE*)

Most drivers decrease their speed to negotiate sharp horizontal curves. The effect of curves on vehicle speed has been widely studied. The World Bank model (*2*) related vehicle speed on a horizontal curve to the "maximum perceived friction ratio," called *FRATIO*. *FRATIO* is defined as the ratio of lateral forces on a vehicle to the normal force on the vehicle. The vehicle speed on the curves, with simplifying assumptions can be written as

$$VCURVE = [(FRATIO + SP)\, g\, RC]^{0.5} \qquad (4)$$

where:

$VCURVE$ = maximum allowable speed on curves,
$FRATIO$ = maximum perceived friction ratio,
SP = superelevation of curve,
RC = radius of curvature.

The *FRATIO* value is used to characterize different vehicle classes. A *FRATIO* value of 0.155 was found to provide a good fit to the speed-curve model used in the present HPMS for automobiles, pickups and single unit trucks. For large trucks and semitrailer units, a value of .103 was used, based on the relationship between cars and trucks determined from the World Bank study.

Other forms of this model could be used; however, the PLVM requires that this model predict high speeds on curves with large radii to avoid interaction with other terms in the PLVM that would falsely decrease the predicted speeds. Field studies of the performance of trucks on curves and the suitability of this model for U.S. conditions appear warranted.

Maximum Allowable Ride Severity Speed (*VROUGH*)

It is a common observation that road roughness influences vehicle speed. Few studies, however, relate vehicle speed to the roughness measures used in the United States or in the HPMS. A model is needed that explains differences in vehicle type and road type and accounts for limiting roughness thresholds and minimum speeds at maximum roughness levels.

Based on the information developed in Brazil and the speed-roughness model used in the current HPMS analytical process, the following equations were developed:

$$VROUGH = 1.0/(.0250 - .00275(PSR)) \text{ automobiles} \qquad (5)$$

$$VROUGH = 0.9/(.0255 - .00333(PSR)) \text{ large trucks} \qquad (6)$$

where

$VROUGH$ = ride severity speed, mph;
PSR = present serviceability rating, (0-5).

More work is needed on speed-roughness relationships. The above equations are primarily based on engineering judgment. Relationships derived from direct measurements and based on common roughness measures used in the U.S. are needed to extend the accuracy and usefulness of these speed prediction models.

Desired Speed of Travel (*VDESIR*)

The desired travel speed is the speed at which drivers travel when they are not constrained, typically less than the maximum possible speed a vehicle can attain. This speed is governed by subjective considerations of safety, speed law enforcement, fuel cost, and vehicle wear. For the purposes of the HPMS, the term should also be sensitive to the effects of traffic congestion and traffic control devices.

A limited nationwide source of information on *VDESIR* is the annual free flow speed tables published by the FHWA (*7*). Average, median, and 85th-percentile speeds on highways on which the 55-mph speed limit is the primary speed constraint are published.

To incorporate the effects of traffic congestion and traffic control devices into the model, tables of average speed as a function of highway type, traffic control, number of lanes, and speed limit were developed (*1*). These tables were developed from tables of initial running speed contained in the current HPMS model and are based on and extrapolated from the general speed-volume capacity relationships shown in the 1965 and 1985 Highway Capacity Manuals.

Due to the generalized information from which these speeds were developed, more work is needed to relate the desired speed constraint to physical road characteristics. A separate speed constraint term related to a simple measure of traffic congestion such as volume capacity ratio should also needs be developed, particularly for signalized urban streets.

B and S^2 Parameters

The exponential *B* parameter and variance parameter S^2 are part of the probabilistic limiting velocity model (PLVM). They are included to account for the stochastic nature of observed vehicle speeds. *B* and S^2 are primarily used to reduce the predicted speed when two or more constraining speeds become dominant. The *B* parameter acts similarly to the coefficient of the standard deviation of a normal distribution typically used to determine confidence levels. The smaller the value of *B*, the closer the probabilistic speed is to the minimum constraining speed. Meanwhile, the S^2 parameter is associated with errors in speed prediction, due to variations in vehicle and road characteristics, and other errors, due to speed measurements and quantification of road attributes.

These parameters are properly determined using a nonlinear least-squares regression analysis between observed speeds and speed constraint terms. Because there is no U.S. data base from which to develop these parameters, the values of $B = 0.1$ and $S^2 = 0.01$ were selected to cause the model to predict speeds that are less than the minimum speed constraint. The choice of these values causes the MLVM and PLVM models discussed in this paper to produce essentially the same results.

SUMMARY

The minimum and probabilistic forms of the limiting velocity model offer an excellent method to predict vehicle speeds as a function of relevant constraining speeds due to curvature, gradient, road roughness, braking capability, and other road features. The models presented here can be implemented into planning models for use in predicting user impacts. As discussed, further development of these models is required in order to better define the interrelations between road characteristics and vehicle speed.

The recommended method of further refinement of these models is through a structured nationwide study of vehicle speeds. This study would consist of spot speed studies on sections selected to study a particular speed constraint term. A full statistical analysis similar to that performed by the World Bank should be performed on this data base. In the face of a more limited study, the authors feel that the speed models presented here could best be improved by studies into the following topics in the following order:

1. Effects of signalization and traffic control
2. Effects of traffic congestion
3. Effect of roughness
4. Large truck performance on downgrades and horizontal curves

Input from experts in vehicle mechanics, dynamics, and human factors is also important in better defining the needed speed estimation models.

ACKNOWLEDGMENT

This work was performed under the sponsorship of the U.S. Department of Transportation, Federal Highway Administration, Office of Highway Planning.

REFERENCES

1. G. E. Elkins, B. C. Butler, R. High, and R. Harrison. *Estimating Vehicle Performance Measures: Volume I.* Technical Report, Federal Highway Administration, Washington, D.C., July 1987.
2. Watanatada, P. R. S. Rezende-Lima, and A. Dhareshwar. *Models for Predicting Vehicle Speed and Operating Costs Based on Principles of Vehicle Mechanics and Driver Behavior: Theory and Qualification.* Highway Design and Maintenance Standards Study, Volume II, World Bank, March 1985.
3. *Horsepower Considerations for Trucks and Truck Combinations, 1969, Acceleration Tests.* Supplement No. 1, Research Committee Report No. 2, Western Highway Institute, 1970.
4. H. Hide, S. W. Abayanayaka, I. Sayer, and R. I. Wyatt. *The Kenya Road Transport Cost Study: Research on Vehicle Operating Costs.* TRRL Laboratory Report 672, Transport and Road Research Laboratory, Crowthorne, Berkshire, U. K., 1975.
5. A. D. St. John and D. R. Kobett. *Grade Effects on Traffic Flow Stability and Capacity.* National Cooperative Highway Research Program Report 185, TRB, National Research Council, Washington, D.C., 1978.
6. *Highway Capacity Manual.* Special Report 209, TRB, National Research Council, Washington, D.C., 1985.
7. *Highway Statistics*, Federal Highway Administration, Washington, D.C., annual publication.

Publication of this paper sponsored by Committee on Geometric Design.