STATE OF THE ART REPORT 6

RELATIONSHIP BETWEEN SAFETY AND KEY HIGHWAY FEATURES

A Synthesis of Prior Research

Papers commissioned for the Study of Geometric Design Standards for Highway Improvements

Transportation Research Board
National Research Council
Washington, D.C. 1987

State of the Art Report 6
Editor: Edythe Traylor Crump

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Printed in the United States of America

Library of Congress Cataloging-in-Publication Data

Relationship between safety and key highway features.

(State of the art report ; 6)
1. Traffic safety—Research—United States. 2. Highway engineering—Research—United States. 3. Roads—Design—Research—United States. I. National Research Council (U.S.). Transportation Research Board. Committee for the Study of Geometric Design Standards for Highway Improvements. II. Series: State-of-the-art report (Washington, D.C.) ; 6.
HE5613.6.U5R44 1987 363.1'256'0973 87-35855
ISBN 0-309-04502-9 ISSN 0892-6891

The papers in this State of the Art Report were commissioned for the study of geometric design standards for highway improvements.

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The views expressed in this report are those of the authors and do not necessarily reflect the views of the committee, the Transportation Research Board, or the National Research Council.

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Foreword

For more than 10 years, a controversy has persisted over highway geometric standards for federal-aid resurfacing, restoration, and rehabilitation (RRR) projects. Reacting to increasing concern over deteriorating highways, the U.S. Congress first authorized the use of federal highway funds for RRR projects in 1976. In doing so, the Congress provided not only funds to help preserve and repair federal-aid primary, secondary, and urban highways but also an opportunity to enhance highway safety by upgrading roadway geometric features, particularly on older facilities with narrow lanes, sharp curves, or restricted sight distances.

Striking a balance between using RRR funds to preserve existing highways, especially pavement surfaces, and using funds to enhance highway safety has proved controversial, and the controversy has centered on which minimum geometric standards should be applied to RRR projects. Over the years various organizations have proposed or commented on minimum standards, but none have been adopted for nationwide use. Finally, in the Surface Transportation Assistance Act of 1982, Congress requested the National Research Council, the principal operating agency of the National Academy of Sciences and the National Academy of Engineering, to study the safety cost-effectiveness of design standards and recommend minimum geometric standards for RRR projects. A special committee, under the guidance of the National Research Council's Transportation Research Board, was appointed for this purpose. The committee's findings were published by the Transportation Research Board in *Special Report 214: Designing Safer Roads—Practices for Resurfacing, Restoration, and Rehabilitation.*

As part of its work, the Committee for the Study of Geometric Design Standards for Highway Improvements asked several experienced highway safety researchers to critically review current knowledge about the relationships between safety and the following key highway features:

- Curvature;
- Sight distance;
- Lane width, shoulder width, and shoulder type;
- Bridge width;
- Intersections;
- Pavement edge drop; and
- Pavement resurfacing.

Wherever possible, the committee asked the researchers to identify, based on prior research and their seasoned judgment, the "most probable" relationships between highway features and safety (as measured by accident frequencies or rates). Although such relationships are often poorly understood, they were essential to the committee's task of examining safety cost-effectiveness of different design standards. In addition, the committee commissioned a critical review of expected changes in vehicle characteristics from the standpoint of their effect on relationships between safety and highway geometry.

Because of the considerable interest shown in these reviews and the possibility that they will be of immediate value to highway designers and safety specialists, the committee recommended that abbreviated versions be published as a collection of papers in this State of the Art Report.

In general, the papers contain reviews of only research completed and published before January 1986. Important work may have been completed since then that alters or extends the conclusions presented. For example, the committee sponsored two research projects that were completed after the critical reviews. These projects included a study of the safety effects of cross-section design for two-lane roads, and a study of the performance of drivers in negotiating pavement edge drops of different heights and shapes.

The papers that are published in this report are entirely the work of their authors and do not reflect the views of the committee.

Effect of Lane Width, Shoulder Width, and Shoulder Type on Highway Safety

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Accident experience on rural highways is a complex function of many factors, including not only those associated with physical aspects of the roadway and the roadside but also a multitude of others related to driver, vehicle, traffic, and environmental conditions. Among the many roadway-related features of importance—estimated by one 1978 study to total at least 50 *(1)*—three that are often underscored as being among those having the greatest impact include lane width, shoulder width, and shoulder type.

The purpose of this investigation was to critically review relevant literature and develop a model for estimating the effect of lane width, shoulder width, and shoulder type on motor vehicle accidents on two-lane, rural highways. Preliminary issues considered important to this task include *(a)* criteria for selecting and evaluating useful studies, and *(b)* definitional issues.

CRITERIA FOR SELECTING STUDIES

More than 30 articles and reports dated between the early 1940s and the mid-1980s were reviewed. The conclusions of these studies were often not only inconsistent, but, in many cases, totally contradictory. For example, some studies concluded that wider shoulders result in an *increased* number of accidents, whereas others found that shoulder width had little or no effect on accidents (or only influenced accident frequency for specific levels of traffic volume). Still other studies revealed significantly fewer accidents on roadways with paved or widened shoulders than on those with unpaved or narrow shoulders, or both.

Because of this disparity in research findings, considerable selectivity was demanded in determining which studies should be considered among the most reliable. Such a task had been considered in detail by Zegeer and Perkins relative to the following study elements *(2)*: *(a)* type of data analysis and statistical testing, *(b)* reliability of the accident data sample, *(c)* characteristics of roadway sections, and *(d)* types of accidents analyzed. Criteria used herein to determine the major strengths and weaknesses of each source are given as follows:

Criteria related to data reliability

- Is the study data reasonably current or is it outdated?
- Did the author collect a sufficient sample for establishing reliable results?
- Was adequate detail maintained in the collection of important data variables?
- Did the author adequately control for possible data errors?
- What data biases exist in terms of state, geographic region, section lengths, roadway classes, and so forth? (It should also include the zero-accident sections.)

Criteria related to data analysis and results

- Were adequate control variables used?
- What accident types (rear-end, run-off-road, etc.) and units (frequencies, rates, etc.) were used in the analysis and were they properly handled?
- What assumptions were made in conducting the analysis and were they valid?
- Were appropriate analysis techniques and statistical tests applied?
- Did the author correctly interpret the analysis results?

Basic principles outlined in the Federal Highway Administration (FHWA) *Accident Research Manual* and the user's manual on *Highway Safety Evaluation* were also considered in the critical review *(3, 4)*.

Initial review of the 30 articles revealed numerous major flaws in many of the older (pre-1960) accident studies including the following:

- Specific attention was not focused on the types of accidents likely to be affected by lane and shoulder conditions.
- No measure of vehicle exposure was used in comparing accident experience for various lane and shoulder widths.
- The full effects of lane and shoulder conditions were obscured because the study was limited to straight, level, tangent sections.
- Because few or no "control variables" were used, relationships between lane or shoulder conditions and accidents were influenced in unknown ways by other roadway features.
- Although several studies incorporated appropriate statistical analysis techniques, others made gross or unsupported assumptions or used inappropriate tests for data analysis.

In addition to these flaws, use of data from older studies was considered undesirable for the following reasons:

• Current accident data bases are likely to be more reliable than older ones.

• Important safety-related vehicle characteristics have changed through the years, including such features as acceleration and braking ability, truck sizes and weights, availability of occupant restraints, and many others.

• The use of pavement delineation, signing, and other traffic control practices also differs today compared with earlier years.

As a result of these considerations, all pre-1960 studies were excluded from this critical assessment of the literature. Many post-1960 studies were also dismissed because of flaws, questionable study procedures, or other critical study limitations. Only nine studies, identified in Table 1, survived preliminary screening. Of these nine, the study by Rinde dealt with shoulder widening, whereas studies by Dart and Mann, Shannon and Stanley, and Zegeer et al. involved analyses of both lane and shoulder widths. Studies by Heimbach et al., Turner et al., and Rogness et al. involved only shoulder type, whereas studies by Foody and Long and Jorgensen analyzed lane width, shoulder width, and shoulder type *(1–9)*.

TABLE 1 Summary of Selected Studies

			Cross Sectional Elements Analyzed			Type of Analysis		
							Comparative Analysis	
Author	Date	States Included	Lane Width	Shoulder Width	Shoulder Type	Before/After	No Predictive Equation	Predictive Equation
Dart and Mann	1970	Louisiana	X	X				X
Heimbach, Hunter, and Chao	1974	North Carolina			X		X	
Foody, Long	1974	Ohio	X	X	X			X
Shannon, Stanley	1976	Idaho Washington	X[a]	X[a]				X
Rinde	1977	California		X		X		
Jorgensen & Associates	1978	Washington[b] Maryland	X	X	X			X
Zegeer, Mayes, Deen	1979	Kentucky	X	X			X	
Turner, Fambro, Rogness	1981	Texas			X		X	
Rogness, Fambro, Turner	1982	Texas			X	X		

[a]In this study, pavement width was the variable used in the analysis, which included total paved width (lanes plus shoulders).
[b]New York State was used for initial analysis, but excluded for development of accident relationships.

The studies by Rinde and Rogness et al. were before-and-after studies of completed shoulder widening projects in which the authors controlled for external factors *(5, 11)*. The remaining seven studies were comparative analyses, which 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 were desired that covered a wide range of lane- and shoulder-width and shoulder-type combinations. Also, data showing accident experience for the specific accident types most related to lane and shoulder deficiencies was considered most useful. Ultimately, data were selected from four of the nine studies to develop the general effects of these elements on safety. The studies included Zegeer et al., Kentucky; Foody and Long, Ohio; Rinde, California; and Rogness et al., Texas *(5, 8, 11, 12)*. Data from the Kentucky and Ohio studies *(8)* were used in the development of mathematical models that represent the most likely relationships between the rate of related (run-off-road and opposite-direction) accidents and combinations of lane width, shoulder width, and shoulder type.

DEFINITIONAL ISSUES

In the critical review, the following definitional issues had to be addressed:

- When comparing paved versus unpaved shoulders, what type of surfaces are included in the unpaved category (stabilized, gravel, grass, dirt)?
- How wide are the paved shoulders, and does the term "unpaved shoulders" imply that trees and other fixed objects may be located within 2 to 10 ft of the roadway?
- How much of the roadbed width is considered to be the lane and how much is considered to be the shoulder?

Attempts to resolve these and other issues required telephone contacts with the authors or others familiar with the studies and the data bases, or the use of unpublished research reports. Considerable clarification resulted from these followup investigations. For example, in a Texas study of shoulder effects, the term paved shoulder was defined as "any one of a wide range of all-weather surfaces—bituminous surface-treated shoulders, bituminous aggregate shoulders, full-depth asphalt shoulders, and portland cement concrete shoulders. They are constructed next to main line pavements of equal or better type" *(13)*.

Considerable variations were found among definitions used in the studies. Generally, a paved shoulder was considered to be an all-weather bituminous treatment. However, in studies comparing paved versus unpaved shoulders, a paved shoulder generally implied an 8- to 10-ft surface, whereas unpaved typically implied a grass or dirt shoulder free of obstructions for approximately 10 ft.

Citing another example, the North Carolina study by Heimbach et al. considered unpaved shoulders to be gravel, dirt, or grass surfaces on which obstructions generally do not exist for approximately 10 ft or more from the pavement edge *(9)*. Thus, unpaved shoulders may be considered to be driveable surfaces (except when wet), readily distinguishable from "no shoulder" situations. The study in Ohio by Foody and Long used several categories of shoulder—paved, stabilized, unstabilized, and other *(12)*. Unstabilized shoulders consisted of slag, gravel, soil, or grass. Although no clear definition is given for the "other" category, it was later learned that this represented situations in which no specific treatment was provided beyond the roadway edge.

In studies of the effect of shoulder width on accidents, variations were again found in the definition of width. In most studies, width apparently refers to all-weather paved or stabilized shoulders, or both. Such studies made a distinction between lane and shoulder width by comparing different surface types and noting a definite break between the roadway and the paved shoulder. However, in the study by Shannon and Stanley *(7)* and Rinde *(5)*, the authors refer to the entire pavement width, including the paved lane width and shoulders, even though the stated goal of the Rinde study involved the specific analysis of shoulder widening.

For use in developing accident relationships and predictive models in this paper, lane width is defined as the width of the travel lane, which is the width from the center of the roadway to one of the following points: *(a)* the edgeline, *(b)* where a visible joint separates the lane from the shoulder (if no edgeline is present), or *(c)* where the paved surface ends (if no paved shoulder exists). A shoulder is the area provided on some roadways intended primarily for emergency stopping or as a recovery area for vehicles leaving the travel lane. Paved shoulders are considered to be the width of bituminous or concrete material next to the travel lane. Stabilized shoulders are considered to consist of a mixture of bituminous material with gravel, so the surface is generally more smooth and compacted than loose gravel alone. Unstabilized shoulders (for the purpose of the accident model) are constructed of slag, gravel, crushed stone, grass, or soil, which are generally free of trees and most other roadside obstacles.

CRITICAL REVIEW AND ANALYSIS OF THE LITERATURE

Review and analysis of the nine most reliable studies addressed four specific questions related to the most likely relationships between accident experience and lane width and shoulder width and type:

- What dependent variables (i.e., accident measures) are most appropriate for expressing the relationships between safety and the three variables of primary interest?
- What other independent variables (e.g., widths, curvature, volume groups, roadside condition) should and can be included in developing accident relationships?
- What studies and data results are the most complete and reliable for determining the expected accident relationships?
- What is the most likely relationship between accident experience and lane width, shoulder width, and shoulder type?

Selection of Dependent Variable

The first major issue was to determine the types of accidents that are related to lane width, shoulder width, and shoulder type. Although total accidents had been commonly used in past accident studies, unrelated accident types influence the data base and mask the true effects of the lane or shoulder improvement. The importance of careful selection of the dependent safety variable has been emphasized in definitive procedural guides *(3, 4)*.

Of the nine studies selected following preliminary screening of the literature, three—Heimbach et al., Shannon and Stanley, and Jorgensen—analyzed total accidents or accidents stratified only by severity level *(1, 7, 9)*. Dart and Mann used total accidents stratified by severity, pavement wetness, and time of day, but did not separately analyze accident types such as rear-end, run-off-road, and the like *(6)*. Foody and Long analyzed only single-vehicle accidents, whereas Turner et al. analyzed run-off-road accidents, hit-other-car accidents, nondaylight accidents, total accidents, and accidents classified by severity level *(10, 12)*.

Detailed accident types were analyzed in the studies by Zegeer et al., Rinde, and Rogness et al. *(5, 8, 11)*. The seven categories of accidents analyzed by Zegeer et al. include run-off-road; opposite-direction; rear-end; passing vehicle; driveway and intersection; pedestrian, bicycle, animal, and train; and other or not stated *(8)*.

Only run-off-road (ROR) and opposite-direction (OD) accidents were found by Zegeer et al. to be associated with lane and shoulder width. The percentage of ROR and OD accidents ranged from more than 90 percent of total accidents for lane widths of 7 ft to as low as 31 percent for 13-ft lane widths (Table 2) *(8)*. It should be mentioned, however, that the sample size was small for both the 7-ft and 13-ft lane-width categories—123 and 135 accidents, respectively—and that 97 percent of the 16,000 mi of roadway in the data base included sections with 5,000 average daily traffic (ADT) or less.

Turner et al. found a higher frequency of run-off-road accidents on two-lane road sections with no shoulder *(10)*. Rates of hit-other-car accidents were also higher on these sections for certain volume levels. Unfortunately, head-on accidents were not separately analyzed.

Before-and-after studies by Rogness et al. and Rinde also analyzed specific accident types relative to shoulder improvement projects *(5,11)*. Rogness found that the frequency of single-vehicle accidents (run-off-road and fixed-object accidents) was reduced by adding shoulders on low-volume, two-lane roads (ADT levels of 1,000 to 3,000) *(11)*. For ADT levels of 3,000 to 5,000, shoulder additions reduced not only ROR accidents but multiple-vehicle accidents as well. However, the effect of head-on, multiple-vehicle accidents was not specifically addressed.

Rinde categorized accidents by accident type (head-on, rear-end, hit-object, overturn, and sideswipe) and also by movements after the collision *(5)*. As a result of pavement widening, head-on accidents were reduced by 50 to 60 percent, hit-object accidents by 27 to 53 percent, and rear-end accidents by 17 to 69 percent. Results were mixed for overturn and sideswipe accidents.

In summary, strong evidence exists that ROR and OD accidents are the primary accident types affected by lane or shoulder improvements, or both. This is particularly true for roads with low traffic volumes—ADT levels of 3,000 or less. Therefore, the rate of ROR and OD accidents was selected as the primary dependent variable for developing the accident relationships.

Selection of Independent Variables

Next, an examination was conducted of the possible need for adding to lane width, shoulder width, and shoulder type other interacting independent variables whose levels might influence the effect of lane and shoulder conditions on highway safety. Ideally, all independent variables chosen for inclusion in an accident model should interrelate with the three variables of concern in affecting the related accident types.

Previous literature has addressed the range of variables that may influence accidents on two-lane roads. For example, Jorgensen reviewed more than 400 reports and other

TABLE 2 Summary of Accident Frequencies by Type for Various Lane Widths *(8)*

Accident Frequencies by Type					Percent of Total Accidents		
Lane Width (ft)	Total Accidents	Run-off-Road	Opposite Direction	All Others	Run-off-Road	Opposite Direction	Run-off-Road and Opposite Direction
7	123	58	54	11	47.2	43.9	91.1
8	1,143	576	368	199	50.4	32.2	82.6
9	6,652	3,399	1,160	2,093	51.1	17.4	68.5
10	4,947	2,189	720	2,038	44.2	14.6	58.8
11	2,017	728	190	1,099	36.1	9.4	45.5
12	1,743	555	192	996	31.8	11.0	42.8
13	135	32	10	93	23.7	7.4	31.1
Total	16,760	7,532	2,694	6,534	44.9	16.1	61.0

publications on relationships between highway design elements and accidents *(1)*. Although more than 50 design features were found to affect safety, the authors emphasized that the validity of the various safety relationships had not been evaluated and noted that some of the relationships were contradictory. Although it is difficult to draw solid conclusions from a literature review of this type, it does provide evidence of the complexity of accident relationships and the possibility that numerous roadway factors may be significant.

Other studies confirm that dozens of roadway variables could affect highway safety and thus interrelate with the effects of pavement width, shoulder width, or shoulder type. Hundreds of such studies were compiled and summarized in a two-volume synthesis prepared for the FHWA in 1982 *(14)* but, like Jorgensen, they failed to critically assess the validity of suggested accident relationships.

Predictive accident models that account for interrelationships among roadway variables have been developed in a few studies. For example, Jorgensen developed a predictive model for total accidents based on independent variables such as pavement width, shoulder width, shoulder type, ADT, and horizontal curvature *(11)*. However, the R^2 value for that model was only 0.08, indicating that only about 8 percent of the accident variance was explained by the model. The predictive model of Dart and Mann also used total accident rate (accident rate per 100 million vehicle-mi) as the primary dependent variable and yielded a much better R^2 value of 0.46 (46 percent of accident variance explained) *(6)*. The independent variables in this model included various interactions among percent trucks, traffic volume ratio, cross slope, horizontal alignment, traffic conflicts, lane width, and shoulder width.

Based on a review of the publications identified in the preceding paragraph, as well as many others, the following general conclusions can be drawn:

- Numerous traffic, geometric, and roadway variables have an effect on the highway accident experience. Many of these variables interrelate, and certain variables—when combined—cause an unusually severe accident experience.
- The interrelationships of such variables and accidents are quite complex and have not yet been adequately quantified. There is strong evidence, however, that other independent variables (in addition to lane and shoulder widths and shoulder type) interrelate in affecting accidents on two-lane rural roads. These include roadside characteristics, horizontal and vertical curvature, volume level, access points, intersections, and others.
- Although the complete family of relationships cannot be developed here, it is desirable to determine the general or overall levels of expected accident experience associated with various combinations of pavement and shoulder widening or shoulder surfacing, or both, while controlling for the combined effects of other factors.

Selection of Data for Model Development

Data and information were carefully reviewed in each of the nine studies in order to select the most reliable accident relationships and the most complete information. Each study was characterized by strengths and weaknesses, necessitating constant judgment about the information that was the most reliable and complete. Five of the nine studies were not used to build the accident model for the following reasons:

- The Jorgensen study *(1)* quantified only the total accident experience, and the mathematical model explained only 8 percent of the variance in accidents.

- Although the Shannon and Stanley study *(7)* contained a rigorous statistical analysis of data from two states, it failed to analyze specific accident types and to provide accident experience for various lane- and shoulder-width combinations.
- The Dart and Mann relationships *(6)* explained a reasonable amount of the accident variance but only used total accidents as a dependent variable.
- The Heimbach et al. study *(9)* was one of the better studies on shoulder type and safety, but it did not include an analysis of specific accident types nor did it provide detailed accident rates.
- The Turner et al. study *(10)* presented composite run-off-road and total accidents but did not provide information on the rates for various combinations of lane and shoulder widths.

Although not perfect by any means, the four studies selected for development of most likely safety relationships were those by Zegeer et al., Foody and Long, Rinde, and Rogness et al. *(5, 8, 11, 12)*. The studies by Zegeer et al. in Kentucky and Foody and Long in Ohio were based on statewide data for two-lane roads *(8, 12)*. Data on approximately 16,000 mi of roadway (and nearly 17,000 accidents in one year) were used in the Kentucky study. The Ohio study also used approximately 16,000 mi of roadway (and more than 23,000 single-vehicle accidents in 2 years) in one phase of analysis and a 1,400-mi subfile for analyzing shoulder type. It was the only study that analyzed paved, stabilized, and unstabilized shoulder types separately. The Zegeer et al. study was the only one of the nine that had detailed accident rates for various combinations of lane and shoulder widths *(8)*. Adjustment factors from that study were used to adjust accident rates for the effects of other roadway features.

The studies by Rinde in California and Rogness et al. in Texas reported results of actual pavement or shoulder widening projects, or both *(5, 11)*. The Rogness study sampled 214 mi of roadway where paved shoulders had been added to two-lane highways *(11)*. Rinde studied 143 mi where total pavement widths were increased either to 28 ft (from initial widths of 20 to 24 ft), to 32 ft (from initial widths of 18 to 24 ft), or to 40 ft (from initial widths of 20 to 26 ft) *(5)*. Thus, for some sections of roadway in the Rinde study, lanes as well as shoulders were widened. Although such sample sizes would be small for many comparative analyses, the 357 mi were considered adequate for a before-and-after (with control) type of study, considering sample sizes indicated by the Poisson test as being necessary to detect significant change.

The studies by Rinde and Rogness et al. revealed reductions in both total accidents and in specific accident types *(5, 11)*. The Rinde study adjusted the after-accident experience on the basis of statewide accident trends to control for the external influences of the 55 mph speed limit, the energy crisis, and changes in traffic volume *(5)*. The Rogness study adjusted for changes in traffic volume *(11)*. Both studies used appropriate statistical tests to determine which accident reductions were statistically significant.

Accident Relationships and Reduction Factors

Average accident rates (accidents per million vehicle miles) from the Zegeer et al. study are given in Table 3 for all accidents and also for ROR and OD accidents for various combinations of lane and shoulder widths *(8)*. The interrelated effects of various combinations of lane and shoulder widths on unadjusted rates of ROR and OD accidents are shown in Figure 1 *(8)*. Note that rates generally decrease as lane and shoulder widths increase. However, the unadjusted accident rates were approximately the same (or slightly higher) for 12-ft lanes as for 11-ft lanes, possibly indicating in part the limit beyond which further increases in lane width are ineffectual.

TABLE 3 Average Accident Rates (per million vehicle miles) as a Function of Lane and Shoulder Width for Two-Lane Rural Roads in Kentucky *(8)*

	Shoulder Width (ft)									
	No Shoulder		1–3		4–6		7–9		10–12	
Lane Width (ft)	Accident Rate	No. of Sections[a]	Accident Rate	No. of Sections[a]	Accident Rate	No. of Sections[a]	Accident Rate	No. of Sections[a]	Accident Rate	No. of Sections[a]
All Accidents										
7	5.09	286	1.94	110		_[b]		_[b]		_[b]
8	3.60	2,460	4.06	344		_[b]		_[b]		_[b]
9	3.17	6,032	2.86	2,185	2.92	9	1.83	6		_[b]
10	3.01	1,384	2.73	1,080	3.11	23	2.96	8	2.54	12
11	1.86	382	2.71	275	2.21	31	0.85	21	2.21	38
12	1.91	168	2.43	87	2.26	27	1.82	34	1.86	26
Run-off-Road and Opposite-Direction Accidents										
7	4.70	286	1.71	110		_[b]		_[b]		_[b]
8	2.96	2,460	3.42	344		_[b]		_[b]		_[b]
9	2.22	6,032	1.92	2,185	1.34	9	1.22	6		_[b]
10	1.83	1,384	1.62	1,080	1.19	23	1.03	8	1.03	12
11	1.03	382	1.02	275	0.81	31	0.51	21	0.84	38
12	0.77	168	1.08	87	0.98	27	0.70	34	0.90	261

[a]Number of 1-mi sections used to calculate average accident rate.
[b]Fewer than five sections were available in test sample.

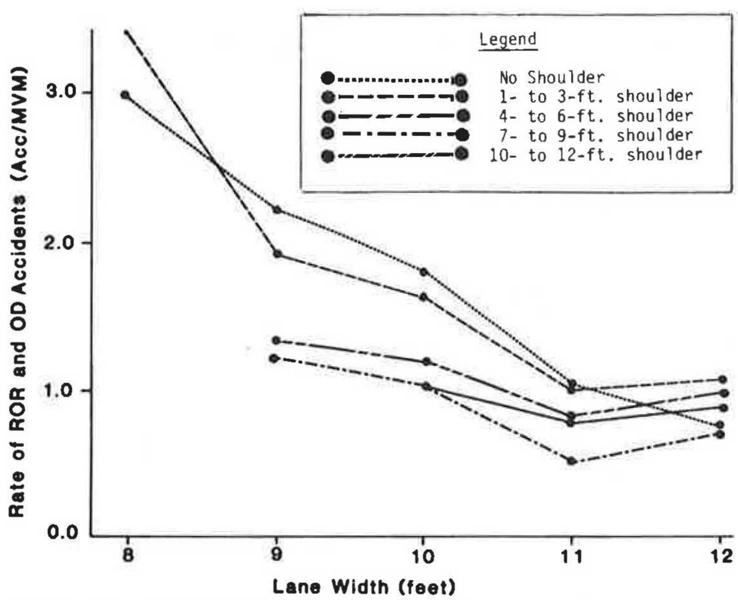

FIGURE 1 Relationship between accidents and lane and shoulder width in Kentucky (8).

Zegeer et al. adjusted these accident rates in an attempt to control for the effects of traffic and other roadway variables based on a plot of the unadjusted accident rates as a function of volume level (Figure 2) *(8)*. Although this was not an ideal method of control, the higher accident rates for low ADT groups (with sharp curves, poor roadsides, and other deficiencies) were clearly seen for different pavement-width classes. The authors then developed accident reduction factors that might realistically be anticipated as a result of lane and shoulder widening (Tables 4 and 5) *(8)*.

After other factors were controlled for, the expected reduction in ROR and OD accidents from shoulder widening projects ranged from 6 to 21 percent, depending on

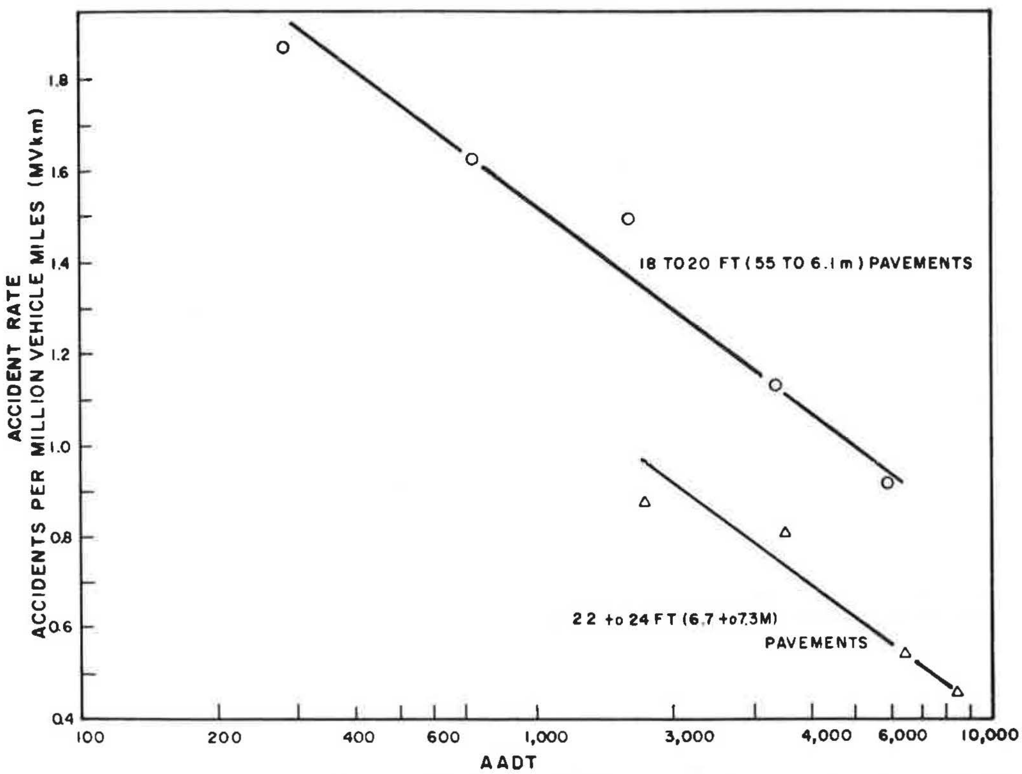

FIGURE 2 Rates of run-off-road accidents for various ADT groups and pavement widths in Kentucky (8).

TABLE 4 Percent Reduction in Run-off-Road and Opposite-Direction Accidents Due to Lane Widening *(8)*

Lane Width (ft)		Total Widening (ft)	Percent Reduction in ROR and OD Accidents
Before Widening	After Widening		
7	8	2	10
7	9	4	23
7	10	6	29
7	11	8	39
8	9	2	16
8	10	4	23
8	11	6	36
9	10	2	10
9	11	4	29
10	11	2	23

TABLE 5 Percent Reduction in Run-off-Road and Opposite-Direction Accidents Due to Shoulder Widening *(8)*

Shoulder Width (ft)		Total Widening (ft)	Percent Reduction in ROR and OD Accidents
Before Widening	After Widening		
None	1–3	4	6
None	4–6	10	15
None	7–9	16	21
1–3	4–6	6	10
1–3	7–9	12	16
4–6	7–9	6	8

the amount of widening. Lane widening was expected to cause greater accident reductions—10 to 39 percent after adjusting for other factors—again depending on the amount of widening *(8)*. Although this information is useful, it would have been more appropriate if accident reduction factors had been determined for various combinations of lane and shoulder widths. For example, what would be the expected accident reduction for shoulder widening from 0 to 3 ft for an existing lane width of 10 ft, as compared with similar shoulder widening for existing lane widths of 11 or 12 ft? Fortunately, this deficiency was found to be correctable in the current study.

Foody and Long performed several types of analyses for single-vehicle accidents, including an attempt to model such accidents by using data for 16,000 mi of roadway—an attempt that proved to be of only limited success *(12)*. The second phase of the study, however, was a detailed analysis of shoulder type for 1,400 mi of roadway sample data. Results of a series of analysis of variance (ANOVA) tests revealed that the mean rates of single-vehicle accidents were not significantly different for sections with paved shoulders compared with those with stabilized (tar with gravel) shoulders *(12)*. Subsequently, paved and stabilized shoulders were grouped into a single category termed "stabilized." Also, no significant differences were found in the mean rates of single-vehicle accidents for other types of shoulders. Accordingly, all such types were subsequently collected into one group, termed "unstabilized." Most important, however, the mean accident rate for stabilized shoulder sections was significantly less than that for sections having unstabilized shoulders *(12)*.

Mean rates of single-vehicle accidents are given in Table 6 for sections with both unstabilized and stabilized shoulders and for three pavement width categories—16 to 20 ft, 20 to 24 ft, and 24 to 28 ft. Note that these rates (or rate differences) are *not* adjusted for effects of other factors (curvature, ADT, etc.) because roadway width, shoulder quality, and roadside quality were the only independent variables used in the analysis.

These results indicate that shoulder stabilization or paving may be quite effective in reducing run-off-road accidents on narrow roadways, typically 20 ft or less in width, but have little effect on roads having widths of 24 ft or more. This finding basically agrees with data from the Zegeer et al. study *(8)*, which found a greater reduction in ROR and OD accidents as a result of shoulder widening for narrow lane widths as opposed to 12-ft lane widths.

Rogness et al. reported results of shoulder and roadway improvements that included 30 sections (214 mi) where paved shoulders had been added to two-lane roads *(11)*. Two years of accident data were analyzed for each of the before-and-after periods at each site. The effects of the treatments were analyzed for specific accident types within three ADT categories: 1,000 to 3,000, 3,000 to 5,000, and 5,000 to 7,000. The *t*-test was used (at the 90 percent confidence level) to determine whether changes in the accident pattern were statistically significant *(11)*.

TABLE 6 Rates of Single-Vehicle Accidents for Pavement Width and Shoulder Type Combinations in Ohio *(12)*

	Base Rate of SV Accidents (ACC/MVM)		
Pavement Width (ft) (excluding shoulder)	Unstabilized Shoulder	Stabilized Shoulder	Difference in Accident Rate (*D*)
16–20	3.57	1.11	2.46
20–24	2.04	1.40	0.64
24–28	1.02	0.98	0.04

Note: SV = single-vehicle; ACC/MVM = accidents per million vehicle miles.

TABLE 7 Accident Reductions as a Result of Adding Shoulders to Two-Lane Roadways in Texas *(11)*

Volume Range	Type of Accident	No. of Accidents Before	No. of Accidents After[a]	Percent Change
1,000–3,000	Multivehicle	35	36.4	+4.0
	Single vehicle[b]	58	26.1	–55.0
	Other[c]	27	25.1	–7.0
	Total	120	87.6	–27.0
3,000–5,000	Multivehicle	68	53.9	–14.7
	Single vehicle[b]	67	52.9	–21.4
	Other[c]	29	36.7	+26.6
	Total	164	143.5	–12.5
5,000–7,000	Multivehicle	27	16.9	–37.4
	Single vehicle[b]	12	12.0	0
	Other[c]	6	8.2	+36.6
	Total	45	37.1	–17.6

NOTE: These include nonintersection accidents only.
[a]Adjusted for changes in average daily traffic.
[b]Run-off-road and hit-fixed-object accidents.
[c]Other single-vehicle accidents.

TABLE 8 Summary of Reductions in Total Accident Rates in California Due to Shoulder Widening *(5)*

Pavement Width[a] (ft)	AADT	Percent Change
28	<3,000	–16
32	<5,000	–35
40	>5,000	–29

[a]Pavement width refers to the paved width of lanes plus shoulders after widening.

Reductions in the frequency of single-vehicle accidents were found to be 55 percent for ADT levels of 1,000 to 3,000, 21.4 percent for ADTs of 3,000 to 5,000, and 0 percent for ADTs of 5,000 to 7,000 *(11)*. This trend appears consistent with other studies that have found greater accident reductions from lane and shoulder improvements on roads with lower ADT levels. No significant reductions were found for head-on accidents. The summary of the percent changes and accidents (Table 7) is for nonintersection accidents only. Accident numbers in the after period were adjusted by the authors to account for any volume differences between the before-and-after periods *(11)*.

The study by Rinde was a before-and-after evaluation of shoulder (pavement) widening for 37 projects representing 143 mi of two-lane, state-maintained highway in California *(5)*. Sections selected for evaluation had been constructed between 1964 and 1974 onexisting alignment, and the chi-square test (95 percent confidence level) was used to detect significant changes in various accident types. An attempt was made to control for the effects of external factors (estimated to account for only 4 to 6 percent of the reduction in accidents) during the analysis period. These effects were believed to primarily include the energy crisis and the resulting 55 mph speed limit. Statewide accident experience throughout the analysis period was used to determine the effect on these external factors, yielding adjustments of 4 to 6 percent *(5)*.

The summary given in Tables 8 and 9 shows reductions of 50 to 60 percent in head-on accidents *(5)*. Reductions of 27 to 53 percent were observed for hit-object accidents. The larger (53 percent) reduction for the middle (32 ft) category cannot be readily explained, except that such fluctuations are not uncommon in accident-based evaluations because of data instability or randomness, or both. These percent reductions included adjustments for volume changes but not for other external influences. After adjustments for

TABLE 9 Summary of Reductions in Accident Rates for Collision Types in California by Lane Width and Traffic Volumes *(5)*

Pavement Width (ft)	No. of Accidents Before	No. of Accidents After	Accident Rate Before	Accident Rate After	Percent Change
Head-on collision					
28 (<3,000 ADT)	3	2	0.10	0.05	–50
32 (<5,000 ADT)	32	19	1.04	0.50	–52[a]
40 (All)	29	14	0.14	0.06	–57[a]
Rear-end collision					
28(<3,000 ADT)	2	2	0.06	0.05	–17
32 (<5,000 ADT)	10	4	0.32	0.10	–69[a]
40 (All)	80	71	0.37	0.29	–22
Hit-object collision					
28 (<3,000 ADT)	37	35	1.19	0.87	–27
32 (<5,000 ADT)	34	20	1.10	0.52	–53[a]
40 (All)	137	112	0.64	0.46	–28[a]
Overturn					
28 (<3,000 ADT)	13	18	0.42	0.45	+7
32 (<5,000 ADT)	10	18	0.32	0.47	+47
40 (All)	61	41	0.29	0.17	–41[a]
Sideswipe					
28 (<3,000 ADT)	1	8	0.03	0.20	+567[b]
32 (<5,000 ADT)	14	14	0.45	0.37	–18
40 (All)	43	37	0.20	0.15	–25

[a]Statistically significant decrease.
[b]Statistically significant increase.

TABLE 10 Summary of Accident Reductions for Pavement Widening Projects *(5, 11)*

Type of Project	ADT Range	Expected Percent Reduction in Accidents: Total Accidents	Single-Vehicle Accidents	Head-On Accidents
Widening 20 to 24-ft pavement to 28 ft	0–3,000	16 (C)	22 (C)	45 (C)
Widening 18 to 24-ft pavement to 32 ft	<5,000	35[a] (C)	49[a] (C)	48[a] (C)
Widening 18 to 24-ft pavement to 40 ft	>5,000	29[a] (C)	22[a] (C)	51[a] (C)
Adding full-width paved	1,000–3,000	27[a] (T)	55[a] (T)	Unknown
shoulders to two-lane	3,000–5,000	12.5 (T)	21.4[a] (T)	Unknown
roads	5,000–7,000	17.6[a] (T)	0 (T)	Unknown

NOTES: (C) indicates values from the Rinde study in California, and (T) indicates values from the Rogness et al. study in Texas. The single-vehicle and head-on accident percentages for California were adjusted by 4 to 6 percent to account for external effects. These adjusted percentages are now on the same basis as total accidents.
[a]These percent differences were significant at the 95 percent level of confidence for California sites (C) and 90 percent confidence level at the Texas sites (T).

other external influences had been made, the authors recommended percent reductions of 16, 35, and 29 percent in total accidents (for the three ADT groups) *(5)*.

Accident reduction factors for the Rinde and Rogness et al. studies are summarized for comparative purposes in Table 10 *(5, 11)*. These include percent accident reductions for total accidents with similar adjustments (for 4 to 6 percent) for single-vehicle accidents and head-on accidents in California. Reductions in total accidents ranged from 16 to 35 percent. Single-vehicle accidents dropped by as much as 55 percent as a result of widening but were unchanged in the 5,000 to 7,000 ADT group on Texas highways. Head-on accidents were reduced by 45 to 51 percent, based on the California data.

Several seemingly illogical patterns in the summary given in Table 10 warrant further discussion. For example, a reduction of 49 percent in single-vehicle accidents was found in California as a result of widening lanes to 32 ft, whereas only a 22 percent reduction was found as a result of widening lanes to 40 ft. In both cases, pavements were 18 to 24 ft in the before condition. Note, however, that projects involving widening to 32 ft included lower ADT levels (i.e., ≤ 5,000) compared with widening to 40 ft (i.e., ADT ≥ 5,000). Most research has indicated that a larger percentage of single-vehicle accidents are eliminated in the lower ADT groups because single-vehicle accidents are typically more of a problem on low-volume roads that have sharper curves, less forgiving roadsides, and so forth. Thus, the pattern observed in California, though counterintuitive at first glance, is not unreasonable.

Another interesting pattern is the reduction in total accidents as a result of the addition of full-width, paved shoulders in Texas. A 27 percent reduction was found for the low-volume (1,000 to 3,000 ADT) group, compared with 12.5 and 17.6 percent reductions for the 3,000 to 5,000 and 5,000 to 7,000 ADT groups, respectively. Although these reductions are not completely consistent, a plausible explanation is that widening projects are likely to be more effective on low-volume roads—which are more likely to have deficient roadways and roadsides—than on higher-volume roads. Random accident fluctuations may be responsible for the inconsistent upturn in the highest volume category.

Certainly differences are apparent between California and Texas data—possibly because of differences between the types of projects in the two states. For example, all of the projects in Texas involved adding paved, full-width shoulders to existing two-lane roads, whereas California projects involved differing amounts of total pavement widening. Nevertheless, the accident reduction factors in Table 10 represent the best information currently available on the effects of actual shoulder or pavement widening projects, or both.

DEVELOPMENT OF SAFETY RELATIONSHIPS

Although no satisfactory quantitative model relating accident rate to lane and shoulder conditions was found in the published literature, 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 ROR and 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 effect of shoulder-width increments is diminished as either the base lane width or base shoulder width increases.
- Lane width has a greater effect on accident rates than shoulder width.
- Nonstabilized shoulders, including those constructed of loose gravel, crushed stone, raw earth, and turf, exhibit larger accident rates than stabilized (e.g., tar with gravel) or paved (e.g., bituminous or concrete) shoulders.

Among numerous mathematical relationships capable of replicating these patterns, one of the simplest has the following form:

$$AR = (C_1)\ (C_2)^L\ (C_3)^S\ (C_4)^{LS}\ (C_5)^P\ (C_6)^{LP} \quad (1)$$

where

AR = number of ROR and OD accidents per million vehicle miles,
L = lane width in feet,
S = shoulder width in feet (including stabilized and unstabilized components),
P = width in feet of stabilized component of shoulder ($0 \leq P \leq S$: $P = 0$ for unstabilized shoulders and $P = S$ for full-width stabilization), and
C_i's = constants.

This model was calibrated using the best available data—presented earlier in Tables 3 and 6—taken from the Kentucky and Ohio studies *(8, 12)*. The first part of the two-part process involved a weighted, least-squares fit of Equation 1 to the data in Table 3. "No shoulder" data were excluded from this exercise because it actually contained data from highway sections having unstabilized shoulders of varying width. The calibrated model, reflecting only the effects of stabilized shoulders at this stage, was extended in the second part by data from Table 6. Only two data points were used: one indicating a 4 percent increase in accident rate for unstabilized as opposed to stabilized shoulders for wide pavements (e.g., 12-ft lanes with 8-ft shoulders), and the other indicating a 46 percent increase in accident rate for pavements of intermediate width (e.g., 10-ft lanes with 8-ft shoulders). Data for narrow pavements were excluded because of the apparently unreasonable accident rate for stabilized shoulders. In using the data in Table 6, the effect of shoulder stabilization on the rate of ROR and OD accidents was assumed to be the same as its effect on the rate of single-vehicle accidents.

The calibrated model, applicable only to lane widths between 7 and 12 ft and shoulder widths of 10 ft or less, is identified as follows:

$$AR = 40.290\ (0.7329)^L\ (0.8497)^S\ (1.0132)^{LS}\ (0.7727)^P\ (1.0213)^{LP} \quad (2)$$

Comparisons of estimates from Equation 2 with the actual data from which it was calibrated emphasize that the "fit" is far from perfect (Table 11). Nevertheless, the general trends are accurately reproduced: abnormalities in the available data bases cannot and should not be reproduced by any modeled relationship.

As a complication, the accident model reflects at this stage not only the effects of lane and shoulder conditions, but also the effects of other variables, such as curvature, sight distance, clear zones, sideslopes, and roadside obstacles. Because highways with inferior cross-sectional and roadside characteristics are also likely to have inferior geometrics, the modeled accident rates overstate the effect of safety gains resulting from improvements in lane and shoulder conditions alone. Actual accident reductions resulting from lane and shoulder improvements without accompanying improvements in other features may be as low as 50 percent of the reductions anticipated by the preceding model, according to information derived in the Kentucky study *(8)*.

Although available data bases do not provide an accurate guide for identifying the effects of these contributing factors, prior analysis of the Kentucky data provided a reasonable first approximation *(8)*. Central to this approximation is the hypothesis that when the difference between before-and-after accident rates (as estimated by Equation 2) is small, the confounding effects of the exogenous variables are also likely to be small. As a result, actual safety gains will be similar to modeled gains. As the modeled

TABLE 11 Comparison of Accident Model with Kentucky and Ohio Data

		Number of ROR and OD per MVM		Ratio of Accident Rates for Unstabilized and Stabilized Shoulders	
Lane Width (ft)	Shoulder Width (ft)	Kentucky Data	Accident Model[a]	Ohio Data	Accident Model[b]
7	1–3	1.7	2.6–3.8		
8	1–3	3.4	2.2–2.9		
9	1–3	1.9	1.8–2.2		
9	4–6	1.3	1.2–1.6		
9	7–9	1.2	0.9–1.1		
10	1–3	1.6	1.4–1.7		
10	4–6	1.2	1.1–1.3		
10	7–9	1.0	0.9–1.0		
11	1–3	1.0	1.2–1.3		
11	4–6	0.8	1.0–1.1		
11	7–9	0.5	0.9–1.0		
12	1–3	1.1	0.9–1.0		
12	4–6	1.0	0.9		
12	7–9	0.7	0.9		
8–10	4–8			3.22	1.21–2.04
10–12	4–8			1.46	1.02–1.46
12–14	4–8			1.04	1.02–1.04[c]

[a]Tabulated range in shoulder widths.
[b]Tabulated ranges in pavement and shoulder widths.
[c]For 12-ft lane only.

safety gains become larger, however, effects of the confounding variables become more pronounced, and actual gains are likely to represent a smaller fraction of modeled gains.

Such a relationship can be expressed in terms of accident reduction factors (ARF)—the expected percent reduction in accidents due to an improvement—as follows:

$$ARF_a = (ARF_m)^c \tag{3}$$

where

ARF_a = an estimate of the accident reduction factor that can actually be achieved by lane and shoulder improvements;
ARF_m = the accident reduction factor resulting from application of Equation 2, which overstates the effect of lane and shoulder conditions; and
c = a calibration constant.

The constant, c, was calibrated by using data from the Kentucky study *(8)* (Table 12). Entries in Table 12 indicate the percent reduction in ROR and OD accidents expected to result from various widening projects. Zegeer et al. found the unadjusted differences (computed directly from the entries in Table 4) to overstate achievable gains because of the correlation, on Kentucky highways, between poor lane and shoulder conditions and poor geometric and roadside conditions *(8)*. The adjusted differences in Table 12 are a best estimate of the actual safety gains that can be achieved by widening, assuming that concurrent improvements in other roadway features are not made.

The last column of Table 12 provided the necessary information for calibrating Equation 3: a least-squares fit yielded an estimate of 0.4293 for c. The following model,

TABLE 12 Comparison of Adjusted and Unadjusted Differences in ROR and OD Accidents for Various Amounts of Pavement Widening *(8)*

Lane Width (ft)		Unadjusted Percent Differences[a]			Average Percent Difference (Unadjusted) (%)	Adjusted AR Factor (%)	Ratio of Adjusted to Unadjusted
Before	After	0–3-ft Shoulders (%)	4–6-ft Shoulders (%)	7–9-ft Shoulders (%)			
8	10	47	–	–	47	23	0.49
8	11–12	69	–	–	69	36	0.52
9	10	18/16[b]	–	–	17	10	0.59
9	11–12	51	–	–	51	29	0.57
10	11–12	41	33	–	37	23	0.62

NOTE: Dashes indicate insufficient data.
[a]From Table 4.
[b]Zero ft and 1 to 3 ft.

adjusted to remove unwanted effects of the confounding variables, is derived from the application of this exponent to Equation 2:

$$AR = 4.7918\ (0.8766)^{L}\ (0.9333)^{S}\ (1.0056)^{LS}\ (0.8964)^{P}\ (1.0090)^{LP} \qquad (4)$$

Although Equation 4 generally conforms with the known qualitative effects of lane and shoulder conditions on accident rates, the effects of lane and shoulder increments for wide initial cross sections are questionably small. Only a 3 percent reduction in accidents is estimated by Equation 4 for an increase in lane width from 10 to 12 ft for roadways with 8-ft stabilized shoulders, or for an improvement from no shoulders to 8-ft stabilized shoulders for roadways with 12-ft lanes. In comparison, the addition of full-width paved shoulders in one instance has been found to reduce single-vehicle accidents by as much as 55 percent (Table 10).

Accordingly, further adjustment in Equation 4 was deemed desirable. Because of the absence of firm data, adjustments were largely intuitive. First, a 20 percent difference in the accident rates for 12-ft lanes with no shoulders and those with 8-ft stabilized shoulders was assumed. This is the approximate value observed within the Kentucky data (Table 3) after adjusting for external effects. Second, the comparative effect of stabilized versus unstabilized shoulders, as indicated by Equation 4, was generally considered to be valid for mid-range lane and shoulder widths. Third, the adjustment maintained the accident rates established by Equation 4 for 9-ft lanes. Shown in Figures 3 and 4 for stabilized and unstabilized shoulders, the final model is described as follows:

$$AR = 4.1501\ (0.8907)^{L}\ (0.9562)^{S}\ (1.0026)^{LS}\ (0.9403)^{P}\ (1.0040)^{LP} \qquad (5)$$

APPLICATION OF RESULTS TO RRR PROJECTS

The purpose of this investigation was to develop, from published sources, a model for estimating the effect of lane and shoulder conditions on motor vehicle accidents on two-lane rural highways. Of the more than 30 articles reviewed, 9 studies were deemed most appropriate for detailed consideration, and information from 4 of the 9 was ultimately used in developing the most likely accident relationships.

The accident types found to be most related to lane and shoulder widths and shoulder type were run-off-road and opposite-direction accidents. Opposite-direction

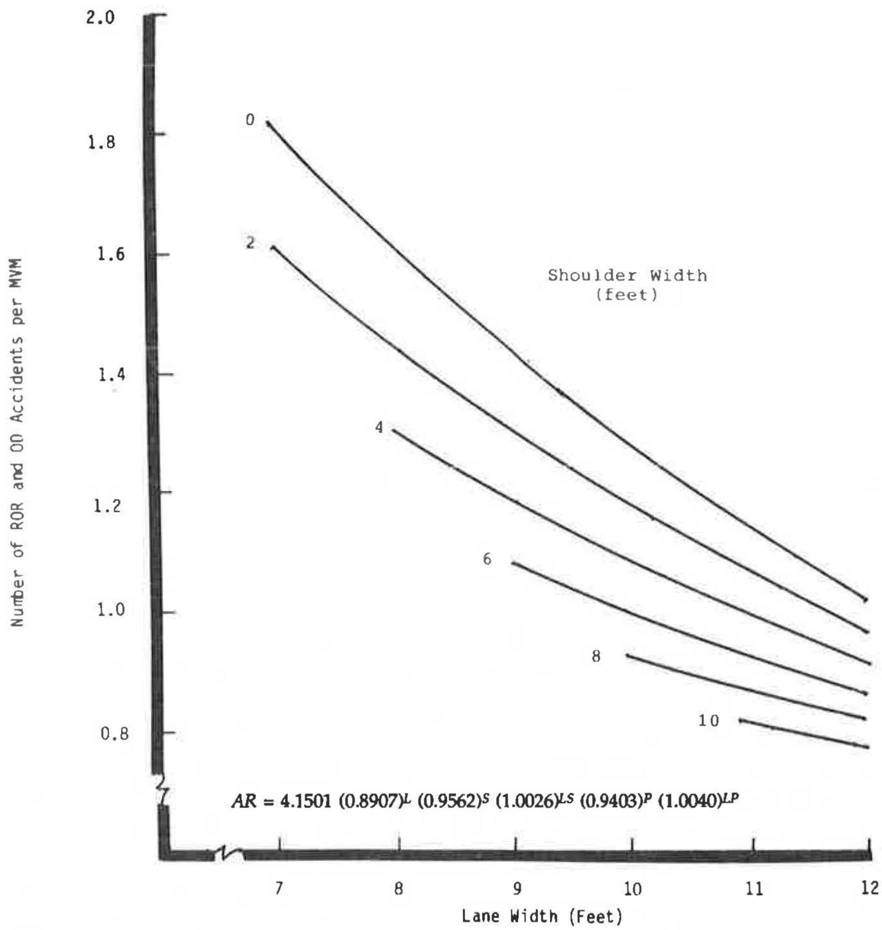

FIGURE 3 Adjusted rate of ROR and OD accidents for stabilized shoulders.

accidents include head-on and sideswipe accidents between vehicles traveling in opposite directions. Thus, the rate of ROR and OD accidents was considered to be the most appropriate dependent measure. The literature did not contain sufficient information to enable development of a complete group or family of accident relationships that incorporated traffic and other roadway effects as independent variables. However, it was possible to develop accident relationships that at least accounted for interrelationships among the variables of primary interest, namely, lane width, shoulder width, and shoulder type.

Of more than 30 research studies reviewed on accident effects of lane and shoulder conditions, the following 4 had supportable results and useful data for developing accident relationships:

- Zegeer et al. in Kentucky *(8)*;
- Foody and Long in Ohio *(12)*;
- Rinde in California *(5)*; and
- Rogness et al. in Texas *(11)*.

Primarily on the basis of the results of these four studies, lane width and shoulder width and type were found to have a significant impact on highway safety. Collectively, these studies indicated the following:

- Lane and shoulder conditions directly affect run-off-road and opposite-direction accidents. Other accident types, such as rear-end and angle accidents, are not directly affected by these conditions.
- 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 effect of shoulder-width increments is diminished as either the base lane width or base shoulder width increases.
- Lane width has a greater effect on accident rates than shoulder width.
- Larger accident rates are exhibited on unstabilized shoulders, including loose gravel, crushed stone, raw earth, or turf, than on stabilized (e.g., tar plus gravel) or paved (e.g., bituminous or concrete) shoulders.

These qualitative relationships served in large part as the basis for developing a quantitative accident model. Data for calibration of the model were extracted from the 1979 Kentucky study *(8)* by Zegeer et al. and the 1974 Ohio study *(12)* by Foody and Long. Adjustments were made to remove unwanted effects of other confounding variables (such as curvature, ADT, roadside condition, etc.) and to assure appropriate consideration of shoulder-width effects for roadways having wider lanes.

The final model is defined as

$$AR = 4.1501\ (0.8907)^{L}\ (0.9562)^{S}\ (1.0026)^{LS}\ (0.9403)^{P}\ (1.0040)^{LP} \qquad (6)$$

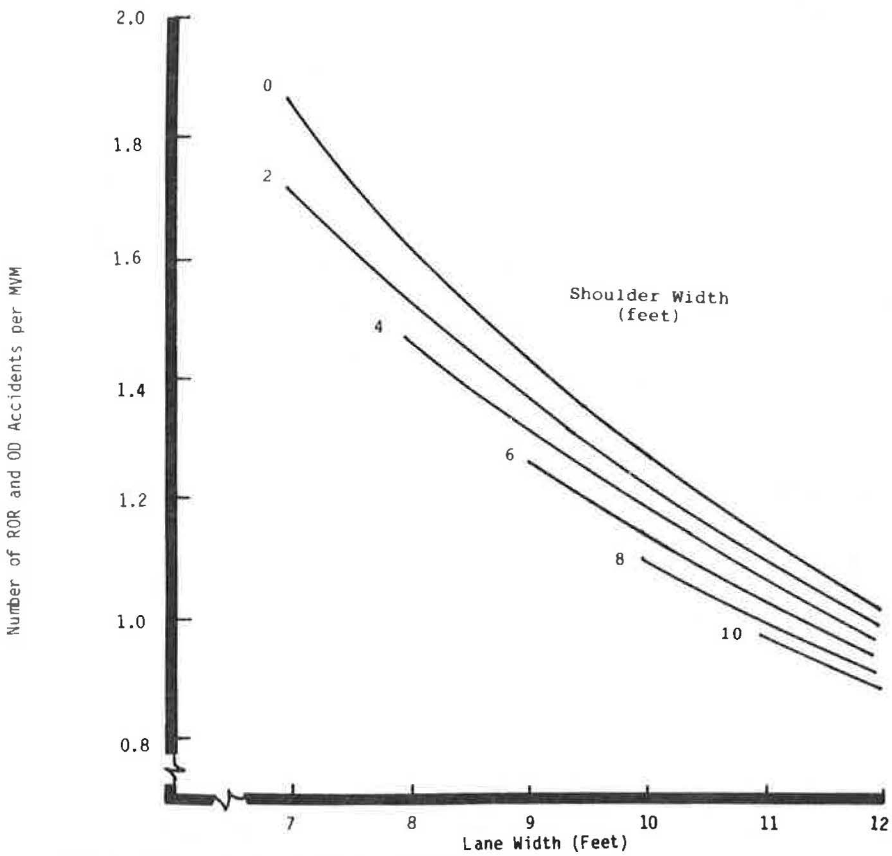

FIGURE 4 Adjusted rate of ROR and OD accidents for unstabilized shoulders.

where

AR = number of ROR and OD accidents per million vehicle miles,
L = lane width in feet,
S = shoulder width in feet (including stabilized and unstabilized components), and
P = width in feet of stabilized component of shoulder ($0 \leq P \leq S$).

Because of the many assumptions necessary in its development and the reliance on available data bases from only two states for its calibration and validation, this model is not considered to be a precise representation of the effects of lane and shoulder conditions on accident rates for all possible situations. However, when applied judiciously, it can serve as a useful first approximation of such effects. It does represent the best information currently available, and its most legitimate use is in the development of accident reduction factors that can be applied to actual accident rates to estimate likely reductions due to lane and shoulder improvements.

Limitations of the accident prediction model include the following:

- The model applies only to lane widths of 7 to 12 ft and shoulder widths of 0 to 10 ft. Furthermore, combinations of lane and shoulder widths that can be reasonably modeled are limited to those shown in Figure 3.
- The results relate to two-lane, two-way roads on state primary or secondary systems, or both.
- The results relate to rural, homogeneous roadway sections and generally exclude signalized intersections and corresponding intersection accidents.
- The results apply to paved roadways and include sections with curves and tangents and various types of terrain and roadway conditions.

This paper is strictly a critique of literature on the accident relationships of lane width, shoulder width, and shoulder type together with the development of most likely effects of pavement widening or shoulder paving, or both, on accidents. The economic impacts of widening pavements or improving shoulders were not addressed. Also, this review did not determine the pavement widths that should be used under various traffic conditions or roadway classes. Finally, no attempts were made to review literature or make judgments regarding the operational effects of lane and shoulder widths or shoulder type (e.g., effects on travel time or highway capacity).

ACKNOWLEDGMENTS

Charles V. Zegeer was employed by Goodell-Grivas, Inc., Southfield, Michigan, when this paper was written. Don Reinfurt of the University of North Carolina provided helpful technical and editorial comments on this paper.

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Effect of Bridge Width on Highway Safety

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In 1983, 44 percent of the nation's 550,000 highway bridges were reported to be deficient in one or more ways. Structural condition and deck geometry were considered the most pervasive deficiencies *(1)*. Deck geometry was reported the primary deficiency for 34,135 (12.7 percent) bridges on arterial and collector highways, most of which have widths narrower than the approach roadways.

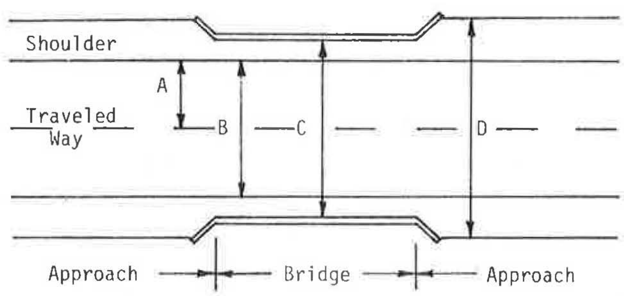
Mak and Calcote *(2)* found that the number of bridge-related fatal accidents per 100 million vehicle miles of travel was significantly higher than average for all road types. The number of bridge-related nonfatal accidents per 100 million vehicle miles of travel were also higher than average for Interstates and rural arterials and collectors, but lower for urban arterials and collectors. Hilton reported that the fatality rate for bridge-related accidents was roughly two times that of the average accident *(3)*. It is evident that a safety problem does exist with bridges in general, especially those on Interstates and rural highways.

Bridge width, both absolute and relative, has long been considered a major factor affecting safety at bridge sites. Ideally, the bridge width should be at least the same as the approach roadway width from a safety standpoint. However, the costs associated with bridge structures are very high in comparison to a normal roadway section, especially for long-span structures. In terms of costs, it is economically prohibitive to upgrade all existing bridges to the full approach roadway width. Some trade-off is therefore necessary, particularly for minor roadways with low traffic volume.

A critical review of available literature on the safety effects of bridge width was conducted as part of a Transportation Research Board study on geometric design standards for resurfacing, restoration, and rehabilitation (RRR) projects on non-Interstate highways. The results of the literature review and synthesis are presented in this paper.

RESEARCH FINDINGS: BRIDGE WIDTH VERSUS SAFETY

In presenting the results of past research, studies are categorized as those that *(a)* use surrogate measures, *(b)* evaluate safety at bridge sites in general, and *(c)* specifically evaluate the safety effect of bridge width. Some studies are in more than one of these categories. Pertinent portions of these studies are reviewed separately under the appropriate category. Also, to assure uniformity of definitions, the key elements at a bridge site are shown in Figure 1.



where A = Lane Width,

B = Traveled Way Width,

C = Bridge Width,

D = Approach Roadway Width,

RW = Relative Bridge Width

= Bridge Width - Traveled Way Width; or (C -B)

FIGURE 1 Key elements at bridge site.

Surrogate Measures Studies

A number of studies have been conducted on driver behavior at bridge sites *(4–6)*. In a 1941 study Walker reported on 11 bridge sites on two-lane, two-way rural highways (9 in Maryland and 2 in Oregon) *(4)*. Data were collected on lateral positions of more than 20,000 vehicles traversing the bridges at these sites, on both straight and level sections of highway.

A West Virginia study on driver behavior involved both an experimental and a field study *(5)*. In the experimental study, 10 subjects were asked to drive an instrumented car over a mock-up two-lane, two-way, 50-ft-long bridge 30 times for each increment of bridge width from 16 to 48 ft. All tests were conducted during daylight hours. A variety of data were collected, including steering wheel reversals and vehicle lateral placement. In the field study, the shoulder width of a two-lane, one-way bridge on an Interstate highway was varied from 2 to 10 ft using mock-up curb and guardrail. Two hundred passenger cars traversing the bridge were monitored for speed and lateral placement for each shoulder width. Again, the data were collected only during daylight hours and under fair weather conditions.

Ivey et al. collected vehicle speed and lateral placement data for more than 2,000 vehicles at 25 bridge sites on rural two-lane, two-way highways in 7 states *(6)*. The characteristics of the bridge sites, as in the Walker study *(4)*, varied widely, with different bridge and approach roadway widths, lengths, traffic volumes, and bridge structure types—from truss to open deck design.

The following conclusions were drawn from the studies on driver behavior:

1. Vehicle speed was affected very little by bridge width, but was more a function of other bridge and approach characteristics, such as vertical alignment *(4–6)*.
2. Walker *(4)* concluded that drivers tended to maintain a more or less uniform distance between their right wheel and the curb or parapet of the bridge, which resulted in a lateral movement of the vehicles toward the left if the bridge was narrower than the approach roadway. This distance varied by bridge width, time of day, and whether the vehicle was free-moving or meeting another vehicle. Other factors that influenced driver behavior included the presence or absence of centerline stripes, truss versus deck design, and bridge length. Ivey et al. reported that the lateral movement was a function of both the absolute and relative bridge widths *(6)*.
3. Under the more critical condition of meeting another vehicle, Ivey et al. *(6)* found that the drivers tended to maintain approximately the same clearance between their vehicle and the opposing vehicle to the left and between their vehicle and the curb or parapet of the bridge to the right.
4. In the instrumented vehicle study *(7)*, shoulder widths of 4 to 6 ft were found to result in the lowest number of steering wheel reversals and the greatest lateral distance between the left wheel of the vehicle and the centerline.
5. Walker *(4)* reasoned that, for complete freedom of movement on a bridge, one-half of the bridge width should equal to the sum of one-half the clearance allowed between vehicles while meeting on the highway, the width of the vehicle (assumed to be 5 ft), and the clearance to the curb or parapet of the bridge under free-moving condition.

The biggest problem encountered in these studies that used surrogate measures is that there is no established linkage that would allow the results to be related to accidents. Also, only passenger cars were included in these studies; no consideration was given to trucks, which have greater widths. Nevertheless, logical relationships, such as those developed by Walker, can be of some use. For example, assuming that a minimum of 6 ft is desired from the right wheel of the vehicle to the curb or bridge rail (a range of 4.2 to 7.4 ft was reported by Walker), a vehicle width of 5 ft, and the distance from the left wheel of the vehicle to the centerline is 3 ft (i.e., one-half of the clearance between opposing vehicles that Ivey found to be roughly the same as the clearance to the curb or bridge rail), the minimum bridge width should be 28 ft, which is twice the sum of 6 + 5 + 3 = 14.

General Bridge Safety Studies

Safety at bridge sites has long been an area of concern and the subject of numerous studies *(3, 8–10)*. All of these studies pointed to the hazard of narrow bridges, but were mostly descriptive in nature and did not provide sufficient data to establish the relationships between bridge width and accidents.

In 1955 Williams and Fritts *(8)* reported that, based on an analysis of accident data from 10 states, the accident rate was 1.0 accident per million vehicles for bridge structures with widths of 1 ft or more narrower than the approach roadway width, 0.58 for widths of between 1 ft narrower and 5 ft wider, and only 0.12 for widths of 5 ft or more. No detail was provided on how these figures were compiled.

In 1966 Hilton identified bridges in Virginia on which an unusually high number of accidents had occurred *(3)*. Thirty bridges on arterial and primary system highways were randomly selected for study from the list of bridges identified. Another 27 bridge sites on Interstate highways on which two or more accidents had occurred during 1966 were also selected for study. Narrow bridge width, curved bridge and approach roadway alignment, and downhill approaches were found to be the most prevalent characteristics at the 30 arterial and primary system bridges. The ratio of bridge roadway to approach roadway width was determined for 19 of the bridges, 17 (89.5 percent) of which had ratios of less than 0.8, and 16 (84.2 percent) of which had ratios of less than 0.7. Adverse surface condition was reported as the most prominent factor responsible for accidents on the high-accident Interstate bridges. Furthermore, nearly two-thirds (63 percent) of these bridges had clear widths of only 28 to 30 ft, whereas 74 percent of the sites had a bridge-to-roadway width ratio of less than 0.8. Unfortunately, no data were available for comparison purposes on bridges with lower accident experience.

In a study of accident statistics for Kentucky during 1972 and 1973, Agent reported that approximately 35 percent of bridges on Interstates and parkways had full-width shoulders and accounted for only 10 percent of bridge accidents *(9)*. Also, none of the nine bridges identified as high-accident sites (seven or more accidents in the 2-year study period) had full-width shoulders. For bridges on primary and secondary highways, none of the 11 bridges identified as high-accident sites (three or more accidents in 1972) had wide shoulders.

Similar findings were reported in an Australian study by Brown and Foster *(10)*. Nearly 70 percent of bridge accidents (single-vehicle accidents that occurred on bridges and their approaches) during 1961–1962 occurred on bridges where the bridge-to-approach roadway width ratio was less than 0.8 ft; only 14 percent occurred on bridges with full approach width.

A series of studies have been aimed at developing a bridge safety index (BSI) that would serve as an indicator of the degree of hazard associated with a bridge and as a means to priority rank bridges for corrective treatment *(6, 7, 11, 12)*. The BSI, as first developed by Ivey et al. *(6)*, is the sum of 10 individual rating factors. Three of the factors are related to bridge width: *(a)* bridge clear width, *(b)* ratio of lane width on bridge to that on approach roadway, and *(c)* percent shoulder reduction. A field evaluation form and accompanying instruction procedures were developed for use by highway department personnel *(11)*.

Two additional rating factors were later added to the BSI to provide an indication of the presence of safety treatments; that is, delineation and signing at the bridge sites *(7)*. Bridge, roadway, and accident (1978–1979) data were collected and analyzed on 78 bridge sites in Texas where corrective measures were recommended in an effort to validate the BSI. The data were further analyzed statistically by using logistic regression to develop a better and more objective BSI *(12)*.

The BSI is an intuitively appealing concept that allows the relative safety or hazard of bridges to be expressed by using a single index. The rating factors were initially developed subjectively on the basis of "engineering judgment" with no attempt to test and validate the index. Mak and Calcote tested the BSI using accident data and found that it was not a good indicator of safety at bridges. Similar results were reported in follow-up studies to improve on the original BSI *(7, 12)*. However, the results of these follow-up studies are likely to be biased because the 78 bridge sites studied were not representative of the bridge population; that is, these bridge sites had been recommended for corrective treatment, which would mean that they either had high accident experience or were perceived as hazardous.

Bridge width, the first rating factor of the BSI, was found to be significantly related to accident rates in all these studies *(2, 7, 12)*. However, the results are of little use because the bridge width term was either expressed as a rating or was part of a statistical model with poor predictive ability.

Accident Studies on Bridge Width

Few studies have been specifically designed to address the safety effects of bridge width *(2, 13–16)*. These studies were all of the cross-sectional (comparative evaluation) design except for the study by Gunnerson *(13)*, which used a retrospective before-and-after design.

Gunnerson studied the accident data on 72 narrow, rural two-lane bridges in Iowa over a 12-year period from 1948 to 1959 *(13)*. Of the 72 bridges studied, 65 had widths less than 24 ft, which remained unchanged while the approach pavements were widened to 24 ft. The remaining seven bridges had both the bridge and approach roadway width widened to 30 ft and were used for control purposes. Comparisons were made between the number of total accidents, number of bridge hits, injuries, and amount of property damages in the before and after time periods. To account for differences in the amount of time in the before and after periods, the accident frequencies were adjusted to be on a per month basis. No adjustment was made for any change in traffic volume on these bridges although the data were divided into separate average daily traffic (ADT) groups for analysis.

It was concluded that accident frequencies increased sharply when only the approach roadway pavement width was widened and not the bridge width or, conversely, when the approach traveled way width was wider than the bridge width. When both the approach roadway and bridge were widened to the same width, the accident frequency decreased. The study design suffered from the lack of control sites for comparison purposes. The seven bridges used as control sites actually received a different treatment, and the comparison was more that of differences between the two treatments. With a long study period of 12 years, changes to the bridge or approach roadway widening, or both, were likely to have occurred, which could affect accident frequencies, especially when no adjustment for traffic volume changes was made. Despite the drawbacks to the study design, the results illustrated that the bridge width should, at the very minimum, be as wide as the approach traveled way width.

Cirillo reported on the effect of lateral clearance (distance from edge of traveled way to bridge rail) at bridge structures on accident frequency and severity *(14)*. The accident data covered a 3-year period from 1961 to 1963 and approximately 2,000 mi of Interstate highways in 16 states. Accident rates were tabulated for various combinations of structure length and lateral clearance. Cirillo concluded that increase in minimum lateral clearance would reduce accident rates. Also, as the structure length increased, the need for larger lateral clearance was indicated by the increase in the accident rate. Similar tabulations were compiled for accident severity, expressed in terms of property damage costs. The results are not too meaningful, however, because property damage cost is not a good indicator of occupant-injury severity.

A somewhat different conclusion can be drawn from the data by combining some categories of bridge lengths, as shown in Figure 2. For structures with lengths up to 300 ft, the accident rates decreased sharply when the lateral clearance increased from less than 6 ft to 6.0–8.9 ft, but remained little changed or actually increased slightly when the lateral clearance was further increased to 9.0–12.9 ft. For the longer structures, the accident rates continued to decrease with an increase in lateral clearance. This appears

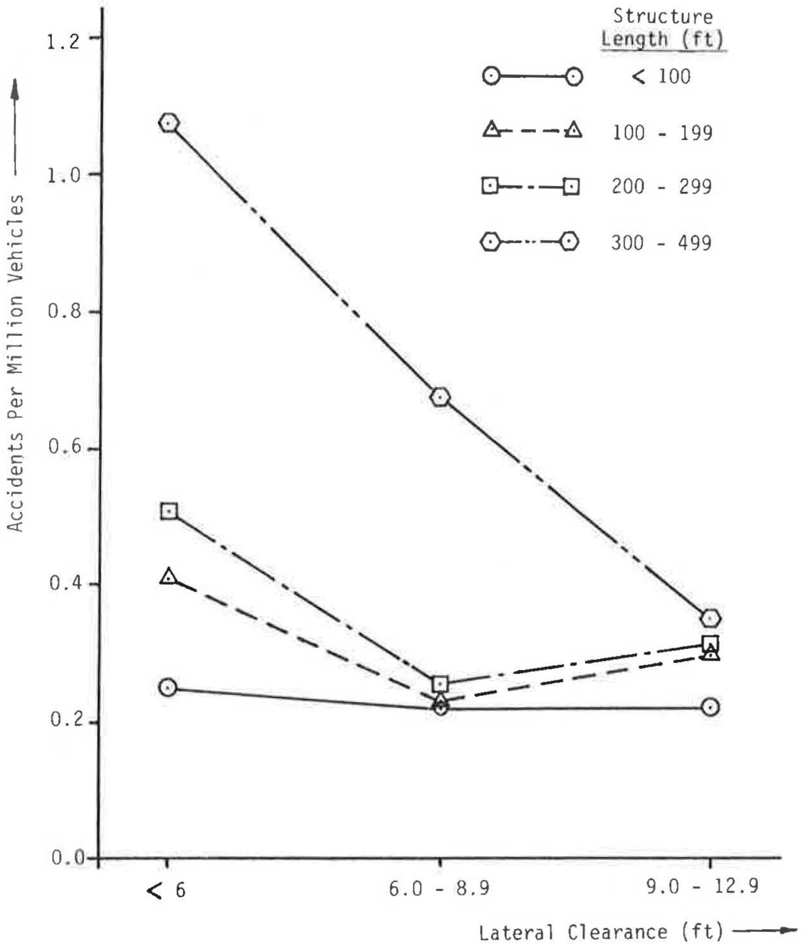


FIGURE 2 Accident rate per million vehicles by structure length and lateral clearance (14).

to indicate that, for structures with lengths up to 300 ft, the minimum lateral clearance should be no less than 6 ft, but little safety benefits are gained by increasing the lateral clearance to more than 9 ft. As for longer structures, it is desirable to have as great a lateral clearance as possible. It should be noted that the values of 6 and 9 ft are artifacts of the data grouping and not necessarily the critical values.

In a study of 58 bridges on Interstate highways in Colorado *(15)*, comparisons of total, nonfatal injury, and fatality rates (accidents per 100 million vehicles) for bridge-related accidents over a 4-year period (1968 to 1971) were made between twin structures with widths of 30 and 38 ft. The study concluded that twin structures 38 ft wide (full approach shoulders of 10 ft right and 4 ft left) were almost four times safer than those 30 ft wide (only 6 ft of shoulder total).

The Colorado study also included a representative sample of 219 structures on rural, two-lane primary highways. Accident rates were purported to be related to *(a)* bridge width; *(b)* relative bridge width (i.e., structure width minus approach traveled way width); and *(c)* bridge shoulder width versus approach roadway shoulder width. However, no statistical analysis was presented.

Of the 219 structures, only 58 experienced one or more accidents during the 4-year study period for a total of 94 accidents. Accident rates by bridge widths and by relative bridge widths are given in Tables 1 and 2. Least-squares quadratic curves were fitted to the data and are shown graphically as Equations 1 and 2 in Figures 3 and 4, respectively. The minimum accident rate according to the curves is at a bridge width of 30.5 ft or when the structure width is approximately 6 ft wider than the approach traveled way width.

TABLE 1 Accident Rate by Bridge Width *(15)*

Bridge Width (ft)	No. of Structures	No. of Accidents	Average ADT	Accidents per Million Vehicles
19	3[a]	1	250	0.91
20	2[a]	10	1,975	1.73
23	1[a]	1	1,350	0.51
24	15	12	1,179	0.46
25	57	21	787	0.32
28	27	5	1,051	0.12
29	24	4	1,325	0.09
30	78	39	1,721	0.20

[a]This data point has a sample size of less than five and is not used in the reanalysis.

TABLE 2 Accident Rate by Relative Bridge Width *(15)*

Relative Width (ft)	No. of Structures	No. of Accidents	Average ADT	Accidents per Million Vehicles
<0[a]	5[b]	11	940	1.60
0[a]	8	8	1,286	0.53
1	9	12	1,581	0.58
2	6	4	1,490	0.31
3	38	7	752	0.17
4	4[c]	0	340	0.00
5	15	4	879	0.21
6	48	20	1,865	0.15
7	18	2	1,160	0.07
8	49	22	1,368	0.22
>8	19[b]	4	1,069	0.13

[a]Narrow bridges having no paved shoulders.
[b]This data point is not used in the reanalysis because the boundary for the interval is not defined.
[c]This data point has a sample size of less than five and is not used in the reanalysis.

There were 55 structures with full-approach shoulder width, and the average accident rate was 0.20 accidents per million vehicles, which was 20 percent lower than the accident rate of 0.24 accidents per million vehicles for the 164 structures without full-approach shoulder width.

The study concluded that the optimum bridge width should be the greater of 30.5 ft, 6 ft wider than the approach traveled way width, or carrying the full approach shoulder.

The study results should be viewed with caution because the data were collapsed into one-way tables for analysis, which could mask or confound the effects of other

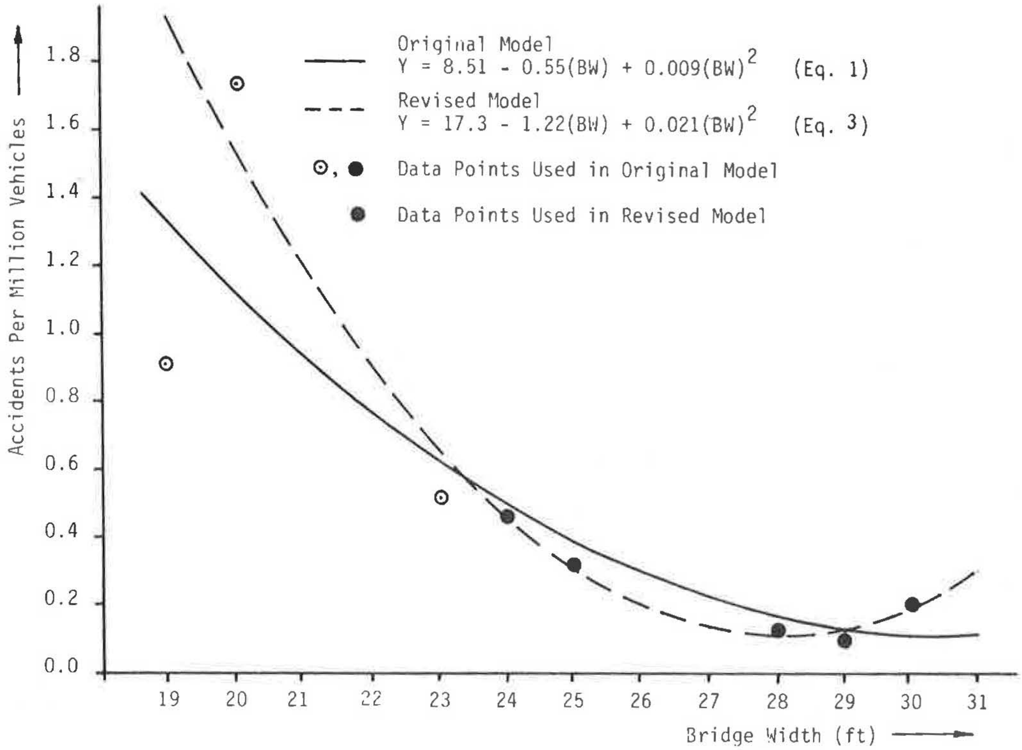


FIGURE 3 Accident rate by bridge width (15).

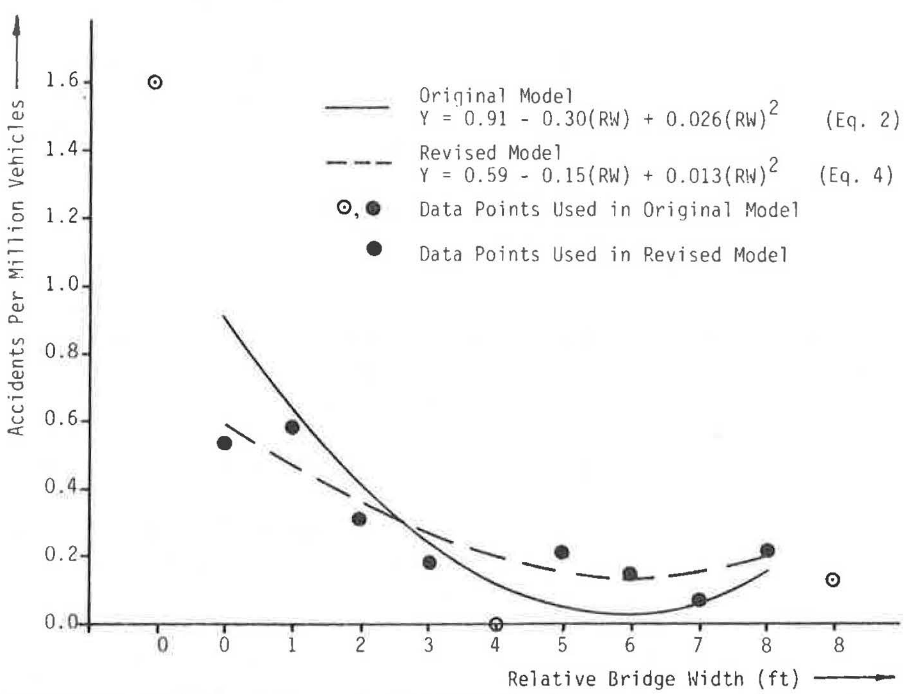


FIGURE 4 Accident rate by relative bridge width (15).

influencing factors on accident rates. The sample sizes were very small for some of the bridge widths or relative bridge widths, and the calculated accident rates could be somewhat unstable. Also, no statistical testing was attempted with the analyses.

To overcome the concern of unstable accident rates due to small sample sizes, data from categories of bridge widths or relative bridge widths with less than five structures are eliminated and the data reanalyzed. Again, least-squares regression models are fitted to the data, and the revised curves are shown graphically as Equations 3 and 4 in Figures 3 and 4, respectively, together with the original curves. It should be noted that the revised curves are based on a small number of data points (shown as solid circles in Figures 3 and 4).

In general, the reanalysis does not substantially alter the conclusions except that the minimum accident rate occurs at a bridge width of 28 ft instead of 30 ft. As for relative bridge width, the minimum accident rate occurs at a relative bridge width of 6 ft in both cases.

In a more recent study by Mak and Calcote *(2)*, computerized data files on bridge inventory and accidents (1975–1977) from five states were used to create a data base for study. Accidents were matched to the bridges and their approaches (500 ft for each approach) through a milepoint matching process. The final data base contained bridge, roadway, traffic, and accident data on 11,880 bridges with 24,809 associated accidents. A stratified random sample of 1,989 bridges was then selected for manual collection of data to supplement the computerized data. A variety of analyses were conducted in an effort to relate accident frequency (accidents per year per bridge), rate (accidents per million vehicles), and severity (average cost per accident) to the bridge, approach, operational, and countermeasure characteristics of the bridge sites.

The study revealed that for two-lane single structures the accident rate generally decreased with increasing bridge width for those bridges narrower than the approach roadway. However, for those bridges that were wider than the approach roadways, the highest accident rates were at bridge widths of between 20 and 22 ft. For structures under 24 ft wide, the majority of the bridges either had no approach shoulders or shoulder reduction on the bridges. The percentage of bridges on which accidents occurred and the accident rates were much lower for bridges with no shoulder reduction or no approach shoulders. For structures wider than 24 ft, the percentage of bridges on which accidents occurred and the accident rates were highest for bridges with more than 50 percent shoulder reduction, and both decreased with less shoulder reduction.

For two-lane twin structures, bridges with widths of 24 ft or less and those with greater than 50 percent shoulder reduction had similarly high accident rates. The accident rates decreased significantly for bridges with 1 to 50 percent shoulder reduction and remained almost unchanged for bridges with no shoulder reduction. This suggests that there is little difference in safety benefits between bridges with no shoulder reduction or 1 to 50 percent shoulder reduction.

As for accident severity, shoulder reduction appeared to have some marginal effect on twin structures, with higher accident severity for bridges with greater than 50 percent shoulder reduction. Otherwise, bridge narrowness appeared to have no effect on accident severity.

Mak and Calcote *(2)* evaluated the safety effects of absolute or relative bridge width categorically in terms of no approach shoulder, greater than 50 percent shoulder reduction, 1 to 50 percent shoulder reduction, and no shoulder reduction. Results of statistical analyses in which accident frequency, rate, and severity were related to bridge width, shoulder reduction, and other bridge, roadway, and traffic characteristics were generally weak. However, the results did illustrate that bridge width was only one of many factors influencing accident rates at bridge sites.

Turner used bridge, roadway, and accident data on rural two-lane highways in Texas to predict bridge accident rates *(16)*. The sample consisted of 2,849 bridge-related accidents on 2,087 bridges over a 4-year period from 1975 to 1978. After some preliminary analyses, three variables—ADT, approach roadway width, and bridge relative width—were selected for further analysis. ADT was then dropped from the analysis because it was already included in the determination of accident rate, expressed as the number of accidents per million vehicles. A 99-cell matrix of 9 approach roadway widths and 11 bridge relative widths was formed, and the accident rate per million vehicles was calculated for each cell.

Plots of accident rate by bridge relative width for each approach roadway width category displayed a similar trend of high accident rates at small relative widths that decreased with increasing relative width. Also, approach roadway width was found to be nonsignificant in the regression analysis and was dropped from further consideration. The data were then combined for all approach widths, and accident rates were regressed against bridge relative widths using weighted regression analysis. The resulting weighted regression equation is as follows and the curve is shown in Figure 5:

$$Y = 0.50 - 0.061\,(RW) + 0.0022\,(RW)^2 \tag{5}$$

where Y is the number of accidents per million vehicles, and RW is the bridge relative width in feet.

At relative widths of 6 ft or wider, the curve remained fairly flat with an accident rate of between 0.07 and 0.2 accidents per million vehicles. For relative widths of less than 6

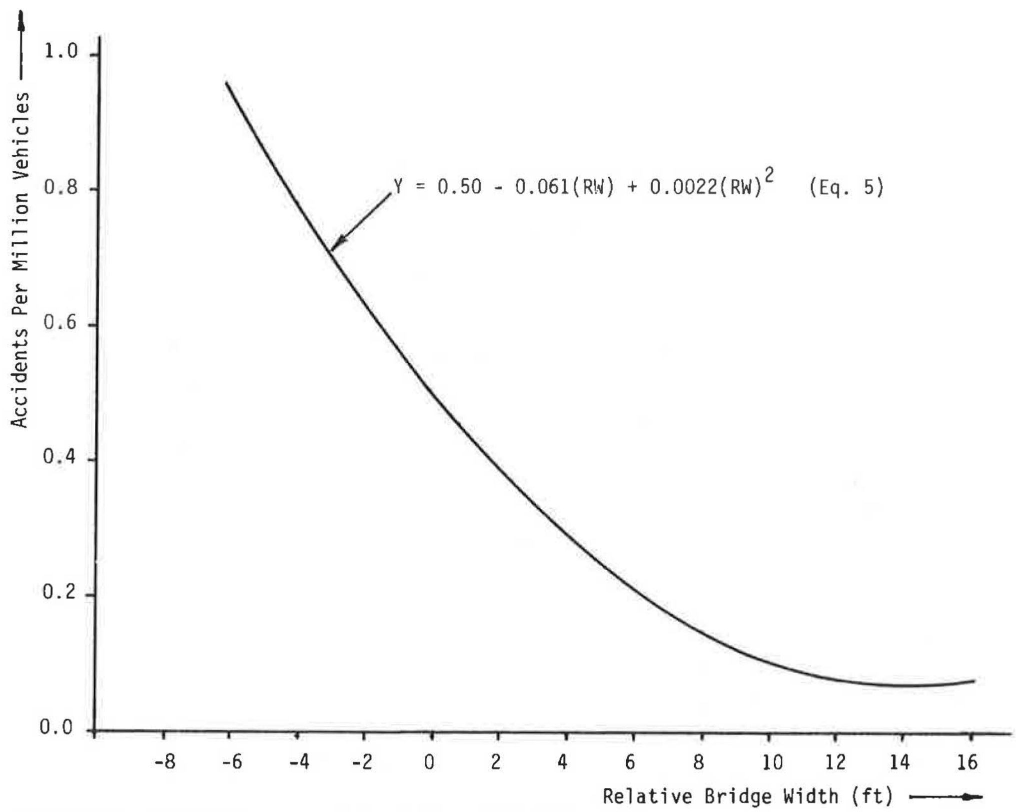


FIGURE 5 Accident rate by relative bridge width (16).

ft, the curve had a steep slope, indicating a rapidly increasing accident rate with decreasing relative widths. The results suggest that a minimum of 3-ft-wide shoulders should be provided for bridges on rural two-lane highways; wider shoulders would be of little additional safety benefit.

The study has some problems, such as the reliability of accidents identified as bridge-related in the accident reports. There is some concern over the small sample size for very narrow or very wide bridges because more than 98 percent of the bridges are located on roadways 18 to 26 ft wide, as illustrated by the drastic difference between the two equations. Also, there is the question of the effect of responsible factors other than bridge width.

In both the Colorado *(15)* and the Turner *(16)* studies, the best fitting models are quadratic curves so that a minimum accident rate occurs at some bridge width or relative bridge width. One question that arises is whether the upturn in the curves is an artifact of the curve-fitting procedure or a true indication that accident rates would increase if the bridge width or relative bridge width is increased to beyond the optimum (i.e., the width with the lowest accident rate). Intuition would suggest that the accident rate should continue to decrease with increasing bridge width or relative bridge width, and the concept of an optimum bridge width or relative bridge width is not supportable. However, the data suggest the contrary.

Similar trends are found in several studies *(2, 14–16)* indicating that there is little change or even slight increase in the accident rate when the bridge width or relative bridge width is increased to beyond a certain point. There are no apparent explanations for such an upturn in the accident rate, and there is simply insufficient information available from the literature to answer this question more thoroughly. The best guess is that there may not necessarily be an optimum bridge width or relative bridge width that has the minimum accident rate. Nevertheless, there is sufficient evidence in the literature to suggest that there is a certain bridge width or relative bridge width beyond which the incremental safety benefit would be minimal.

The results of some of the preceding studies on bridge safety have been synthesized in a number of reports *(17–20)*. Most of these syntheses simply reported the relationships developed from previous studies or used the relationships for their own applications. The relationships were accepted at face value without any critical review. One exception is the study by Jorgensen-Westat *(17)* in which relationships between relative bridge width and accident rate, injury rate, and accident property damage were developed using data from the studies by Gunnerson *(13)* and Fritts *(8)*. Accident reduction factors were developed from these relationships so that expected benefits from bridge widening could be estimated. There are severe problems associated with how the data were used in developing these relationships. Thus, the findings and conclusions of this study are highly questionable.

In another study Jorgensen *(18)* reported that wider lane and pavement widths on bridges resulted in lower accident rates, citing the studies by Jorgensen-Westat *(17)* and Gunnerson *(13)*, and that wider shoulders on bridges or greater lateral clearance reduced the accident rate on Interstate highways, citing the study by Cirillo *(14)*. The study also developed the logical relationships (i.e., nonestablished) that accident rate would decrease as lane width on a bridge increases with little difference between 11- and 12-ft lanes, and that accident rate would increase on multilane highways if the right shoulders would not accommodate a parked vehicle off the travel lanes.

McFarland et al. *(19)* used the relationships developed in the Colorado study *(15)* and the Jorgensen-Westat study *(17)* to estimate accident reduction factors and the effectiveness of widening a bridge. The relationships were accepted at face value with no critical review although the authors appropriately pointed out that the results were based on numerous assumptions and could be subjected to substantial error.

In a recent report on synthesis of safety-related research *(20)*, three studies were cited: Jorgensen-Westat *(17)*, McFarland et al. *(19)*, and Woods et al. [note that this study was part of the study by Ivey et al. *(6)*]. Again, the results of the cited studies were reported at face value in the synthesis with no critical review.

CONCLUSIONS AND DISCUSSION

The major conclusions that can be drawn from the studies reviewed are summarized next.

Bridge width should, at the very minimum, be as wide as the approach traveled way width; that is, there should be no lane width reduction on the bridge. Accident rates are shown to be significantly higher for bridges with width less than the approach traveled way width.

For bridges on two-lane highways, there is general agreement among several studies that the bridge width should desirably be at least 6 ft wider than the traveled way width; that is, 3-ft shoulders should be provided. There appears to be little change in accident rate or safety benefits gained when the bridge width is increased further. This conclusion is also supported by results from studies of driver behavior. The minimum desirable bridge width is therefore 26 ft for roadways with 10-ft lanes, 28 ft for 11-ft lanes, and 30 ft for 12-ft lanes. Bridges of similar width were found to have higher accident rates on roadways with no approach shoulders than those on roadways with approach shoulders. Also, for roadways with approach shoulders, accident rates are highest for bridges with greater than 50 percent shoulder reduction. Accident rates decrease significantly for bridges with 1 to 50 percent shoulder reduction. However, there is little difference in accident rates between bridges with 1 to 50 percent shoulder reduction and those with no shoulder reduction. This suggests that the shoulder width on a bridge should be at least one-half of that of the approach roadway.

In summary, the shoulder width on a two-lane bridge should desirably be at least 3 ft or one-half of the approach roadway shoulder width, whichever is greater. The desirable bridge width is simply twice the sum of lane plus shoulder widths.

Relationships between accident rate and the width or relative width of bridges on two-lane highways were developed in several studies. The best available relationship is judged to be the quadratic equation developed by Turner *(16)* as follows:

$$Y = 0.50 - 0.061\,(RW) + 0.0022\,(RW)^2$$

where Y is the number of accidents per million vehicles, and RW is the bridge relative width in feet.

Table 3 gives the expected accident rates for various relative bridge widths as calculated from this equation. It should be cautioned, however, that the accident rates were based on "bridge-related" accidents and may be subjected to substantial error. Thus, the accident rates should be viewed in relative terms and not as absolute measures. Using these expected accident rates, it is possible to estimate the potential percent increases in accident rate for bridge widths between the minimum (i.e., equal to the traveled way width) and the desirable minimum (i.e., shoulder width of 3 ft or one-half of the approach shoulder width), as given in Table 3.

TABLE 3 Expected Percent Increase in Accident Rate for Relative Bridge Widths Less than the Desirable Minimum

Relative Bridge Width (ft)	Expected No. of Accidents per Million Vehicles	Desirable Minimum Relative Bridge Width (ft)		
		6	8	10
0	0.50	139	239	381
1	0.44	111	199	323
2	0.38	84	161	270
3	0.33	60	127	221
4	0.29	38	95	177
5	0.24	18	67	137
6	0.21	0	42	101
7	0.17	–	19	69
8	0.15	–	0	42
9	0.12	–	–	18
10	0.10	–	–	0

NOTE: Dashes indicate not applicable.

For approach shoulder widths of 0 to 6, 8, and 10 ft, the desirable minimum relative widths are 6, 8, and 10 ft, respectively. The expected percent increase in accident rate is then calculated as

$$\text{Percent increase} = \frac{R(X) - R(DM)}{R(DM)} \times 100\% \tag{6}$$

where $R(X)$ is the expected accident rate for relative width X, and $R(DM)$ is the expected accident rate for desirable minimum relative width.

For two-lane twin structures, the extent of information available from the literature is more limited and is categorical in nature. Mak and Calcote *(2)* reported that accident rates decreased sharply when shoulder width was increased from greater than 50 percent shoulder reduction to between 1 and 50 percent. However, there was little difference in accident rates between bridges with 1 to 50 percent shoulder reduction and those with no shoulder reduction. Cirillo *(14)* reported that accident rates were lowered significantly when the shoulder width increased from less than 6 ft to between 6 and 9 ft and remained little changed for shoulder widths of beyond 9 ft. A study in Colorado *(15)* revealed that accident rates were much higher for bridges with 30-ft width (i.e., a total of 6 ft of shoulders) than those with 38-ft width carrying the full approach shoulders (i.e., 10-ft right shoulder and 4-ft left shoulder). A driver behavior study *(5)* indicated that vehicle lateral movement toward the centerline was minimal for a shoulder width of 6 ft.

The data suggest that the minimum shoulder width on a bridge should be the greater of 6 ft or one-half of the approach shoulder width. A minimum bridge width of 33 ft, consisting of two 12-ft lanes, a 3-ft left shoulder, and a 6-ft right shoulder, is therefore recommended for two-lane twin structures. There is insufficient information available from the literature to establish more detailed relationships between accident rate and bridge width or relative bridge width.

Bridge width appears to have some marginal effect on accident severity for two-lane twin structures, with higher severity for bridges with greater than 50 percent shoulder reduction, but does not appear to have any effect on accident severity for two-lane single structures.

In using the study findings presented in this paper, the following considerations should be borne in mind:

1. The relationships represent the best information available from the literature. However, it should be emphasized that these relationships are usually fairly weak and lacking in specificity, such as by ADT groups and highway type.

2. The relationships developed did not take into account factors other than bridge width and relative bridge width that may influence accident rates at bridge sites, such as structure length, type (e.g., deck versus truss), presence or absence of curb on the bridge, approach alignment, pavement surface condition, traffic mix, operating speed, and so forth. Unfortunately, there is insufficient information available from the literature to determine the effects of these factors on accident rates and their interactions with bridge width or relative bridge width. Nevertheless, these other factors should be taken into consideration when selecting the appropriate bridge width.

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Effect of Pavement/Shoulder Drop-Offs on Highway Safety

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Drop-offs at the pavement/shoulder (or shoulder/roadside) edge have been recognized as a potential highway safety problem for many years. In the American Association of State Highway Officials' (AASHO) 1954 publication *A Policy on Geometric Design of Rural Highways,* the subject of drop-offs is covered as follows (*1*, p. 205):

> Unstabilized shoulders frequently are hazardous because the elevation of the shoulder at the pavement edge may be several inches lower than the pavement.

This passage was expanded in the 1965 edition of the publication to add further caution as follows (2, p. 239):

> Unstabilized shoulders frequently are hazardous because the elevation of the shoulder at the pavement edge tends to become one-half to several inches lower than the pavement.

These statements in the AASHO policies must have come from the general perceptions of the policy writers because no research on the differential effects of drop-off heights and shapes was available before about 1977. Research since 1977 has demonstrated that the probability of severe consequences from a pavement/shoulder drop-off traversal are a function of drop-off height and shape and vehicle speed and reentry angle.

CHARACTERIZATION OF THE PAVEMENT/SHOULDER DROP-OFF TRAVERSAL

A pavement/shoulder drop-off traversal occurs when the driver inadvertently leaves the travel lane and drops onto a lower shoulder. Depending on the severity of the vehicle departure angle and the driver's level of surprise and response, seven general outcomes are possible as shown in Figure 1.

If the departure angle is high, recovery is quite unlikely, and a collision of some type on the roadside is probable (Outcome 7). If the departure angle is shallow to moderate, the driver has some potential to respond with a recovery attempt (Outcomes 1–6).

If the driver steers a high reentry angle the vehicle will skid on the shoulder resulting in immediate loss of control (Outcome 6). If the driver is particularly alert and knowledgeable about the potentially hazardous effects of drop-offs, he may be able to steer parallel to the drop-off and slow the vehicle to a stop on the shoulder (Outcome 1.

If the driver steers a moderate reentry angle that is just high enough to remount the pavement/shoulder drop-off, he will usually recover within his travel lane (Outcome 3). For moderate reentry angles that are higher than the minimum necessary to remount, particularly at higher speeds, the vehicle will either encroach on adjacent travel lanes or skid out of control, depending on the severity of the driver's steering response (Outcomes 4 and 5).

If the driver steers a shallow reentry angle, the vehicle's tires will either scrub along the drop-off face and either subsequently remount the drop-off (Outcomes 3 to 5) or possibly rebound out of control (Outcome 2), depending on the driver's steering response.

A scrubbing reentry occurs when the wheel that contacts the drop-off edge has insufficient normal velocity to overcome the retarding force produced by the tire and edge contact. The term "scrubbing" describes the near-parallel traversal of the tire along the drop-off edge in which a relatively large contact area occurs between the tire

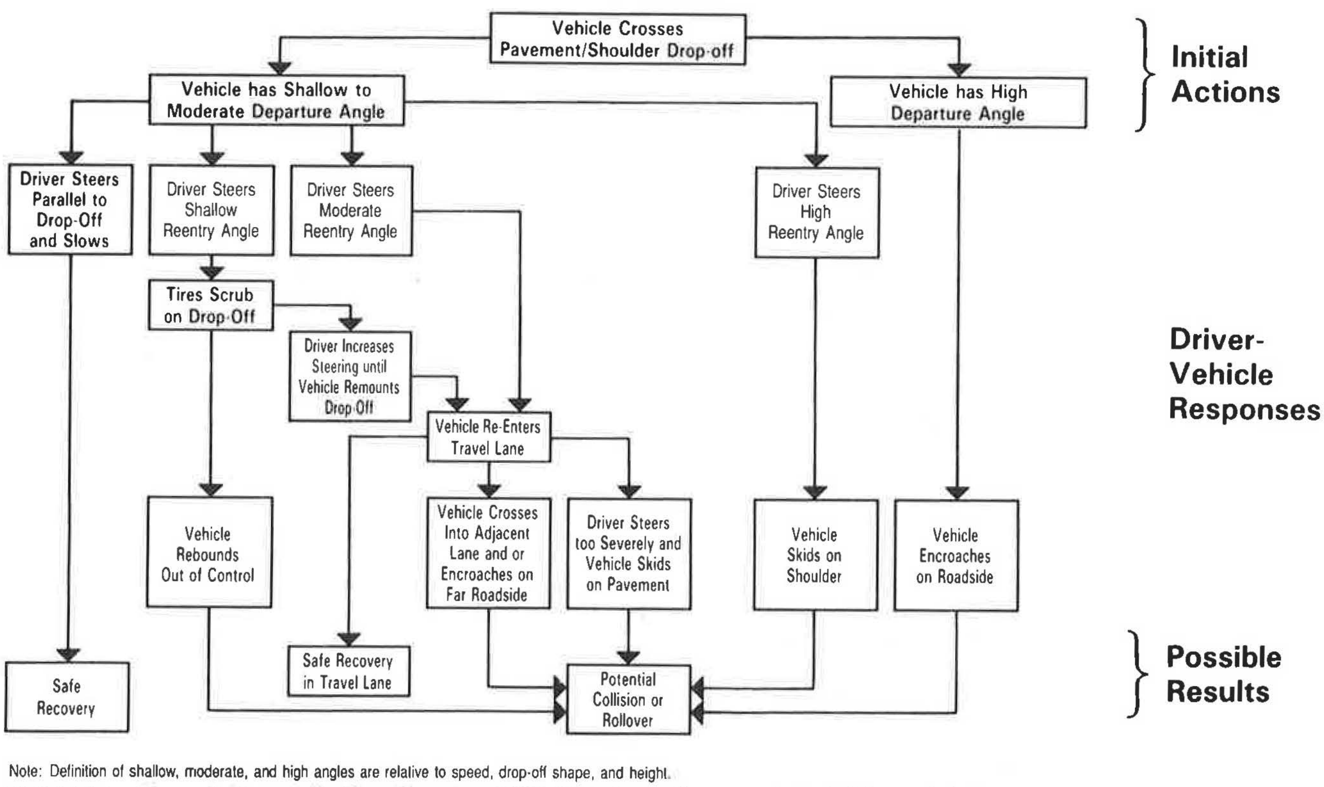

FIGURE 1 General characterization of pavement/shoulder drop-off traversal. Definitions of shallow, moderate, and high angles are relative to speed, drop-off shape, and height.

sidewall and the drop-off edge. The wheel develops a high resistance to mounting the pavement/shoulder drop-off, and the driver continues to increase the steer angle in a further attempt to mount the drop-off. The contact of the tire and the drop-off edge continues until the front-wheel steer angle is sufficient to overcome the retarding force and to create a sufficient side force at the unobstructed (left) front wheel to lift the obstructed (right) tire over the drop-off. Once the obstructed front tire has mounted the drop-off, the large steer angle produces a large lateral acceleration and a large yaw velocity that combine to produce rapid lateral movement. This lateral movement will continue until the driver reverses the steer angle and the vehicle has time to respond to the steer reversal.

The responses produced by scrubbing reentries constitute the primary hazard associated with pavement/shoulder drop-offs. At higher speeds and drop-off heights, the results are either excessive lateral encroachments on adjacent lanes or opposite roadsides (Outcome 4) or loss of control because of excessive steering corrections (Outcome 5).

SCOPE AND OBJECTIVES

The purpose of this paper is to synthesize the state of the art on the safety of pavement-shoulder drop-offs so that practical guidelines can be developed for their treatment in the improvement of existing highways under the current resurfacing, rehabilitation, and restoration (RRR) process.

A pavement/shoulder drop-off may be an existing condition on a candidate RRR highway or it may be created by a RRR pavement overlay project. The existing drop-off may have been created by a previous pavement overlay or may have evolved from shoulder wear, settlement, or erosion. The two questions for considering existing drop-offs in the RRR process are

1. What combination of drop-off shape and height constitute an intolerable hazard?
2. What is the most cost-effective method for treating intolerable drop-offs?

An all too common practice in many areas is to leave a pavement/shoulder drop-off when overlaying an existing highway, particularly one with unpaved shoulders. The RRR questions to be addressed are

1. What are the drop-off heights and shapes that are tolerable for safety?
2. What are the design and construction methods that can be used to produce tolerable pavement/shoulder edge conditions?

A critical review is presented of available studies on pavement/shoulder drop-offs for the purpose of identifying the combinations of shape and height that are tolerable for various highway design speeds.

CRITICAL REVIEW AND ANALYSIS

Research on the safety of pavement/shoulder drop-offs is limited. A review of the literature produced only five recent references with any reasonable contribution to the state of the art.

These five studies are comparatively analyzed in an attempt to synthesize the body of knowledge. Various aspects of the subject are discussed in separate paragraphs.

Role of Pavement/Shoulder Drop-Offs in Highway Accidents

The literature on the relationship between highway accidents and pavement/shoulder drop-offs is limited. This relationship is discussed in only two studies.

In 1976 Ivey and Griffin examined all of the 15,968 single-vehicle accidents that occurred in North Carolina during 1974 for the purpose of studying the contribution of surface discontinuities (bumps, dips, rocks, holes, drop-offs, etc.) *(3)*. Computerized police officers' narratives for all of these accidents were examined for any one of 19 key words that denoted a surface discontinuity. Some 566 (3.5 percent) of these accidents were associated with surface discontinuities; of these, 154 (1.0 percent of total) appear to have been related to drop-offs.

Klein et al. reviewed accident data from three different sources to analyze the frequency with which surface discontinuities contribute to highway accidents *(4)*. In this analysis, the authors concluded that the most significant discontinuity was the pavement/shoulder drop-off. Depending on the source of data, surface discontinuities contributed to 0.8 to 2.6 percent of all accidents, and accidents involving pavement/shoulder drop-offs ranged from 0.2 to 1.3 percent of all accidents.

Although these studies indicate a relatively small percentage of total accidents that involve pavement/shoulder drop-offs, they did not attempt to determine the relative exposure to drop-off conditions. If only highways with pavement/shoulder drop-offs were considered, the contribution of drop-offs to the accident experience might be substantial.

Probability of Severe Consequences

Klein et al. conducted several full-scale tests using "naive" drivers *(4)*. They were interested in the statistical distribution of consequences when these drivers encountered a 4.5-in. vertical face drop-off and tried to recover. Of 73 tests, 53 percent produced tire scrubbing on the drop-off edge. Of those tests that produced scrubbing, 56 percent resulted in exceedance of the 12-ft lane boundary after mounting the drop-off. The likelihood of a lane boundary exceedance after scrubbing was strongly correlated with speed. Forty-seven percent of the tests resulted in a nonscrubbing reentry, none of which produced lane exceedance.

These results may be somewhat misleading because the report did not document the distribution of speeds used in the tests. Therefore, the probability of scrubbing in a particular test may be higher or lower than indicated earlier depending on the speed used. These studies also did not account for the element of surprise.

Shallow-Angle Approach

Klein et al. conducted several full-scale, shallow-angle reentry tests to evaluate the potential for mounting a vertical face drop-off when a vehicle is not initially scrubbing *(4)*. Various drop-off heights were tested with various reentry angles and speeds to produce a relationship between drop-off height and the minimum normal speed necessary to mount the drop-off. Normal speed is that component of vehicle speed perpendicular to the pavement/shoulder edge.

This relationship between vertical face drop-off height and the minimum normal speed necessary to mount the drop-off is shown in Figure 2. This relationship indicates

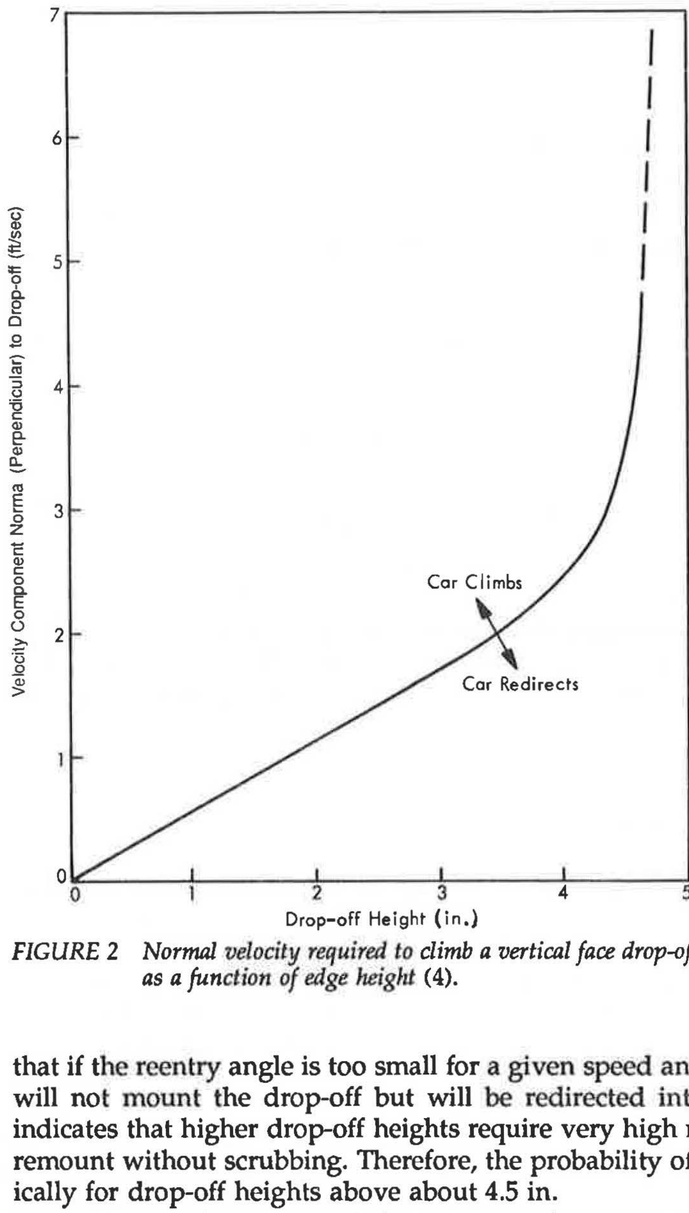

FIGURE 2 Normal velocity required to climb a vertical face drop-off as a function of edge height (4).

that if the reentry angle is too small for a given speed and drop-off height, the vehicle will not mount the drop-off but will be redirected into a scrubbing mode. It also indicates that higher drop-off heights require very high normal speeds for vehicles to remount without scrubbing. Therefore, the probability of scrubbing increases dramatically for drop-off heights above about 4.5 in.

This basic relationship of the pavement/shoulder drop-off maneuver shown in Figure 2 was at least partly validated by Graham and Glennon who used HVOSM computer simulation techniques to study vertical face drop-offs *(5)*. Unfortunately, no such relationship has been established for any other drop-off shape.

Nonscrubbing Reentry

Four separate studies indicate that successful recovery from a pavement/shoulder drop-off maneuver is possible even at fairly high speeds. Tests conducted by Stoughton et al. *(6)* with various sizes of passenger cars and a standard pickup truck indicated that a professional driver had little difficulty recovering from a drop-off with a typical irregularly rounded edge of asphalt pour, even at speeds of 60 mph and drop-off heights of 4.5 in. These tests appear to have been carefully controlled to avoid scrubbing.

The full-scale tests reported by Klein et al. indicated that the naive drivers who did not scrub on a 4.5-in. vertical face drop-off were able to successfully recover within the travel lane *(4)*. Only 47 percent of the tests conducted resulted in a nonscrubbing reentry.

Zimmer and Ivey conducted full-scale nonscrubbing tests using a professional driver to test drop-offs of various heights with a 1.5-in. tapered corner *(7, 8)*. The averaged subjective severity ratings made by the driver indicate that he was able to recover successfully with little difficulty at speeds up to 55 mph and drop-offs as high as 4.5 in. These tests indicated little sensitivity to type of vehicle for various sizes of passenger cars and a standard pickup truck.

Graham and Glennon used analytical methods to show the reentry angle boundaries of successful nonscrubbing recovery for vertical face drop-offs *(5)*. From the relationship shown in Figure 2, the minimum reentry angle to avoid scrubbing was solved to form the lower boundary of successful recovery for any combination of vehicle speed and drop-off height. For the upper boundary, Graham and Glennon developed an analytical model that determined the maximum reentry angle that would allow the vehicle to still recover within the lane as a function of lane width. To exercise this model, constraints of 0.3*g*'s lateral acceleration and 0.7-sec driver perception-reaction time were used. Figure 3 shows two example plots from those calculations.

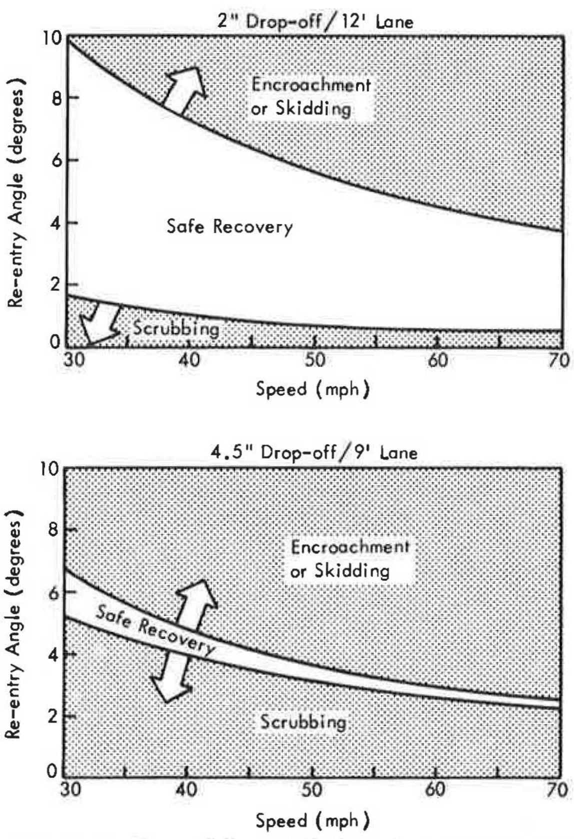

FIGURE 3 Example boundaries for safe reentry angles for traversal of vertical face drop-offs (5).

Although the exact form of the analytical model and the constraint values used might be debated, the analysis indicates that the reentry angle range available for successful nonscrubbing recovery decreases as speed and drop-off height increase and as lane width decreases.

All of the studies indicate the possibility of successful nonscrubbing recoveries, even at high speeds and drop-off heights if reentry angles are maintained within a precise range. However, the probability that this type of maneuver can be performed by a nonprofessional driver, particularly one who is surprised when his right-front wheel suddenly drops, has not been addressed by the research but probably becomes smaller with increasing speed and drop-off height.

Scrubbing Reentry

For those studies that included tests of the scrubbing reentry, this maneuver was found to be the most hazardous type of reentry. The reason for this hazard is best explained by the results of Klein et al. shown in Figure 4. For a drop-off with only a 0.5-in. radius

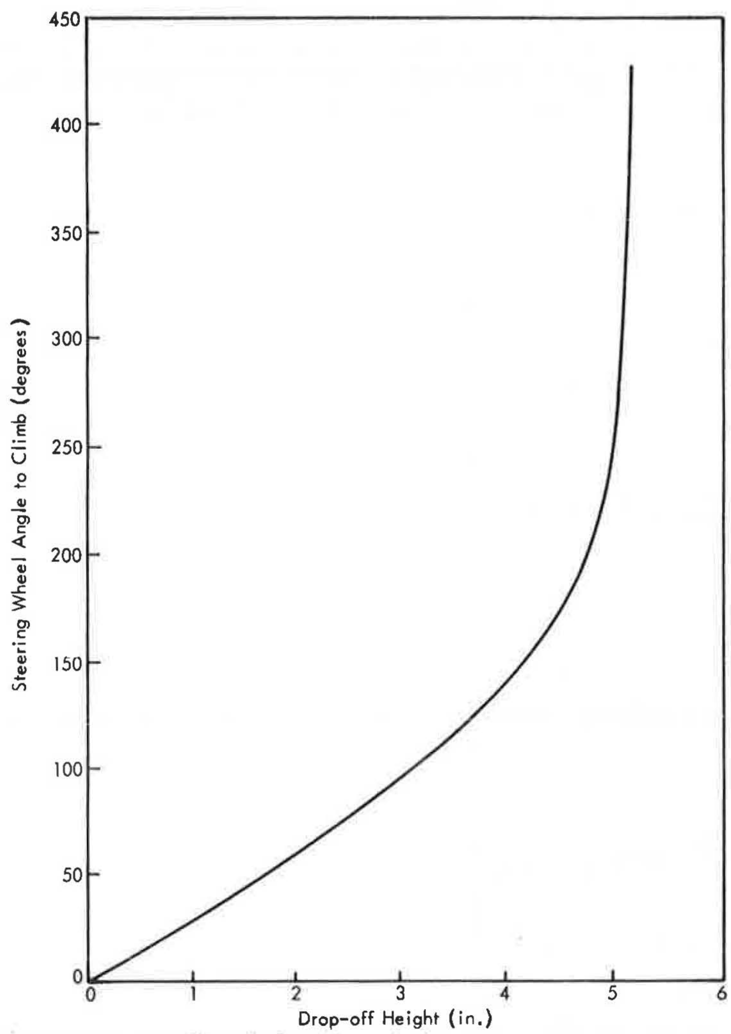

FIGURE 4 Steering wheel angle required to climb various edge heights from a scrubbing condition (4).

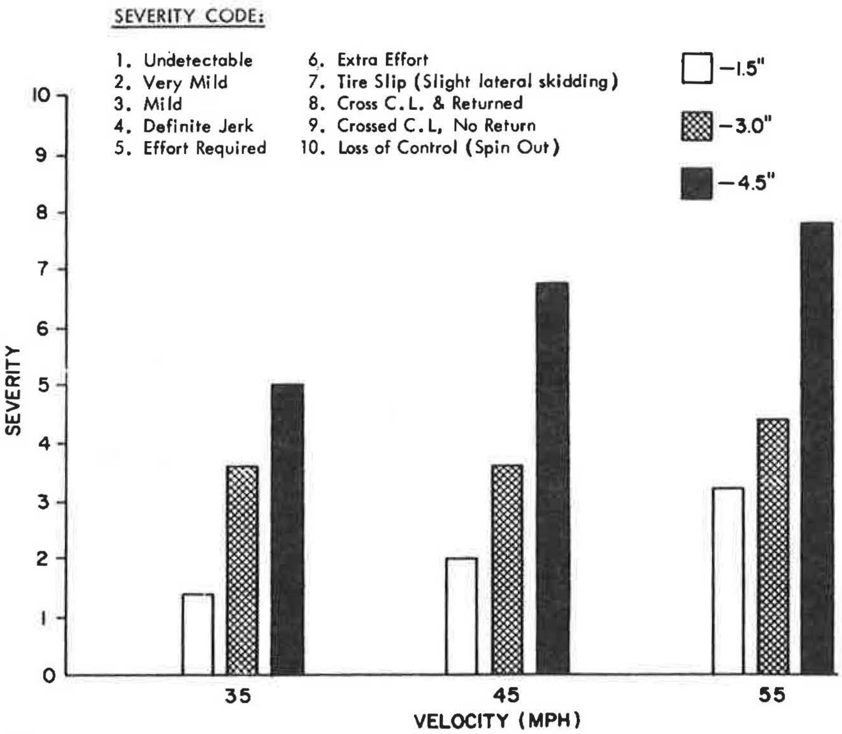

FIGURE 5 Severity ratings for traversals of tapered edge drop-offs (7–9).

corner rounding, this figure shows that as drop-off heights exceed about 3 in., large steering wheel inputs are required to mount the drop-off. Under these conditions, when the wheel mounts the drop-off, the large steer angle produces a large lateral acceleration and yaw velocity that combine to produce rapid lateral movement.

Zimmer and Ivey provide the most complete analysis on scrubbing reentry *(7, 8)*. Figure 5 shows the severity ratings of a professional driver averaged over various vehicles and tires for a large matrix of full-scale scrubbing tests conducted on a drop-off with a 1.5-in. corner taper. These results must be interpreted as average (not critical), alerted (not surprised), professional (not nonprofessional) driver responses; as such they may indicate lower maneuver severities than can be expected on the highway. Nevertheless, these results do indicate undesirable consequences for 4.5-in. drop-offs at speeds of 45 mph and above.

Table 1 includes the limited data available from three of the studies showing more detailed results from selected scrubbing tests that exhibited lane exceedance or high lateral acceleration, or both. It can be seen from Table 1 that the scrubbing reentry maneuver becomes more severe as the drop-off approaches a full vertical face, as the drop-off height increases, and as the vehicle speed increases.

APPLICATION OF RESULTS TO RRR PRACTICE

When drivers experience a nominal encroachment onto a flush shoulder, their safe recovery within the travel lane depends on their ability to avoid steering a severe reentry angle that will either cause the vehicle to encroach on adjacent lanes or cause them to lose control within the lane when secondary steering correction is attempted.

One characterization of this upper boundary of reentry angles has been shown to be a function of vehicle speed and lane width as shown in Figure 6.

When a drop-off is introduced at the pavement/shoulder edge, the upper boundary shown in Figure 6 is joined by a lower boundary for safe reentry angles imposed by the

TABLE 1 Available Results of Scrubbing Tests

Source	Drop-Off Shape (in.)	Drop-Off Height (in.)	Vehicle Speed (mph)	Driver Type	Result
Zimmer and Ivey *(1)*	1.5 corner taper	4.5	45	Professional	Lateral acceleration of 0.74
Zimmer and Ivey *(1)*	1.5 corner taper	4.5	55	Professional	Lane exceedance and lateral acceleration of 0.79
Zimmer and Ivey *(7)*	0.75 corner radius	4.5	45	Professional	Lane exceedance and lateral acceleration of 0.71
Zimmer and Ivey *(7)*	0.75 corner radius	4.5	55	Professional	Lane exceedance and lateral acceleration of 0.88
Klein et al. *(4)*	0.5 corner radius	3.5	40	Unknown	Loss of control
Graham and Glennon *(5)*	Vertical face	2.0	30	Modeled	Minor lane exceedance and lateral acceleration of 0.30
Graham and Glennon *(5)*	Vertical face	2.0	45	Modeled	Major lane exceedance and lateral acceleration of 0.60
Graham and Glennon *(5)*	Vertical face	3.0	45	Modeled	Major lane exceedance and lateral acceleration of 0.80

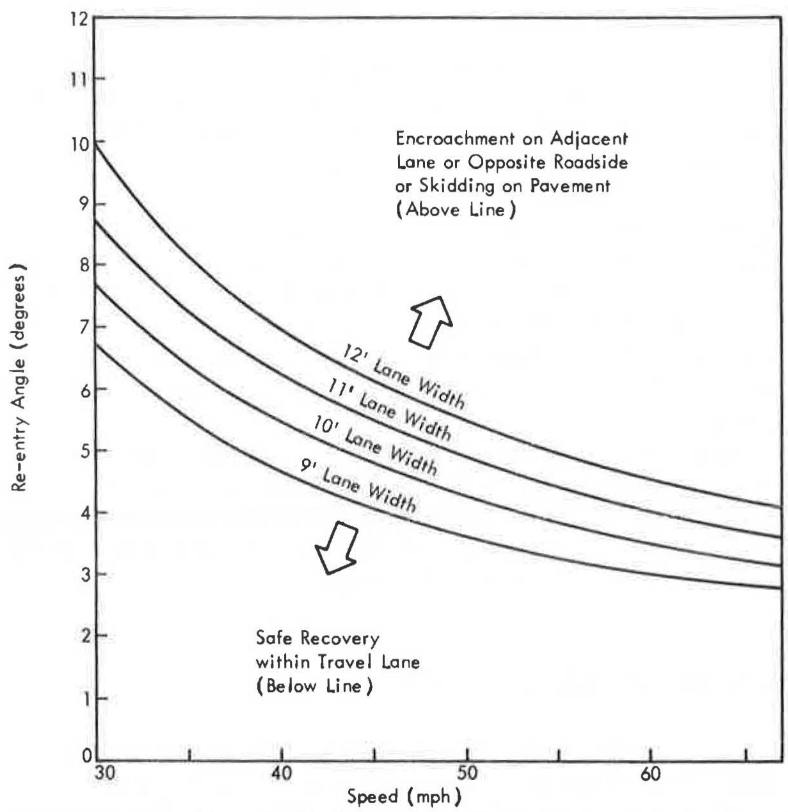

FIGURE 6 Maximum safe reentry angle for shoulder traversal as a function of speed and lane width (5).

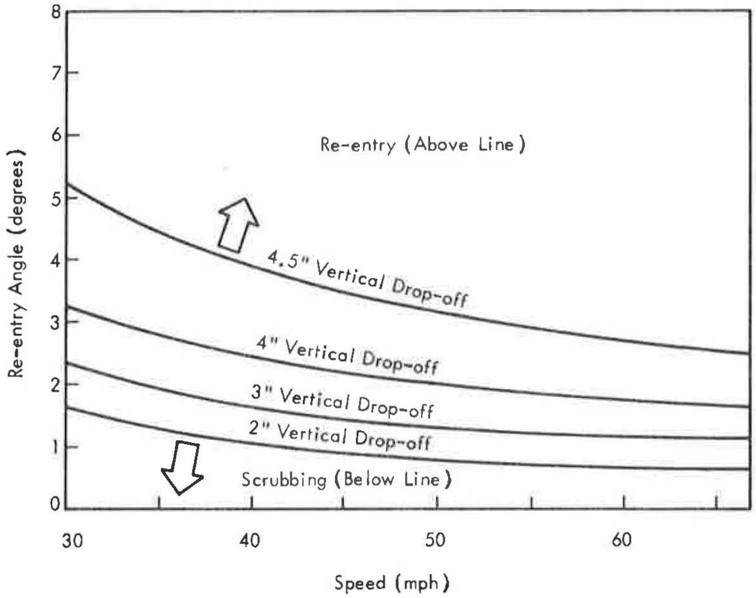

FIGURE 7 Minimum safe reentry angle for traversal of vertical face pavement/shoulder drop-off as a function of speed and drop-off height (4, 5).

potential for hazardous scrubbing reentry maneuvers, which are more likely as the drop-off height increases. Figure 7 shows this lower boundary relationship for vertical face drop-offs.

For any combination of vertical-face drop-off height and lane width, the upper and lower boundary conditions shown in Figures 6 and 7 can be combined to depict the range of safe reentry angles for any vehicle speed. Figure 3 shows a sample of these composite plots that indicates the high likelihood of a severe reentry maneuver, particularly for higher drop-off heights, higher speeds, and narrower lanes.

The available literature does not provide a refined set of data that can precisely predict the accident consequences of a pavement/shoulder drop-off maneuver. However, it does provide some useful insights into the safety of pavement/shoulder drop-offs as a function of vehicle speed, drop-off shape and height, and lane width. From the results reported in this paper, the following general conclusions can be drawn:

1. The most obvious hazard associated with pavement/shoulder drop-offs occurs when a driver tries to recover from a scrubbing condition.
2. The probability of a scrubbing reentry at a pavement/shoulder drop-off increases as *(a)* the drop-off face approaches a full vertical edge and *(b)* the drop-off height increases.
3. The probability of a successful recovery from a drop-off maneuver decreases as *(a)* the drop-off face approaches a full vertical edge, *(b)* the drop-off height increases, *(c)* the vehicle speed increases, and *(d)* the lane width decreases.
4. The severity (yaw velocity, lateral encroachment, etc.) of a scrubbing reentry maneuver increases as the drop-off shape approaches a full vertical edge and as the drop-off height and vehicle speed increase (see Table 1 and Figure 5).
5. A 5-in. drop-off height is a practical maximum to prevent hazardous undercarriage contact on most vehicles.

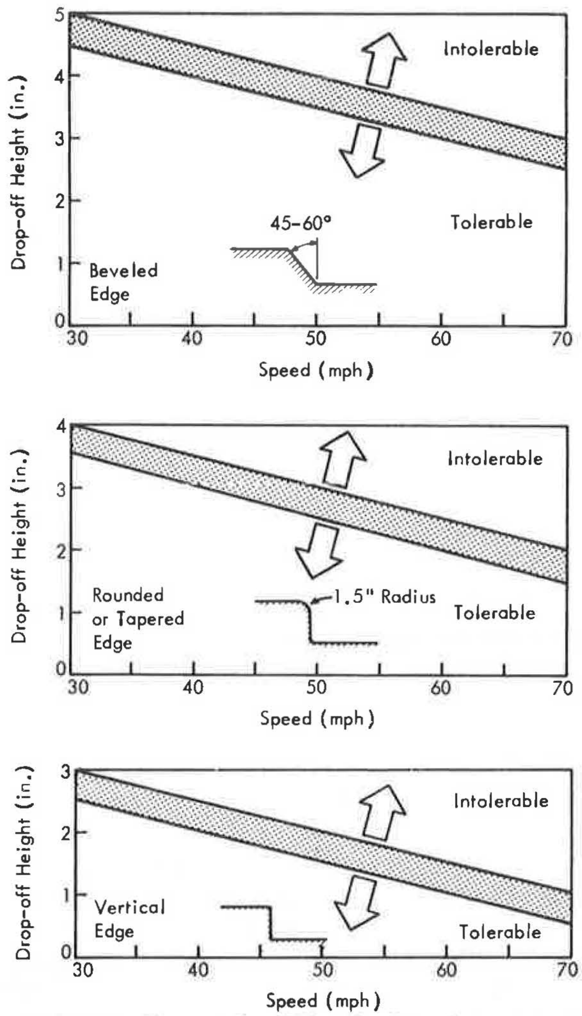

FIGURE 8 Suggested guidelines for determining tolerable pavement/shoulder drop-off heights.

Using these general conclusions and a safety-conservative interpretation of the fairly sparse results of the available studies, the data shown in Figure 8 suggest a provisional guideline for determining if existing drop-offs are tolerable on RRR candidate highways. As can be observed in Figure 8, the suggested tolerable drop-off height decreases with speed as the drop-off shape approaches a full vertical edge. The shaded band on each graph allows for *(a)* some discretion because of the uncertainty of available data, and *(b)* the slight variance of criticality associated with lane width whereby narrow lanes would have lower tolerable drop-off heights than wide lanes.

The suggested guidelines are based on the studied dynamic sensitivity of passenger cars and pickup trucks of various sizes. Lower tolerable drop-off heights may be appropriate on roadways that carry significant proportions of heavy trucks or motorcycles.

Using Figure 8 (or a similarly derived guideline), if an existing pavement/shoulder drop-off is found to be intolerable for prevailing highway speeds, then one of two alternatives would be reasonable to reduce the hazard. First, material (preferably stabilized) can be added to raise the shoulder elevation to a tolerable level (preferably flush). Second, the edge shape can be changed to more closely approximate a 40 to 60 degree continuous taper by either adding an asphalt wedge or grinding the existing edge. Where tolerable pavement/shoulder drop-offs are allowed to remain, some consideration should also be given to the use of warning signs.

Figure 8 also provides guidance when placing a pavement overlay on RRR projects. It is desirable that such an overlay should be flush with the shoulder, but if project economics prohibit this condition, the created pavement/shoulder drop-off height should be as low as possible and the edge shape should be as close to the 45 to 60 degree taper as possible, all within tolerable limits. Where tolerable pavement/shoulder drop-offs are created, some consideration should be given to the use of warning signs.

Most of the conclusions in this study also apply to the drop-offs sometimes found at the shoulder/roadside edge. However, because of the lower probability of encroachment at the shoulder/roadside edge and a greater effective recovery width compared with the pavement/shoulder edge, the suggested guidelines for Figure 8 might be relaxed somewhat to allow higher tolerable drop-offs for this condition, particularly for the wider shoulder widths.

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Effect of Alignment on Highway Safety

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The highway designer working on the plans for a new highway will always strive for gentle highway alignment consisting of flat horizontal curves, noncritical grades, and long vertical curves. During the last 45 years, this process has been guided by the design policies of the American Association of State Highway and Transportation Officials (AASHTO), which has defined acceptable limits on these design features based on perceived safety and operational effects. Although cost-effectiveness has always been an underlying basis for design, these design limits have been largely governed by acceptable performance criteria rather than cost-effectiveness considerations.

When considering the safety enhancement of resurfacing, restoration, and rehabilitation (RRR) projects, the designer has a different perspective than when he is designing a new highway. Changes in existing alignment are very expensive and require careful cost-effectiveness comparisons with competing alternatives for funds. For this reason, it is important to know the expected safety benefits for any proposed changes to existing alignment.

This critical review of the literature was undertaken to synthesize the available knowledge on the relationships between highway alignment and safety in order to provide guidance in selecting cost-effective improvements that will enhance safety on RRR projects. This review was basically limited to the physical aspects of highway alignment that relate to vehicle dynamics. Another review evaluating the safety aspects of sight distance considers the safety relationship between alignment and stopping sight distance.

The available research on the accident effects of horizontal and vertical alignment is limited. A search of the literature produced 24 references that appear to have contributed to the state of knowledge. This body of literature is critically analyzed in the following sections of this paper.

HIGHWAY CURVES

Highway curves are a necessary and important element of nearly all highways. Their form has evolved from what appeared to be reasonable to the builder's eye to the more modern geometrically designed form of a circular curve with superelevation, cross-slope transitions, and often spiral transitions.

Despite a reasonably well-conceived design procedure, which considers a tolerable level of lateral acceleration on the driver, highway curves continually show a tendency to be high-accident locations. Several studies have indicated that highway curves exhibit higher accident rates than tangent sections, and that the accident rate increases as the degree of curve increases. But degree of curve may be just one element that is interdependent with other elements that together contribute to accident rate. For example, the sharpest curves tend to be located on lower quality highways; those with narrow roadways, narrow shoulders, marginal sight distance, hazardous roadsides, and the like.

The highway curve is one of the most complex features on the highways. The elements or aspects of highway curves given in Table 1 are all potential candidates for study in relating highway design to safety.

TABLE 1 Elements of Highway Curves

Element	Description
Horizontal alignment	Radius of curvature Length of curve Superelevation runoff length Distribution of superelevation runoff between tangent and curve Presence and length of transition Stopping sight distance around curve
Cross sectional	Superelevation rate Roadway width Shoulder width Shoulder slope Roadside slope Clear-zone width
Vertical alignment	Coordination of edge profiles Stopping sight distance on approach Presence and length of contiguous grades Presence and length of contiguous vertical curves
Other	Distance to adjacent highway curves Distance to nearest intersection Presence and width of contiguous bridges Level of pavement friction Presence and type of traffic control devices Type of shoulder material

Characteristics of Highway Curve Accidents

Few studies have attempted to characterize the accidents that occur on highway curves. A 1983 study of four states by Glennon et al. compared the accident experience on 3,304 rural two-lane curve segments to 253 rural two-lane tangent segments *(1)*. Each segment was 0.6 mi long and was carefully selected to minimize variance associated with intersections, bridges, nearby urban development, and nearby curvature. Figure 1 shows a summary of the significant characteristics of accidents on highway curves in this data base.

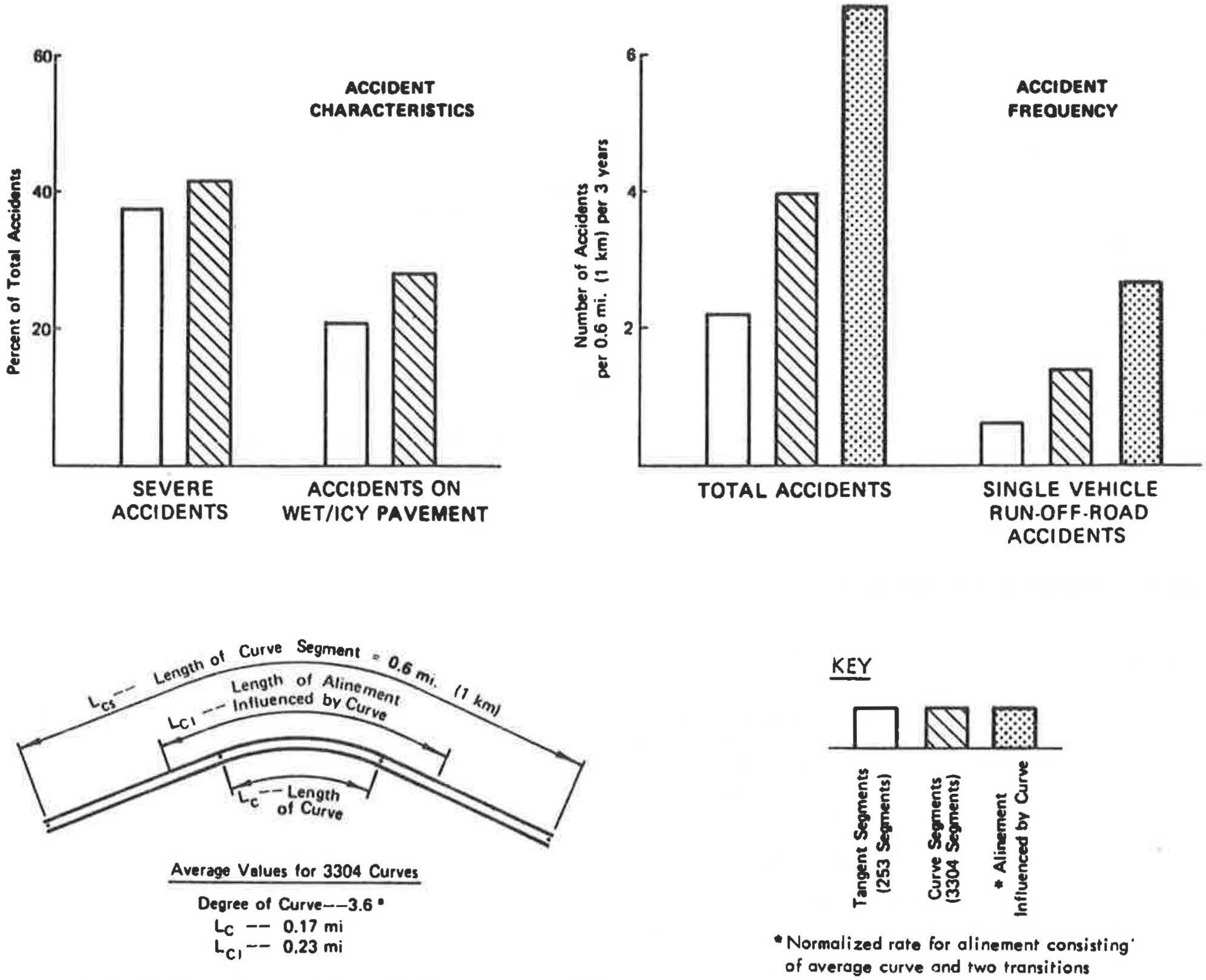

FIGURE 1 Accident characteristics on highway curves (1).

For example, curve segments have higher proportions of severe, wet-icy, and single-vehicle run-off-road accidents. Also, the accident rates on tangent and curve segments were used to compute an effective rate over a portion of the curve segments that included only the length of curve plus 150-ft transitions at each end of the curve. This computation yielded the following conclusions:

1. The average accident rate for highway curves is about three times the average accident rate for highway tangents.
2. The average single-vehicle run-off-road accident rate for highway curves is about four times the average single-vehicle run-off-road accident rate for highway tangents.

Although these conclusions are general, and may vary considerably by degree and length of curve, they do show that curves are substantially more hazardous than tangents and that single-vehicle run-off-road accidents are a prevalent aspect of curves. Another study by Perchonok et al. further defines the characteristics of single-vehicle run-off-road accidents on curves as follows *(2)*:

Degree of Curve	Percentage of Run-Off-Road Accidents	
	Outside	*Inside*
0–4	67	33
4.1–8	74	26
8.1–12	78	22
Above 12	84	16

When considering roadside safety countermeasures on curves, this table indicates a much stronger need for treatment on the outside of the curve.

Relationship of Accident Rate to Degree of Curve

Past research has generally indicated increasing accident rates with increasing degrees of curve. Figure 2, prepared by Jack E. Leisch and Associates *(3)*, shows the results of five studies *(4–8)*. Although these studies represent different road types and countries, there appears to be some general concurrence in their findings, however, they all have most of the following deficiencies:

1. The effect of traffic volume on accident rates and its intercorrelations with degree of curve was neither controlled nor accounted for.
2. One-year accident periods were too short to provide stable accident samples for highway curves.
3. Accident rates were computed by degree of curve ignoring the accident effects of length of curve. However, because curve length was used as part of the exposure base

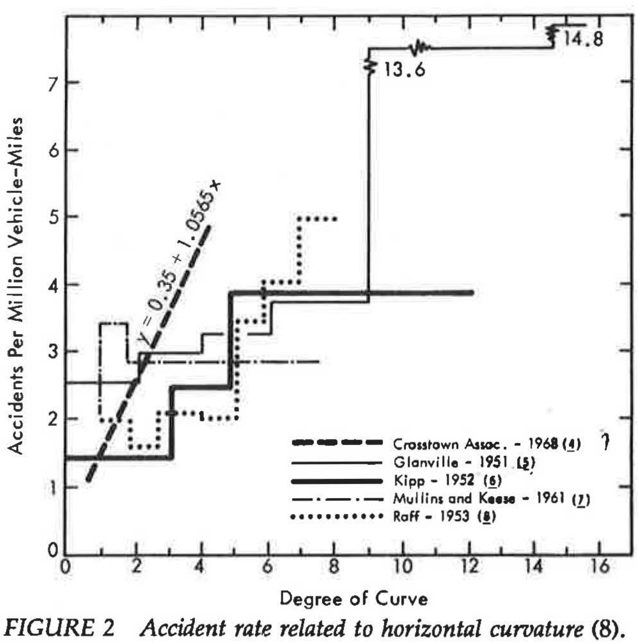

FIGURE 2 Accident rate related to horizontal curvature (8).

in the accident rate calculation, bias was introduced because sharp curves are usually short and flat curves are usually longer.

4. The accident effects of superelevation, lane width, shoulder width, sight distance, approach conditions, contiguous grades, intersections, and structures were not controlled or accounted for. That some of these elements are important was generally demonstrated by Kihlberg and Tharp who showed that intersections, grades, and structures increased accident rates on curves *(9)*. Also, many of the features mentioned earlier tend to be intercorrelated with degree of curve. In other words, sharp curves tend to be located on otherwise poorly designed roadways and flat curves tend to be located on otherwise well-designed roadways.

5. The accident effects of roadside hazards, slopes, and fixed objects, were not controlled or accounted for. These effects may be large when the proportion of roadside accidents on curves is considered. Roadside hazard is another feature that tends to be highly intercorrelated with degree of curve.

Other studies have considered highway curvature as one of several variables that potentially affects accident rates *(10–12)*. These studies have used either some form of multivariate analysis or a sufficiency rating scheme to identify the incremental effects of highway curvature. None of these studies offers any reliable method of determining the accident effects of changing horizontal curves.

A recent study that performed analysis of covariance on 0.6-mi sections that included one curve may shed some light on the net accident reduction associated with curve flattening *(1)*. Although the analysis of covariance results did not indicate any strong relationships, the raw regression found between the accident rate on the section and the degree of curve can be used as a comparison with the five studies cited in Figure 2. This regression indicates a net accident reduction such that the effect on accident rate, ΔR, of a change in degree of curve, ΔD_c, is

$$\Delta R = 0.056\ \Delta D_c \tag{1}$$

Although this study used a 3-year accident period and more effectively measured net accident difference than the studies cited in Figure 2, most of the same caveats listed for these studies would also apply here.

In 1982 the Illinois Department of Transportation conducted a cost-effectiveness analysis of highway curve rehabilitation projects. Two-year, before-after accident comparisons were made on eight highway curves ranging from 4.0 to 10.7 degrees that were reconstructed to include curves ranging from 2.5 to 5.0 degrees *(13)*. Although the study found a 61 percent mean reduction in accident rates, it is not clear over what lengths these rates were calculated. Also, the reported accident effects may very well be influenced by features other than curvature that were improved, including lane width, shoulder width, superelevation, skid surface, and the like.

Computing Accident Reductions from Available Relationships

The accident effects of highway curves is one of the most misunderstood areas of highway safety. Perhaps the most confounding aspect is the interaction between degree and length of curve. To truly evaluate the safety effectiveness of curve flattening, the net safety effect must be calculated over the total length of highway changed rather than the lengths of curves themselves.

To illustrate the misunderstanding that can be generated by this confounding effect, consider a fairly liberal interpretation of the results in Figure 2, which indicate that accident rate, R, is related to degree of curve, D_c, such that $R = 0.4 + D_c$. Using this relationship unconditionally, when a 10-degree curve is flattened to 5 degrees, the curve accident rate is reduced from 10.4 to 5.4 accidents per million vehicle miles. But this comparison is not reasonable because, for a given central angle, when the degree of curve is reduced by one-half, the length of curve is doubled. Therefore, the net safety effect along the highway must be analyzed over the total length of highway affected by the change. If, in the example, the original length of curve was 500 ft, the final length of curve would be 1,000 ft. Therefore, if the preceding accident rate relationship is true, 1,000 ft of 5-degree curve with an accident rate of 5.4 must be compared with the combination of 500 ft of 10-degree curve with a rate of 10.4, and 500 ft of tangent section must be compared with a rate of 0.4. The combined rate for this "before" condition is 5.4 accidents per million vehicle miles and, for this example, flattening the curve would be expected to produce a zero net accident reduction.

Under the preceding example, the same net result is evident for any combination of before-and-after curvature. Although this result would appear to make flattening curves a totally futile proposition, remember that the studies cited predicted the relationship between accident rate and degree of curve while ignoring the confounding effect of length of curve. To truly evaluate the net accident effects of curve flattening requires knowledge of the accident rates for tangent and curve sections by both length and degree of curve.

That the results of the previously cited studies are influenced by the distribution of curve lengths in each data base is illustrated by the data in Table 2, which show the distribution of curve lengths by degree of curve in the data base collected by Glennon et al. *(1)*. These data, which represent every available curve segment in the sampled areas that met study constraints, show a very strong inverse relationship between degree of curve and length of curve. More particularly, sharp curves tend to be short and flat curves tend to be long.

TABLE 2 Distribution of Curve Analysis Segments by Degree and Length of Curve *(1)*

Length of Curve (mi)	Number of Segments by Degree of Curve (deg)						Average Curvature (deg)[a]
	<1.00	1.00–2.99	3.00–4.99	5.00–7.99	≥8.00	Total	
<0.100	104	272	124	218	385	1,103	5.8
0.100–0.199	236	571	198	108	40	1,153	2.7
0.200–0.299	113	383	99	18	6	619	2.3
≥0.300	79	313	31	5	1	429	1.9
Total	532	1,539	452	349	432	3,304	3.6
Average length[b]	0.20	0.20	0.15	0.10	0.05	0.15	

[a]Rounded to nearest 0.1 degree of curvature.
[b]Rounded to nearest 0.05 mi.

The studies analyzed may provide a practical range for the net accident reduction that might be expected from curve flattening. If the five studies cited in Figure 2 indicate that zero may be a lower bound, then the Glennon et al. study might indicate an upper bound. Using the standard accident rate formula, the previously cited relationship for net accident rate reduction can be transformed to the more practical relationship as follows *(1)*:

$$\Delta A = \frac{(\Delta D_c)\ (\mathrm{ADT})}{81{,}540} \qquad (2)$$

where

ΔA = the net number of accidents reduced per year,
ΔD_c = the change in degree of curve, and
ADT = average daily traffic.

This net accident effect can be illustrated by the following example:

	Existing Condition	*Improvement*
Degree of curve	10 degrees	5 degrees
Analysis period	10 years (before)	10 years (after)
Average ADT	5,000	5,000 (projected)
Number of accidents	30	26.9 (calculated)

The analysis of this section shows a definite trade-off between degree and length of curve. Although most designers would agree that flatter curvature is more desirable, the effect of trading more curved roadway for tangent roadway can negate some of the advantage of the flatter curve.

Roadside Features as a Predominant Accident Factor on Highway Curves

As part of a multifaceted investigation of the safety of highway curves, Glennon et al. conducted an additional analysis of the 3,304, 0.6-mi curve segments *(1)*. In an attempt to maximize the potential for discovering accident relationships, two groups of sites were selected on the basis of either a very high or a very low accident rate. Differences in the geometric characteristics of these high- and low-accident populations were then investigated.

The sites were partitioned into three ADT classes to control for any effects of traffic volume. Sites that had accident rates at least twice the mean rate for that state's ADT class were designated as high-accident sites. For all but the highest ADT class, low-accident sites experienced no accidents over a 3-year period. A total of 330 sites that had extreme accident histories was thus selected.

Field measurements were taken at all 330 sites to further define their geometric and environmental features. The formal analysis of the high- and low-accident sites used a statistical technique known as discriminant analysis, which is used to statistically distinguish between two or more populations. The discriminating variables were the geometric and environmental features measured in the field.

Discriminant analysis distinguishes between the populations being studied by forming a linear combination of the discriminating variables whose value is D. The best-derived discriminant function is shown in Figure 3. Roadside rating is developed from Table 3, and the pavement rating is a measure of pavement skid resistance, SN60.

The relative discriminating power of the variables in the discriminant equation is shown in the following table. For example, the roadside rating, *RR*, contributes twice as much as the pavement rating, *PR*, to the ability to distinguish between high and low accident sites.

Variable	*Relative Discriminating Power*
Roadside rating, *RR*	2.1
Shoulder width, *SW*	1.4
Length of curve, *LC*	1.4
Degree of curve, *DC*	1.1
Pavement rating, *PR*	1.0

The discriminant analysis procedure predicts or classifies a site as being a high- or low-accident site based on the distribution of *D* values for the two groups. The procedure decides on whether each *D* score belongs to the high or low distribution by calculating if its probability is more or less than 50 percent. Using this criterion, the discriminant analysis procedure correctly classified 76 percent of the high-accident sites and 60 percent of the low-accident sites.

The value of discriminant analysis is primarily in its ability to predict high-accident locations. Because the *D* score distributions of the high- and low-accident sites overlap considerably, it is probably more efficient to concentrate on sites that have relatively high probabilities of being high-accident sites.

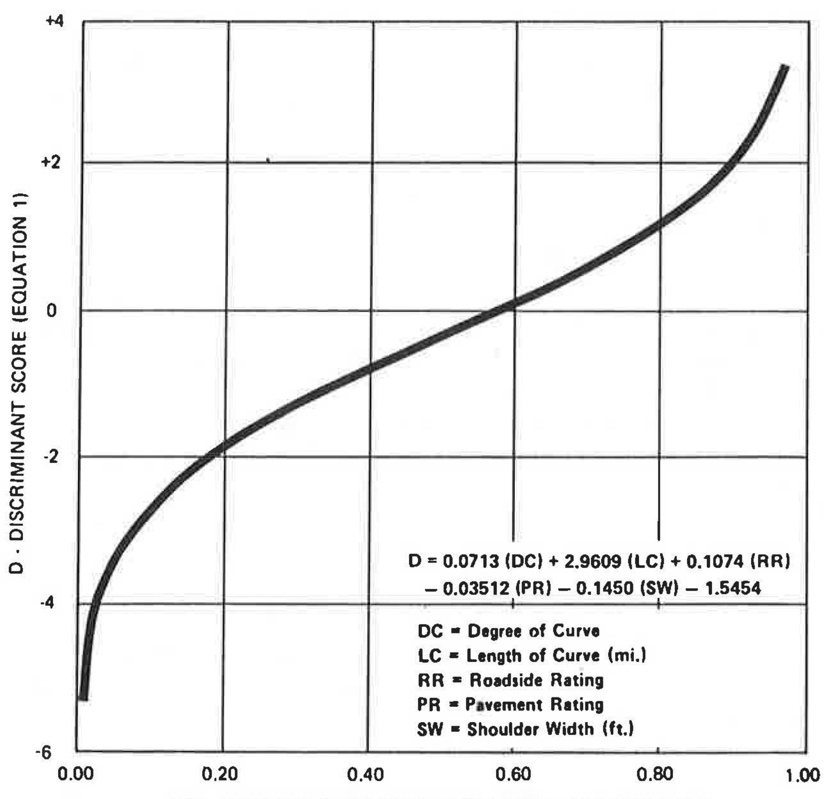

FIGURE 3 Relationship between discriminant score and the probability that a site is a high-accident site (1).

TABLE 3 Roadside Hazard Rating *(1)*

Side Slope	Coverage Factor[a]	Lateral Clear Width (ft) 30	25	20	15	10	5	0
6:1 or flatter	90	24	28	32	34	42	46	47
	60	24	27	29	30	35	38	39
	40	24	27	27	27	32	34	34
	10	24	24	24	24	25	26	26
4:1	90	35	37	39	41	44	48	49
	60	35	36	38	39	40	43	44
	40	35	36	37	37	39	41	41
	10	35	35	35	35	36	37	37
3:1	90	41	42	42	43	44	48	49
	60	41	42	42	42	43	45	46
	40	41	42	42	41	41	44	45
	10	41	42	42	41	41	42	42
2:1 or steeper	90	53	53	53	53	45	49	50
	60	53	53	53	53	46	49	50
	40	53	53	53	53	48	50	50
	10	53	53	53	53	50	50	50

NOTE: The roadside hazard rating represents the probability of an injury or fatal accident (%), given a roadside encroachment as defined by Glennon *(14)*.

[a]The coverage factor represents the probability of impact with a fixed object (%), given a certain lateral displacement as defined by Glennon *(14)*.

The procedure enables analysis of any probability criterion level. Figure 3 shows the relation between *D* score and *P(H)*, the probability that a site is a high-accident site. Selection of any *P(H)* criterion level can be translated into a minimum *D* score for analysis purposes.

A *P(H)* criterion of 80 percent was chosen for further study. The criterion classified 46 of the 330 study sites as high-accident sites with 42 of the 46 being correctly classified. As observed from the data in Table 4, with this criterion it appears that almost all sites that have high roadside hazards would qualify as high-accident sites. Likewise, almost all sites that have low roadside hazards would not qualify. The results are more mixed with moderate roadside hazards. Generally, moderate roadside hazards must be combined with either very sharp curvature or a combination of two variables that are moderate or worse.

For application at existing curves, the discriminant analysis indicates that improving roadside design, pavement skid resistance, and shoulder width may be candidate countermeasures. The reduction of curvature may not be practical or productive because of high costs and the apparent trade-off between degree and length of curve for a given central angle. This study also suggests that other design deficiencies, such as extremely unsatisfactory approach sight distances, narrow lanes, transitions, and extreme shoulder slope breaks, might be considered in an improvement program.

Glennon et al. also conducted a cost-effectiveness analysis of highway curve improvements by developing a rational relationship between accident rate for a 0.6-mi highway segment (with a curve) and the probability that the segment is a high-accident location *(1)*. Figure 4 shows this relationship, which was based on relating accident rates to discriminant scores and on an intuitive link between their large data base of 3,304 curve segments and their smaller data base of 330 high- and low-accident curve segments.

The effectiveness of highway curve improvements can be evaluated by combining the relationships shown in Figures 3 and 4 and the discriminant equation shown in Figure 4 as follows:

1. Compute the D score for the existing highway curve and determine from Figure 3 its probability of being a high-accident location.

2. Compute the D score for the proposed improved highway curve and determine its probability of being a high-accident location.

3. Compute the accident rate reduction over a 0.6-mi highway segment for the improvement using Figure 4.

4. Compute the net accident reduction for the improvement using Equation 3.

$$\Delta A = \frac{(\Delta R)\ (\mathrm{ADT})}{4{,}566} \tag{3}$$

where

ΔA = net accident reduction per year for the improvement;
ΔR = change in accident rate per 0.6-mi segment, accidents per million vehicle miles; and
ADT = average daily traffic.

Cross-slope Breaks on Highway Curves

The cross-slope break is the difference between pavement and shoulder slopes. For the outside of highway curves, AASHTO policy limits the cross-slope break to 8 percent, which in turn puts constraints on either the maximum superelevation rate or the amount of shoulder slope *(15)*. Under this criterion, if the selected superelevation rate is 6 percent, the maximum outside shoulder slope is –2 percent. If, however, the selected

TABLE 4 Percent Probability that a Curve Segment is a High-Accident Location *(1)*

Curve Length (mi)	Shoulder Width (ft)	Degree of Curve 1	3	6	12	20
Low Roadside Hazard Rating (*RR* = 20)/Low Pavement Rating (*PR* = 20)						
Long (0.30)	0	75	77	80	86	91
	8	50	53	60	70	78
Moderate (0.17)	0	68	71	75	84	89
	8	42	45	52	61	71
Short (0.05)	0	61	64	68	77	85
	8	35	38	44	53	65
Moderate Roadside Hazard Rating (*RR* = 35)/Moderate Pavement Rating (*PR* = 35)						
Long (0.30)	0	91	92	93	95	97
	8	73	79	82	87	92
Moderate (0.17)	0	87	89	90	93	96
	8	66	72	75	81	87
Short (0.05)	0	82	84	86	90	94
	8	59	65	68	74	82
High Roadside Hazard Rating (*RR* = 50)/High Pavement Rating (*PR* = 50)						
Long (0.30)	0	94	95	95	97	98
	8	87	90	90	93	96
Moderate (0.17)	0	93	94	94	95	98
	8	84	87	87	90	95
Short (0.05)	0	91	93	93	94	97
	8	79	83	83	86	93

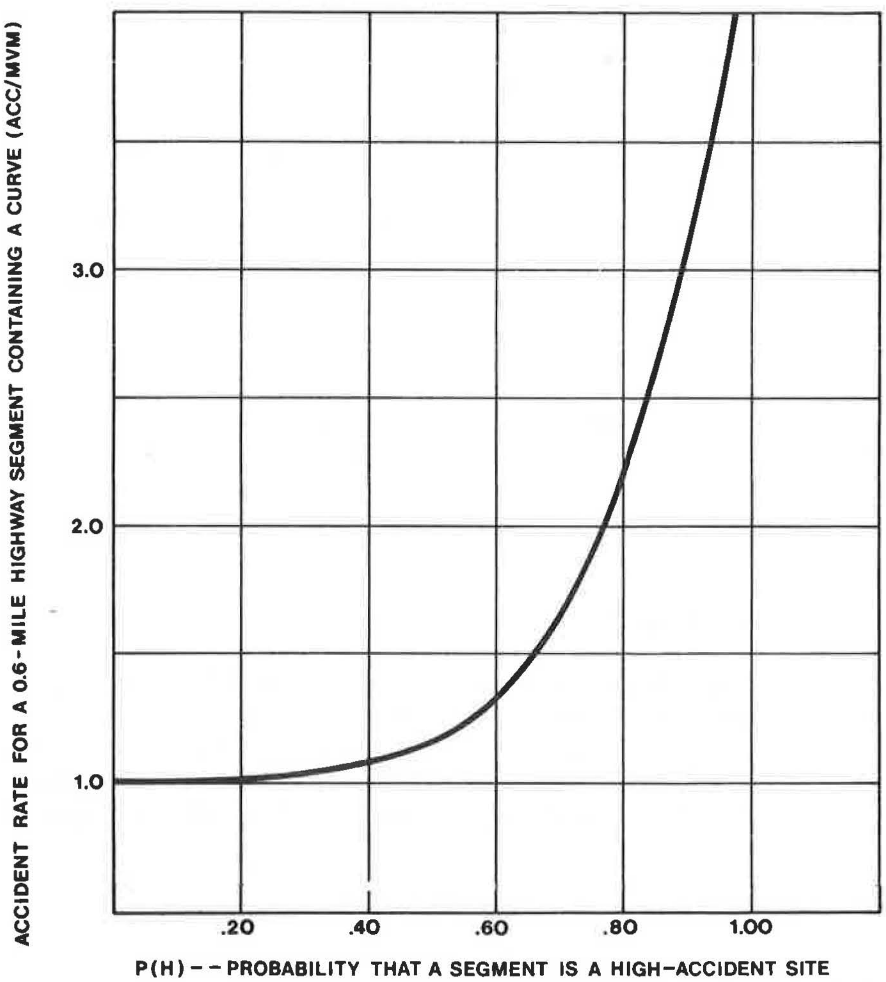

FIGURE 4 Relationship between accident rate and P(H) *for cost-effectiveness analysis.*

superelevation is 9 percent, AASHTO policy dictates that the outside shoulder slope must be tilted up, making the outside shoulder drain across the pavement.

Glennon et al. conducted research for the Federal Highway Administration aimed at verifying the adequacy of the AASHTO criterion for the surface-shoulder cross-slope break *(16)*. The study used the HVOSM computer simulation to test a moderate, four-wheel traversal onto the outside shoulder by a mid-sized passenger car traveling the controlling design speed of the highway curve. Recovery from the shoulder traversal was achieved by using the critical path measured by earlier field studies *(17)* on highway curves and a maximum driver discomfort factor of 0.3*g*'s.

For drivers who encroach onto the outside shoulder with a four-wheel traversal, the shoulder slope, rather than the cross-slope break, was found to be the critical design element. However, because of the relationship between radius and superelevation for controlling design curves, the relationship of critical vehicular dynamics to shoulder slope translates to a design criterion for cross-slope break of 8 percent for stabilized shoulders (6 ft or greater) that will accommodate a four-wheel recovery. This confirms the AASHTO criterion for full-width shoulders where a four-wheel traversal is possible.

For stabilized shoulders (5 ft or less) that will only accommodate a two-wheel traversal, the allowable cross-slope break increases as the shoulder becomes narrower as follows:

Shoulder Width (ft)	*Allowable Cross-Slope Break (%)*
5	9
4	12
3	15
2 or less	18

These greater cross-slope breaks recognize the lesser "effective cross slope" experienced by a two-wheel vehicular recovery on a narrower shoulder that was not explicitly designed to accommodate a full four-wheel recovery. These greater allowable cross-slope breaks are particularly important in RRR highway improvements where either

1. The desire is to increase the superelevation on a roadway with a narrow shoulder and an existing 8 percent cross-slope break; or
2. The plan is to widen the traveled way at the expense of shoulder width, leaving shoulders that are 5 ft or less in width.

In both cases, the results of the referenced study indicate that greater cross-slopes do not compromise safety beyond the prior decision to allow the narrow shoulder.

Other Factors Related to Highway Curve Safety

Over the years several authors have extolled the benefits of spiral transitions to highway curves. More recently, the combination of HVOSM computer simulation and field studies by Glennon et al. have strongly supported these arguments *(1)*. The field studies of path behavior on unspiraled curves indicate that drivers, in attempting to spiral their path from an infinite radius to the radius of the highway curve, always overshoot the curve radius thereby creating higher friction demands. HVOSM comparisons made on curves that were otherwise identical except for the presence of a spiral indicate that aggressive or inattentive drivers will experience a dramatic reduction in the maximum friction demand if a spiral transition is introduced.

The safety effects of curve warning signs and delineators have also been studied *(18–20)*. In 1980, Lyles examined the effectiveness of alternative advance warning sign configurations in reducing speeds on curves. He found that in spite of relatively large speed decreases near the beginning of the curve, no sign configuration was found consistently more effective than another in reducing speeds *(18)*.

Wright et al. studied the effects of reflectorized markers on nighttime accidents for curves of 6 degrees or more in Georgia *(19)*. Although the authors reported an effective reduction in accidents based on the assumption that the reflectors would have no effect on daytime accidents, their actual accident numbers showed a net increase in accidents after the placement of the reflectors. Taylor and Foody reported the before-after differences for the placement of roadside delineators on highway curves *(20)*. The study revealed that degree of curve was not the only important parameter on highway curves. The central angle of the curve was found to be a more efficient parameter.

Specifically, curves with curvature between 5 and 10 degrees and central angles between 20 and 40 degrees showed significant accident reduction when delineated.

Pavement washboard and warp was highlighted as a safety problem on highway curves by Glennon et al. *(1)*. Based on some general analytics and results of previous full-scale vehicular studies, it was noted that very short, high-amplitude bumps cause both vertical and lateral wheel hop. Successive loading and unloading of first front and then rear tires, with contingent wheel hop, greatly increases the effective lateral acceleration on the tires. In addition, loss of steering authority occurs, which forces the driver to input larger steering angles than expected.

One other aspect of safety on highway curves discussed by Glennon et al. relates to roadside slopes *(1)*. They conclude that for identical roadside slope rates, roadside traversals on curves are more severe than on tangents. Because, for any encroachment line, the effective slope is steeper on a curve than on a tangent, vehicle occupants will experience higher vertical accelerations and the vehicle will have a much greater tendency for rollover and a higher probability of producing severe injuries.

VERTICAL ALIGNMENT

The vertical alignment consisting of vertical curves and straight grades has been the subject of accident studies conducted worldwide. Some of these studies produced results that make general distinctions between grades and level sections, upgrades and downgrades, crests and sags, or flat and steep grades *(4, 7–9)*. Although all of these studies lack control of large variances associated with interdependent variables and length of grade, they indicate the following general conclusions:

1. Grade sections have higher accident rates than level sections,
2. Steep grades have higher accident rates than mild grades, and
3. Downgrades have higher accident rates than upgrades.

An often-quoted, pre-1960 German study by Bitzel is one of the few studies that indicates a direct relationship between grade and accident rate *(21)*. However, the relationship found in this study appears to be related to a set of unusual circumstances, which included widely fluctuating annual accident rates over long stretches of highways, a high percentage of accidents involving stationary vehicles, a very high percentage (70 percent) of accidents involving trucks, and a large percentage of trucks with high weight-to-horsepower ratios. These circumstances render the results of this study useless for predicting the accident effects of grade improvement projects in the United States.

The remainder of the studies reviewed used some form of either multivariate analysis or a sufficiency rating scheme to identify the incremental effects of geometric elements including vertical alignment *(10–12)*. None of these studies produced any reliable measures of the accident effects of vertical alignment.

APPLICATION OF RESULTS TO RRR PRACTICE

The incremental accident benefits of flattening grades has not been precisely determined in available studies but appears to be reasonably small within practical ranges of grade change. For highway curves, many past studies have shown substantially lower accident rates for flatter curves; but all of these studies have examined only the accident rate on the curve itself and have ignored the confounding effect of curve length. When these results are used to examine the next accident reduction associated with flattening a curve at a location where the central angle is held constant, the net accident benefits appear to be very small.

These results are consistent with the findings of a study that provided the following measure of net incremental difference in accidents associated with curves of various degrees:

$$\Delta A = \frac{(\Delta D_c)\ (\text{ADT})}{81{,}540}$$

where

ΔA = the net number of accidents reduced per year,
ΔD_c = the change in degree of curve, and
ADT = average daily traffic.

Although flatter curvature is desirable, there appears to be some trade-off (when central angle is held constant) between the benefits of flatter curvature and the disbenefits of more net roadway with curvature.

Another major conclusion is that, because of the high rate of single-vehicle accidents on highway curves, low-cost roadside safety improvements on highway curves may be one of the most effective RRR safety improvements. This is particularly true for improvement of low-height fill slopes and removal of trees to improve the clear-zone width on the outside of curves carrying more than 2,000 vehicles per day.

Another feature of highway curves that can become prominent in RRR projects is the break in cross slope between shoulders and superelevated pavements on curves. Designers of RRR projects face a dilemma because current AASHTO policy limits the break to 8 percent. On curves where increased superelevation is desirable or where shoulder widths will be sacrificed to improve narrow lanes, either the AASHTO criterion must be violated or extensive shoulder and roadside reconstruction must be planned.

Considering that the major function of the outside shoulder at such locations is to provide recovery from moderate roadway departures, recent research confirms the AASHTO policy for shoulders 6 ft or more in width where a four-wheel traversal is possible. For narrower width shoulders that are implicitly designed for two-wheel traversals, larger cross-slope breaks are possible as shown below:

Allowable Shoulder Width (ft)	*Cross-slope Break (%)*
5	9
4	12
3	15
2 or less	18

These greater breaks recognize the less severe "effective" cross slope experienced during a two-wheel shoulder traversal where the inside wheels are still on the superelevated pavement. These greater breaks also do not compromise safety beyond the prior decision to allow the narrow shoulder.

In further consideration of the safety enhancement of RRR projects, other minor treatments on highway curves offer the potential for accident benefits at relatively low costs as follows:

1. Although the literature does not provide a measure of the incremental accident effects of superelevation, consideration should be given to increasing superelevation on highway curves in conjunction with highway resurfacing projects. This incrementally low-cost improvement might be particularly effective either where pavement drainage is inadequate or where the design speed of the curve is below the highway operating speed.
2. On resurfacing projects, attention should be given to eliminating existing pavement irregularities such as washboard, pot holes, bumps, and dips on highway curves. These irregularities have been shown by past research to create severe control problems for drivers on highway curves.
3. On resurfacing projects, quality control should be exercised on highway curves to avoid both reducing the existing superelevation and introducing pavement irregularities.

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Effect of Sight Distance on Highway Safety

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For safety of highway operations, the designer must provide sight distances of sufficient length along the highway that most drivers can control their vehicles to avoid collision with other vehicles and objects that conflict with their path. Since 1940, the American Association of State Highway and Transportation Officials (AASHTO) has defined acceptable limits for stopping, passing, and intersection (corner) sight distances based on a rational analysis of safety requirements *(1–5)*. Adequate sight distances have been defined as a function of operating speeds and are achieved by designing nonrestrictive horizontal and vertical alignment and by avoiding sight obstructions (vegetation, embankments, walls, etc.) in intersection quadrants and on the inside of horizontal curves.

When considering the safety enhancement of resurfacing, restoration, and rehabilitation (RRR) projects, designers have a different perspective than when they are designing a new highway. Changes in existing alignment are very expensive and should not be undertaken unless their cost-effectiveness compares favorably with competing demands for RRR funding. For this reason, it is important to know the expected sight distance safety benefits for any proposed changes to existing alignment. Also important are the safety benefits of alternative low-cost improvements to sight distance such as the removal of roadside obstructions.

This critical review of literature was undertaken to synthesize the available knowledge on the relationships between highway sight distance and safety in order to provide guidance on selecting cost-effective improvements that will enhance the safety of RRR projects. This review is limited to two areas of sight distance design: stopping sight distance and intersection sight distance. The safety effects of improvements to passing sight distance were not studied because, although there are safety aspects to available sight distances within passing zones, the provision of more or longer passing zones is normally considered an operational rather than a safety improvement.

STOPPING SIGHT DISTANCE

Analysis of operational and safety aspects of stopping sight distance (SSD) requires an understanding of the concept of SSD as it relates to highway operations. The geometric design policy published by AASHTO discusses the need for SSD:

> If safety is to be built into highways the designer must provide sight distance of sufficient length in which drivers can control the speed of their vehicles so as to avoid striking an unexpected obstacle on the traveled way . . . .
> The minimum sight distance available on a highway should be sufficiently long to enable a vehicle traveling at or near the likely top speed to stop before reaching an object in its path. While greater length is desirable, sight distance at every point along the highway should be at least that required for a below average operator or vehicle to stop *(2, 3)*.

This short discussion alludes to many of the operational elements of stopping sight distance—namely, vehicle performance, driver ability, and the roadway alignment. This AASHTO operational "model" thus provides a reasonable starting point for considering the relationship between SSD and highway operations.

SSD as defined by AASHTO is the sum of two distances: (a) the distance a vehicle travels between the time a driver sights an object and the time he applies the brakes; and (b) the distance a vehicle travels in braking to a stop. SSD is determined by the following equation:

$$SSD = 1.47\ PV + \frac{V^2}{30\ (f \pm g)}$$

where

V = initial speed, mph;
P = perception-reaction time, sec;
f = coefficient of friction; and
g = percent of grade divided by 100.

AASHTO defines minimum SSD requirements in terms of a passenger car approaching a stationary object in its path. This basic functional model has remained unchanged since 1940. The following review of the evolution of AASHTO stopping sight distance policy illustrates the reasoning behind this model. It also demonstrates the need to go beyond this simple "abstraction" to gain insight into the safety relationships of SSD.

In 1940, the American Association of State Highway Officials (AASHO) *(1)* formally recognized the need for a sight distance requirement to help drivers avoid collision circumstances other than passing encounters. Although AASHO recognized that a clear sight line to the pavement was desirable, analyses of how this requirement affected construction cost led to a compromise. A design object height of 4 in. was selected on the basis of optimizing the trade-off between object height and required vertical curve length. Although the object height criterion is discussed in the AASHO policy as it related to objects in the road, the selection of a 4-in. height clearly was not based on the frequency or severity of such objects. This conclusion is further borne out by subsequent changes in AASHO policy to a 6-in. object height; the same discussion was used in relating this height to roadway events.

Selection of other design parameters such as perception/reaction time, eye height, and pavement friction was rational; individual design values were selected based on the currently known distributions of these physical values, which were periodically

TABLE 1 Evolution of AASHTO Stopping Sight Distance Policy

	Design Parameters					
Year	Eye Height (ft)	Object Height (in.)	Perception/ Reaction Time (sec)	Assumed Tire/ Pavement Coefficient of Friction	Assumed Speed for Design	Effective Change from Previous Policy
1940 *(1)*	4.5	4	Variable: 3.0 sec at 30 mph to 2.0 sec at 70 mph	Dry: *f* ranges from 0.50 at 30 to 0.40 at 70 mph	Design speed	–
1954 *(2)*	4.5	4	2.5	Wet: *f* ranges from 0.36 at 30 to 0.29 at 70 mph	Lower than design speed (28 mph at 30 mph design speed; 59 mph at 70 mph design speed)	No net change in design distances
1965 *(3)*	3.75	6	2.5	Wet: *f* ranges from 0.36 at 30 to 0.27 at 80 mph	Lower than design speed (28 mph at 30 mph design speed; 64 mph at 80 mph design speed)	No net change in design distances
1970 *(4)*	3.75	6	2.5	Wet: *f* ranges from 0.35 at 30 to 0.27 at 80 mph	Minimum values same as 1965; desirable values design speed	Desirable values are up to 250 ft greater than minimum values
1984 *(5)*	3.50	6	2.5	Wet: *f* slightly lower than 1970 values for higher speeds	Minimum values same as 1965; desirable values design speed	Computed values always rounded up giving slightly higher values than 1970

updated as indicated in Table 1. Yet, the underlying methodology was by design an abstraction—a simplified set of elemental factors used to derive a distance—with only an indirect link to the functional needs for sight distance.

The Role of Stopping Sight Distance in Highway Accidents

The literature on the relationship between highway accidents and SSD is highly limited. Several accident studies *(6–12)* were found in which SSD was considered one of several roadway elements that might affect accident rates. All of these studies used some form of either multivariate analysis or a sufficiency rating scheme to identify the incremental effects of SSD. None of these studies is able to offer any reliable method of determining the accident effects of variable SSD.

A study by Olson et al. *(13)* does provide some general insight into the accident effects of SSD. A small but well-designed accident study was conducted on 10 pairs of sites—one site was a crest with limited SSD (118 to 308 ft) and the other was a nearby crest with identical conditions except that it had adequate SSD (greater than 700 ft). Of these comparison pairs, the limited SSD site had more accidents than the adequate SSD site in seven of the pairs. In one of the pairs, the adequate SSD site had more accidents, and in two of the pairs the sites had an equal number of accidents. As a group, the comparison pairs exhibited a 50 percent higher accident rate for the limited SSD sites compared with the adequate SSD sites. Although this study indicates some accident reduction benefits from improved SSD, the comparison of 20 to 40 mph AASHTO designs with those 75 mph or greater may be of little use in the RRR process in helping to decide whether to upgrade restrictive sight distance on highways operating at 55 mph.

With the lack of accident studies documenting the incremental effects of changes in SSD, one method of estimating these effects might be to use probability simulation such as that employed by Farber *(14)*. Farber's model, however, needs further refinements in order to produce realistic estimates.

Functional Analysis of Stopping Sight Distance Requirements

Neuman et al. *(15)* present the results of a recent study that critically reviewed present design practice for SSD. They developed a concept of SSD that focuses on various highway operational requirements. From this operational concept of SSD, shortcomings and inconsistencies in the AASHTO design policy were revealed. A summary of the considerations in that study is discussed next.

Analysis of the functional requirements for SSD gives focus to the types of accidents and hazardous situations that result from limited SSD. The following points are useful in understanding the link between SSD and safety.

- *SSD accidents are event oriented.* The mere presence of a segment of highway with inadequate SSD does not guarantee that accidents will occur. SSD-related accidents occur only after an event or events create a critical situation. These events can take the form of arrivals of conflicting vehicles, the presence of objects on the road, poor visibility, or poor road surface conditions, or all of these events. Some of these events are a function of the highway type (e.g., crossing conflicts at intersections do not occur on freeways); some are related to other geometric or environmental elements (e.g., requirement for severe cornering maneuver on wet pavement); and others may be totally random (e.g., presence of an object in the road).
- *The probabilities of critical events occurring within the influence of SSD restrictions define the relative hazard of these restrictions.* The relative hazard of various SSD-deficient locations can be estimated by examining the probabilities of critical events. Traffic volume, frequency of conflicts (rear-end, head-on, crossing, object in road), and time exposure of each vehicle to the restricted SSD are all useful in estimating these probabilities.
- *Severity as well as frequency is important.* SSD situations that create severe although infrequent conflicts (e.g., head-on or angle collisions) may be just as important as situations with frequent, less severe conflicts. Cost-effectiveness analysis rightfully values injuries and fatalities prevented much higher than property-damage-only accidents.
- *Many uncontrollable or unquantifiable factors also contribute to accident causation.* Driver performance characteristics such as perception/reaction time, vehicle characteristics such as braking ability, and certain imponderables such as the driver's state of mind, all contribute to increased accident potential. Although these factors are exclusive of the presence of a poor SSD location, their importance is undoubtedly heightened when the deficiency in SSD means the driver has less time to react to an event. This reduced time may make the difference between collision avoidance and an accident.

Figure 1 shows the complexity of SSD requirements when viewed as a function of all the elements discussed previously. Present AASHTO policy, which defines SSD requirements based on only one event and one set of conditions, produces sufficient SSD for certain events or conditions but not for others.

Application of these functional relationships for SSD revealed a number of situations for which greater SSD than the minimum AASHTO values might be advisable. These included not only approaches to intersections and sharp highway curves but also highway curves. Truck operations on highway curves with sight restrictions created by vertical obstructions (such as trees and walls) were found to be the situation where the AASHTO model least fit the SSD needs for the following two reasons:

1. When a vehicle brakes on a curve, the frictional demand is greater than for the same braking level and speed on a tangent because the total deceleration is the resultant of the braking deceleration and the lateral cornering acceleration. Because of this compounding of frictional demand, AASHTO-level braking on curves could often lead to loss of control. Therefore, the need for hard braking should be reduced by the provision of longer sight distances.
2. Vertical obstructions on the inside of highway curves create special problems for large trucks. In these situations, the greater eye height of the truck driver is of no value in compensating for the longer truck stopping distances. Therefore, trucks need greater SSD for stopping on curves because of both longer stopping distances and the need to keep resultant friction within a tolerable range.

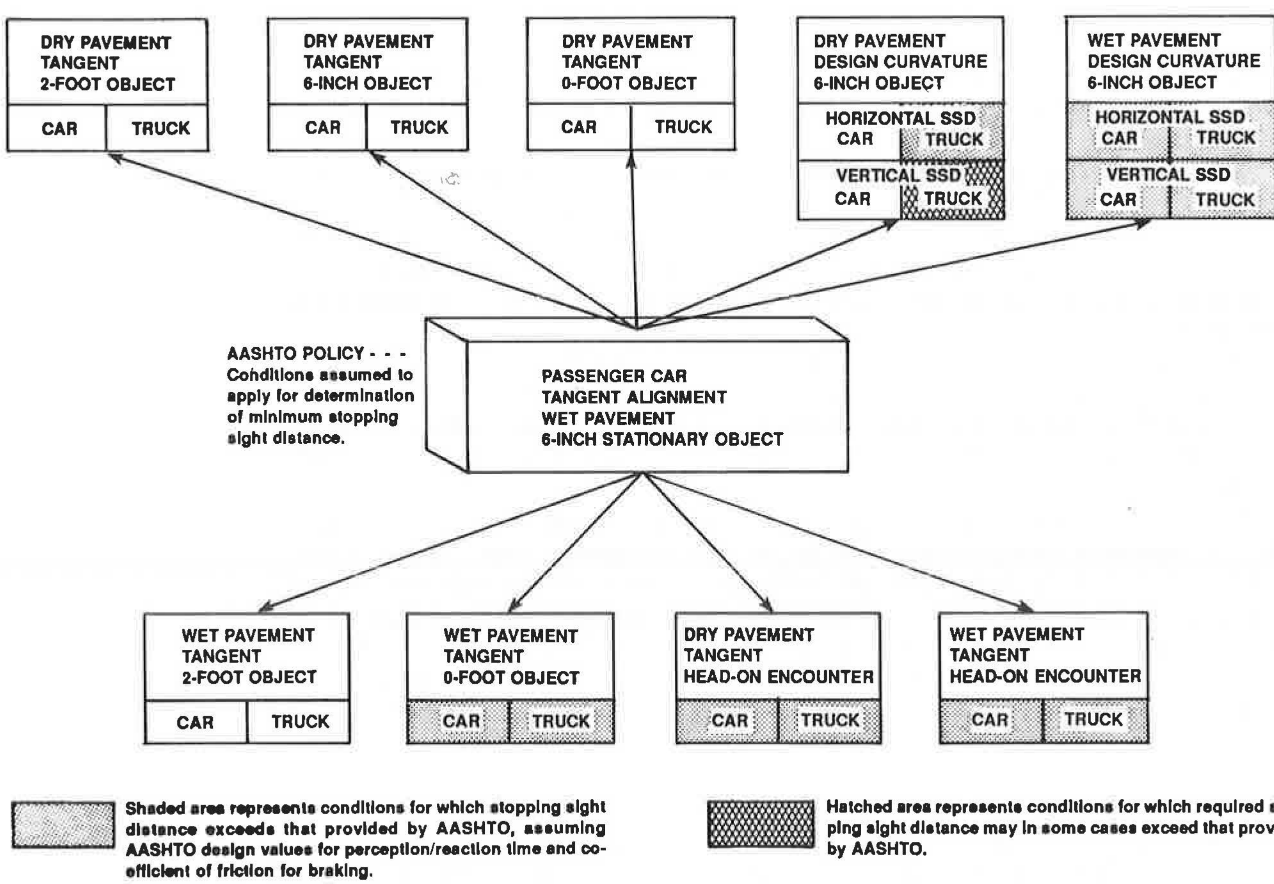

FIGURE 1 Analysis of functional requirements for stopping sight distance (15).

Safety Trade-Off in Lengthening Vertical Curves

Another aspect of SSD that was discovered in the Neuman et al. *(15)* study relates to the fact that longer vertical curves are not always necessarily better. This may be particularly true when an extremely deficient crest is upgraded to provide a design speed that is still below the highway operating speed. This phenomenon is best described by an example using SSD profiles.

Figure 2 shows three different sight distance profiles for different vertical curves joining a severe alignment of two 7 percent grades. Considering Profile 1 as an existing crest with a design speed of 25 mph and a minimum 1984 AASHTO policy *(5)* SSD of 150 ft, the question is, "What are the safety benefits gained by lengthening the vertical curve on an existing highway with a 55-mph operating speed?" If the vertical curve is lengthened to provide a 40-mph minimum SSD of 275 ft (Profile 2), about 400 ft of the highway will be improved. However, because a driver approaching the shorter crest from a distance can see farther up the crest and also more quickly reaches the point where the sight distance opens up, the "improved" geometry has 600 ft of highway where the SSD is worse than before. In comparing each of these vertical curve profiles with Profile 3 for a crest providing a design speed of 55 mph, the shorter crest has a length of about 600 ft with deficient SSD and the longer crest has a length of about 1,000 ft with deficient SSD.

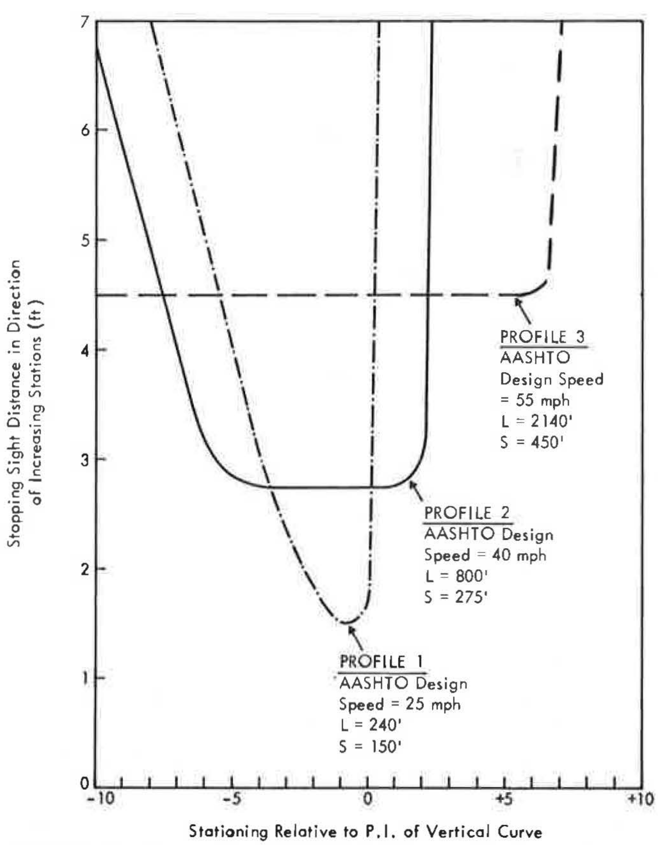

FIGURE 2 *Comparison of sight distance profiles for various design speeds for a crest joining 7 percent grades (A = 14%).*

This example indicates the possible futility in lengthening some existing vertical curves. Not only would the construction expense be high for cutting about 9 ft into the hill to change Profile 1 to Profile 2, but the safety benefits may be small or even negative. Changing from Profile 1 to Profile 3 might be expected to produce positive safety benefits; however, this improvement would require a 33-ft greater cut.

Cost-Effectiveness of Stopping Sight Distance Improvements

Neuman and Glennon performed an analytical study *(16)* to evaluate the cost-effectiveness of SSD improvements at locations where this feature does not meet AASHTO requirements for prevailing operating speeds. Because of a lack of available data on the accident reduction effectiveness of SSD improvements, optimistic assumptions were used to estimate the accident benefits. This way, if certain improvements indicated a benefit-cost ratio less than one, they clearly would be unjustified.

These accident reduction assumptions had the effect of firmly establishing upper limits on the improvement effectiveness. A matrix of accident rate factors ranging from 0 to 4 was developed to describe the hypothesized relationship between accident rate and two basic descriptors of limited sight distance conditions:

1. The severity of the restriction (design speed deficiency), and
2. The presence of other potentially hazardous geometric features (sharp curve, intersection, narrow bridge, etc.) within the sight-restricted area.

These hypothesized accident rate factors are given in Table 2.

TABLE 2 Hypothesized Accident Rate Factors for Evaluating SSD Restrictions

Character of Geometric Condition Within SSD Restriction	Severity of SSD Restriction (Amount Design Speed is Less than Prevailing Speed (mph)			
	0	10	15	20
Minor hazard	0.0	0.5	1.2	2.0
Significant hazard	0.4	1.1	2.0	3.0
Major hazard	1.0	1.8	2.8	4.0

NOTE: Factor multiplied by average statewide accident rate for highway type yields the partial accident rate at the site associated with the combined effects of the roadway geometry and SSD restriction. For example, a very severe curve hidden by a 20-mph SSD deficiency would produce 8.0 accidents per million vehicle miles in a state where the average accident rate was 2.0 accidents per million vehicle miles. If this SSD restriction was removed, the computed accident rate reduction would be 8.0 − 1.0 (2.0) = 6.0 accidents per million vehicle miles, applied over the length of the original SSD restriction.

Several SSD improvement types were identified, their costs calculated, and accident benefits determined using the accident rate factor matrix. Using these determinations, the average daily traffic (ADT) required to produce a benefit-cost ratio of one was calculated. This analysis indicated that the lengthening of vertical curves or the flattening of horizontal curves to eliminate SSD deficiencies may only be cost-effective on roadways with high ADT levels where other significant hazards are present within the sight restriction. However, clearing trees or minor obstructions from the inside of sight-restricted horizontal curves appears to be cost-effective for almost all highways.

Clearing Obstructions on the Inside of Highway Curves

The conclusion of the cost-effectiveness of clearing vegetation from the inside of horizontal curves recognizes that the required offsets to obstacles vary on the approaches and along the curve such that the maximum offset, *m* specified by AASHTO *(2, 3, 5)* and shown in Figure 3 is only required toward the center of longer curves and may not be required at all on shorter curves.

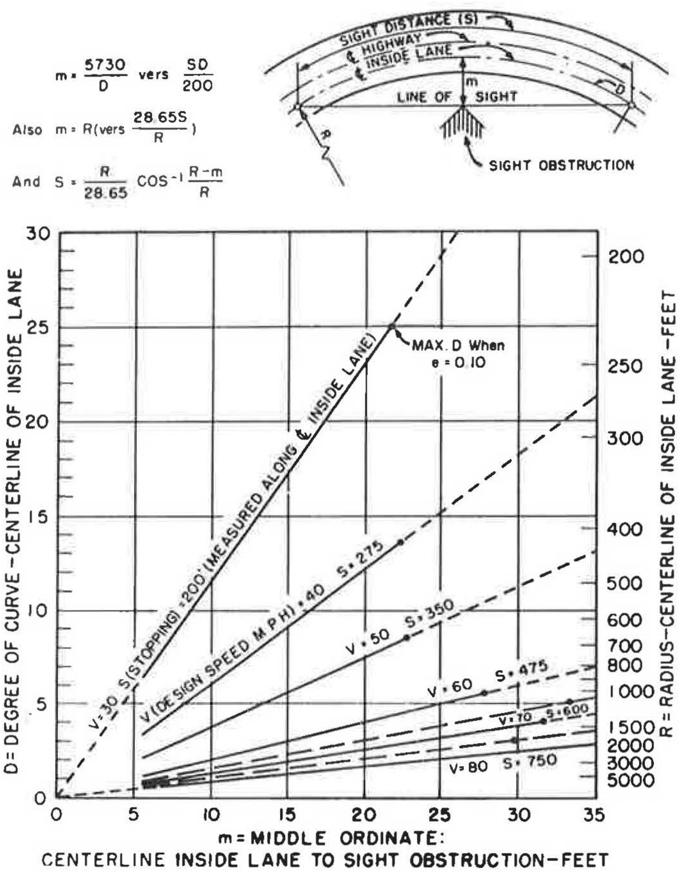

FIGURE 3 Relation between stopping sight distance and middle ordinate on horizontal curves (3).

Both the studies by Olson et al. *(13)* and Neuman and Glennon *(16)* show that the AASHTO specification for *m* is only required for highway curves where the length of the curve, *L*, is longer than the required SSD. As shown by the example sight line analysis in Figure 4, the offset, *m*, is needed from a point that is a distance of SSD/2 from the PC of the curve to a point that is a distance of SSD/2 from the PT of the curve. From these points outward, the required offsets decrease to zero at a distance of SSD. For this case, where *L* is greater than the required SSD, a graphical analysis indicates that the offset relationship is insensitive to both the degree of curve and the length of required sight distance such that Figure 5 is a reasonable approximation of the required offsets.

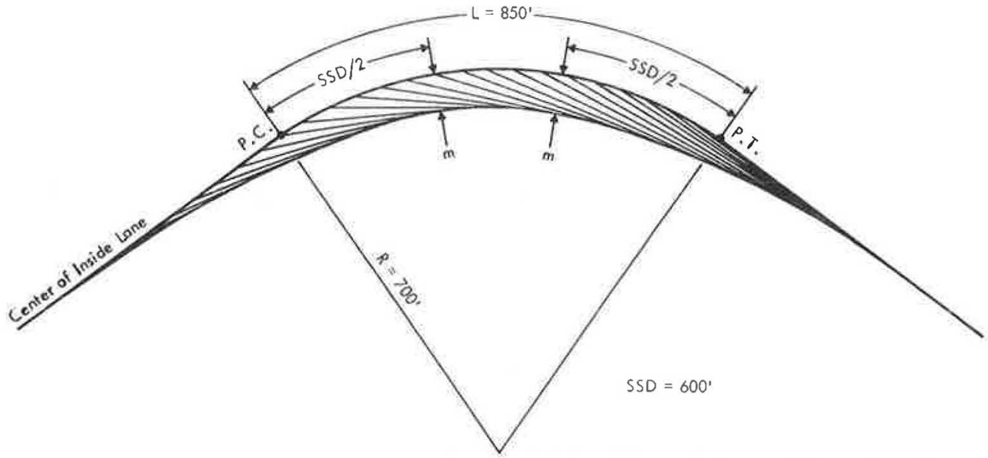

FIGURE 4 Example sight obstruction envelope on horizontal curves for condition where the stopping sight distance is less than the length of the curve.

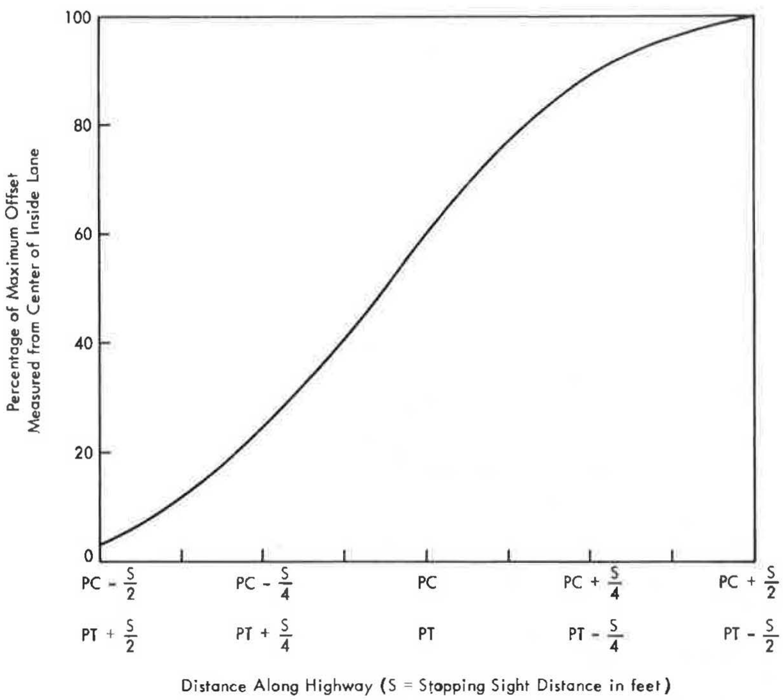

FIGURE 5 Relationship offsets at various points to maximum offset for sight obstruction envelope on a horizontal curve where the stopping sight distance is less than the length of the curve.

For short highway curves where *L* is shorter than the required SSD, Olson et al. have derived a reasonable approximation to the maximum offset as given by the following equation:

$$m_s^2 = \frac{L(2S - L)}{8R}$$

where

m_s = maximum offset, in feet, between center of lane and obstruction at the midpoint of the curves, where S is greater than L;
L = length of curve, ft;
S = stopping sight distance, ft; and
R = radius of curvature, ft.

This maximum offset is always less than the maximum offset, m, required when L is greater than the required SSD.

The relationship for other offsets on the short curve is not so clearcut. Graphical exercises indicate that the locus of offsets for the short curve is a function of degree of curve, length of curve, and the required SSD. Although a mathematical relationship could not be found, the required locus of offsets can always be solved graphically for any combination of parameters. Suffice to say the offset at the PC and PT will vary between 60 and 100 percent of m_s, as L goes from SSD to zero. Also, the required offset at a distance of SSD/2 outside of the PC or PT will always be a small fraction of m_s, such that obstacles outside the traveled way should not restrict the required SSD.

In analyzing the SSD profiles on horizontal curves, they are found to exhibit characteristics different from vertical curves. Because the sight obstruction is off the highway for horizontal curves instead of being the highway alignment itself as for vertical curves, clearing of the sight envelope will never reduce the amount of sight distance at any point. Figure 6 shows both the SSD profile for an existing obstacle offset envelope and the SSD profile for a slight clearing beyond that envelope. This example demonstrates the improvement in the SSD profile.

Effectiveness of Signing for Limited Sight Distance

In 1981 Christian et al. *(17)* conducted a study to evaluate the effectiveness of the standard Limited Sight Distance warning sign applied at sight-restricted highway crests. Spot speed studies were undertaken at 14 locations both with and without the warning sign and its accompanying advisory speed plate. The results of the speed data recorded at the crests of the vertical curves indicated that the warning signs with advisory speed plates had no effect in slowing vehicles. Driver surveys also indicated that these signs were not well understood.

INTERSECTION SIGHT DISTANCE

A driver approaching an intersection should have an unobstructed view of the intersection and a length of the intersecting highway sufficient to avoid colliding with approaching vehicles. AASHTO *(2, 3, 5)* provides recommended values for both uncontrolled and stop-controlled intersections. In both cases, available sight distance is measured from the driver's eye height to the roofline of the conflicting vehicle.

For uncontrolled intersections, the minimum safe sight distance along each highway is related to vehicle speeds and to the resultant distances traveled during driver perception and reaction and during braking. As defined by AASHTO *(2, 3, 5)*, the recommended legs of the sight triangle are equivalent to the length of the SSD

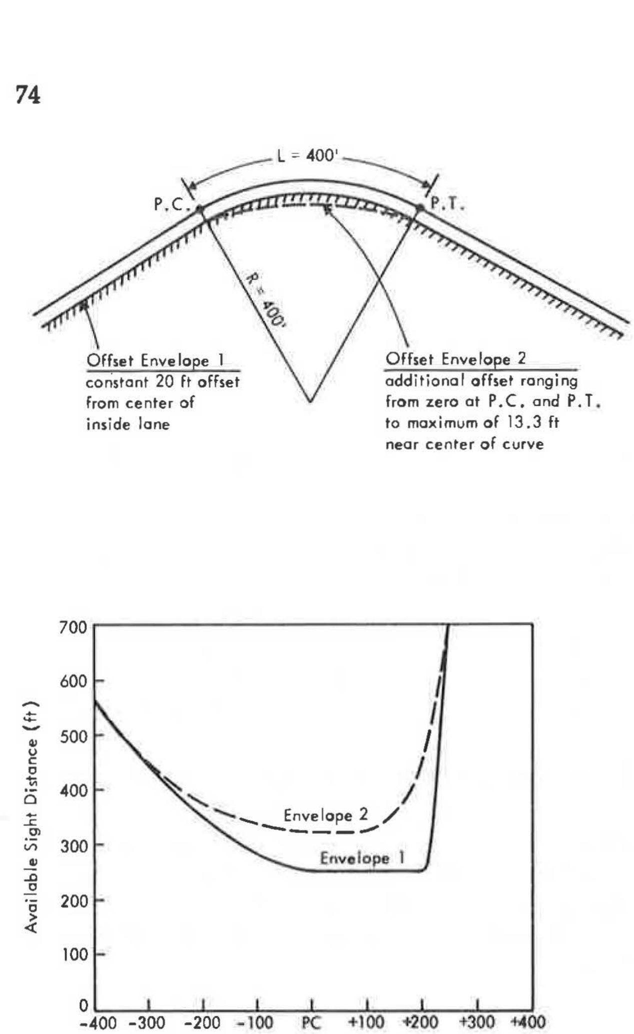

FIGURE 6 Analysis showing an example improvement to stopping sight distance on a horizontal curve.

requirements corresponding to the design speed of each leg. For intersections with stop-control on the minor highway, the required sight distance is a function of the time and distance necessary for the stopped driver to scan the approaches, accelerate, and clear the intersection.

Only a few studies *(18–22)* have addressed the role of intersection sight distance in producing accidents. Only two of these studies *(18, 22)* provide any specific relationships between accidents and intersection sight distance.

Wu studied the relationship between accident rate and what he called "clear vision right-of-way" at 192 signalized intersections *(22)*. Although no specific numbers are given and no statistical tests are cited, the study concludes that intersections where vision is poor have significantly higher injury, property damage, and total accident rates. The conclusions, however, may be misleading because right-of-way widths varied from 66 to 204 ft in both the poor and clear vision categories. Conceivably, then, an intersection with good sight distance could be in the poor category and vice versa.

David and Norman *(18)* studied the relationships between accident rate and various intersection geometric and traffic features. The sample included 558 intersections on which 4,372 accidents occurred over 3 years. Intersections had three and four legs, two or four through lanes, and stop or signal control. The study revealed significant accident rate differences between "obstructed" and "clear" intersections for various levels of restriction. However, these results are reported without regard for number of legs, number of lanes, type of control, presence of turning lanes, and speed limit. Because all of these variables can have major effects on accident rate, the conclusions about sight distance may be misleading.

Although the safety-effectiveness of improved intersection sight distance is unclear, low-cost treatments to remove vegetation or flatten low-height embankments in sight triangles should be encouraged on RRR projects to improve the existing sight distance and to compensate for the loss of sight distance at stop-controlled intersections when the highway is widened as part of RRR improvements.

APPLICATIONS OF RESULTS TO RRR PRACTICE

The critical review and synthesis of literature produced the following major conclusions about sight distance improvements on RRR projects:

1. Horizontal and vertical alignment changes, undertaken to improve stopping sight distances, appear to be safety-effective when very short sight distances are improved to provide very long sight distances. One study indicates a 33 percent lower accident rate for crests with 100 to 300 ft of (AASHTO) stopping sight distance compared with crests with 700 ft or more of stopping sight distance. For more nominal improvements to stopping sight distance (e.g., AASHTO minimum requirements at 55 mph is 450 ft), the accident rate reduction is expected to be less than 33 percent.
2. In spite of these potential safety benefits of stopping sight distance improvements, the results of another study that produced estimates of the upper limits on the safety benefits of sight distance improvements indicate that alignment changes may only be cost-effective on highways with very high traffic volumes where major hazards (such as intersections or sharp curves) are hidden by the sight obstruction.
3. Analysis of sight distance profiles for crest vertical curves indicates a possible caution against minor lengthening of extremely substandard crests. When lengthening a crest vertical curve, there is always a trade-off whereby one portion of the highway will have less sight distance than before (see example in Figure 2). This phenomenon may only be of interest when comparing the available sight distance of an extremely substandard crest with the AASHTO requirement for that highway's operating speed. If the crest is lengthened to provide a minimum sight distance that is still considerably less than the AASHTO requirement, even though a short length of the highway will have better sight distance, the total length of highway with substandard sight distance will increase substantially.
4. Although no documentation could be found on the safety-effectiveness of low-cost treatments at restricted sight-distance crests, applications such as site-specific warning signs, advisory speed indications, or speed zones should be encouraged where hazards such as sharp curves or intersections are hidden by the crest vertical curves. In contrast to this statement, studies of the standard Limited Sight Distance sign indicate that it is vague and ineffective in reducing highway speeds.
5. Providing AASHTO minimum or greater stopping sight distance on horizontal curves may be critical to safety, particularly on highways with moderate to heavy truck traffic, for the following reasons:

   a. AASHTO-level braking on horizontal curves can lead to loss of control because the friction demand is the resultant of both cornering and braking forces. Greater stopping sight distances should reduce the probability of severe braking.

b. The AASHTO stopping sight distance requirements use the passenger car as the design vehicle and do not allow for the much longer stopping distances of large trucks when the sight restriction is a wall or a row of trees on the inside of a horizontal curve. This situation is unlike applying the AASHTO requirements to a vertical curve where the truck driver's higher eye height mostly compensates for the longer braking distances of trucks.

6. Because of the potentially greater criticality of sight distance restrictions on horizontal curves compared with vertical curves, low-cost treatments such as clearing vegetation or other minor obstructions on the inside of horizontal curves may be cost-effective on almost all highways. The offsets to obstructions specified by AASHTO to provide certain sight distances are maximum offsets that are only required toward the center of longer curves and may not be required at all on shorter curves. Also, the offset envelope for providing a certain sight distance has offsets that decrease from the maximum at or near the center of the curve to zero somewhere on the tangent approach. In other words, minor vegetation clearing can sometimes produce substantially longer sight distance, particularly on shorter curves.

7. When the prime improvement on a RRR project is highway resurfacing, particular consideration should be given to improving the skid resistance on the approaches of sight-restricted areas.

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Effect of Resurfacing on Highway Safety

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Critical reviews of the available literature and some original analyses dealing with the safety effects of road resurfacing projects were conducted and are summarized in this paper.

It is essential that the pavement surface retain its shape and structural integrity under its design loads and traffic if effective, safe, and economic highway transportation is to result. Considering the importance and expense of road resurfacing in maintaining high levels of service on the U.S. highway system, there have been few studies that directly address the issue of how accident frequency and severity change following resurfacing. These few studies indicate a response pattern that needs to be strengthened with additional studies using state-of-the-art experimental and analytic methodologies.

In the meantime, there is substantial safety information available on two aspects of pavement condition directly linked to resurfacing: pavement roughness and skid resistance. Pavement roughness is related to resurfacing because resurfacing improves surface smoothness. Skid resistance is often, although not necessarily, improved as a part of resurfacing projects. These pavement condition elements are closely related in their effects because both are important in vehicle control. Accordingly, studies of safety experience related to these factors should be consistent with and provide additional assistance and insight in estimating and bounding the safety response to resurfacing.

This analysis is concerned with resurfacing entire road sections rather than short lengths usually installed for improved skid resistance at curves or intersections.

FINDINGS

Rural resurfacing projects selected because of their pavement structural or riding condition have a small, immediate increase in overall accident experience, averaging 2 percent, and probably less than 5 percent. This is made up of a 10 percent increase in dry pavement accidents and a similar decrease in wet pavement accidents.

Rural projects resurfaced because of a large number of wet pavement accidents; for example, more than 25 percent of the total, have an immediate reduction in wet pavement accidents of from 15 to 70 percent, probably averaging 20 percent over the life of the project. In the first year, dry pavement accidents will increase up to 15 percent and total accident experience will drop as much as 5 percent.

The estimate of safety effects in the first and last years, and over the average life of the project are given in the following table:

Type of Accident	*First Year After Resurfacing*	*Final Year (%)*	*Project Average (%)*
Wet road	Down 15	0	Down 7
Dry road	Up 10	0	Up 6
All accidents	Up 5	0	Up 3

Following resurfacing, urban resurfacing projects should have an average accident reduction of about 25 percent over the life of the resurfaced pavement. There will be a small increase in overall accident severity on rural two-lane roads following resurfacing, probably on the order of 10 percent more injuries and fatalities per accident. Severity on urban streets will decrease about 25 percent.

Rationale

These findings are based on studies and opinions that suggest that resurfacing improves both road smoothness and skid resistance, which in turn ease vehicle control problems, particularly on wet pavements. Drivers respond by increasing speed and driving less carefully, leading to increased accident experience when the surface condition is not important to the safety of the road (usually the case on dry roads). Where there has been a significant wet pavement accident problem, important improvements in safety are realized after resurfacing. Over time these improvements dissipate—those dependent on improved friction more than those dependent on surface smoothness. The effects of resurfacing on accident severity are mixed, with higher speeds leading to more severe accidents and better surfaces making stopping distances shorter and thus reducing accident severity.

ANALYSIS

The remainder of this paper includes a summary of and comments on published research findings used in arriving at the foregoing conclusions. The relation of surface and road condition to vehicle and driver characteristics is discussed first, followed by a discussion of the results of studies of overall rural accident experience before and after resurfacing. Before-and-after resurfacing studies of wet pavement accidents are reported, and the changes in severity after resurfacing are considered. Safety relationships for skid resistance and road roughness are reviewed followed by a discussion of the safety interaction of resurfacing with roadside hazards and geometric design. Methodological issues in this type of analysis are discussed last.

SURFACE CONDITION AND VEHICLE AND DRIVER INTERACTION

Several characteristics of the driver-vehicle-road system contribute to the operational and safety aspects of resurfacing and interact in ways that are complex and poorly understood. The complexity of these interactions leads to safety effects that vary widely, resulting in substantial improvements under some conditions and worse accident experience in other situations.

Among the system elements viewed as contributing most to this process and its complexity are the following:

1. The response of vehicles to emergency conditions encountered on the roadway is dynamically different, depending on both its smoothness and frictional capabilities. Both light automobiles and trucks handle differently from typical automobiles under these conditions *(1)*.
2. Environmental characteristics, particularly rain and standing or running water, affect the surface condition and, hence, the difficulty of vehicle control *(2)*.
3. Roadway geometric design elements such as curves, grades, cross sections, and intersections, as well as other traffic introduce the need for vehicle control at many locations.
4. Many other characteristics are important and originate with or operate through the driver and are interpreted by the driver. The most important of these in the resurfacing context is the selection of an operating speed, which depends on the driver's feeling about the road surface. Speeds are selected at which drivers believe they can control accelerations and turns safely for both existing and anticipated conditions. Accidents stemming from surface conditions occur when the roadway cannot provide the necessary traction or smoothness.

Given that an accident occurs, its severity depends on still another set of factors that includes speed, geometric design, roadside hazards and guardrails, and stopping and control capabilities of the road.

In general, improved highway safety has resulted when there have been changes in the road environment that make it easier for the driver to accomplish both customary and emergency driving tasks. An example related to resurfacing is the large accident reduction following short skid resistance treatments at locations where many decelerations are made, such as at high-speed approaches to signalized intersections *(3)*.

After resurfacing, there are lesser accumulations of water, splash and spray conditions are lessened, and the coefficient of friction is frequently increased *(4)*. Flat curves and grades and wider lanes with better shoulders are easier to drive and are more forgiving. Other RRR critical review studies show an improvement in overall safety under these conditions. However, there may be a counter response by drivers who react by driving with less effective control through inattentiveness or increasing speed. Under these conditions safety may be decreased following an improvement.

TOTAL RURAL ACCIDENT CHANGES AFTER RESURFACING

Several of the most recently published studies indicate small and statistically nonsignificant overall safety changes after resurfacing. Nevertheless, all indicate a small increase in overall accidents. In these studies raw accident experience is usually normalized for traffic and study section length exposure by using the accident rate, *AR* (accidents per million vehicle miles of travel), or the accident frequency, *AF* (accidents per mile of road per year). In some cases the actual numbers of accidents on a section for one or more years before and after resurfacing are compared.

All of these before-after comparisons are valid only when "controlled" by contrasting them with accident experience changes at identical nonimproved sites. This better isolates the effect of the improvement from other influences on safety that may be occurring at the same time. Because resurfacing programs are based on engineering considerations and are selected to achieve an immediate highway agency objective, when that objective is safety, high-accident locations are usually selected for action. Before-after accident comparisons made when sites have been selected in this manner must be carefully reviewed to avoid the "regression-to-the-mean" effect that frequently overstates the effectiveness of the resurfacing because many of these locations are in the high-accident groups due only to chance and will experience fewer accidents "next year," regardless of resurfacing.

Midwest Research Institute (MRI) studied the effectiveness of skid prevention for the Federal Highway Administration (FHWA) *(5)*. The research team obtained information on 428 locations, 142 of which had been resurfaced around 1974, the period of the oil crisis, which affected highway operations and safety significantly. The other sections were not changed and were intended to be used as controls for comparison. The sections used in the MRI data set are from roads with poorer cross sections. Only 20 percent of the rural sections had paved shoulders; 40 percent had shoulder widths of 6 ft or more, and 60 percent had 12-ft-wide lanes.

The sites had been selected for resurfacing because of physical pavement condition, not for safety reasons. Therefore, there was little possibility of a regression-to-the-mean effect in this data set. MRI concluded that there was no significant overall effect of the resurfacing on the *AR* although the average change was a 2 percent increase.

The FHWA separately analyzed 59 rural two-lane sections not matched with control sections from eight states from the MRI data set and found a 2.2 percent increase in the *AR*, from 2.39 to 2.45 accidents per million vehicle miles following resurfacing *(6–8)*. The *AR* increased at 36 of the 59 sites. These changes were far from being statistically significant. In a recent reanalysis of these data where matched control sites were identified and the sections weighted by length and average daily traffic (ADT), the increases at both types of sites were identical and hence, resurfacing had no effect.

Another analysis of 40 of the rural MRI sites defined slightly differently revealed a 2.0 percent increase in the total number of accidents *(9)*. In this study 70 control sites recorded an increase of 6 percent over the same time period. Neither of these results was statistically significant.

The FHWA reported that studies of 48 mi of resurfacing at five sites in Arkansas indicated small and nonsignificant increases in average *AR (6, 7, 10)*. Dale reported that the *AR* dropped 24 percent, from 3.58 to 2.71 percent at 133 sites in several states and that this result was significant at the 95 percent confidence level *(11)*. However, no information on site selection or use of control sections was given for either of these studies.

Brown reported on the results of resurfacing 24 essentially tangent, controlled, two-lane rural sites in Alabama totaling 57 mi *(12)*. Information on site selection methodology was not given. The average increase in *AR* was 2.5 percent, a nonsignificant result.

The New York State Department of Transportation and FHWA reported on several resurfacing programs in that state *(13, 14)*, one of which is the Fast Track program designed to preserve the existing pavement and restore a smooth riding surface.

TABLE 1 Other Estimates of Resurfacing Effect on General and Rural Accident Experience

Road Width	Percent Improvement in Accident Experience					Source	Year
	All	Property Damage Only	Injury	Fatal	Injury/ Fatality		
General							
All sites	12	–	–	–	12	Jorgensen *(13)*	1966
	27	32	16	29	16	FHWA *(22)*	1982
	26	27	–	–	55	Creasy and Agent *(21)*	1985
	36	38	33	40	–	New Jersey *(25)*	1982
	58	58	57	–	–	FHWA *(22)*	1982
	55	73	24	–	24	Pennsylvania *(26)*	1981
	20	–	–	–	–	Creasy and Agent *(21)*	1985
Wet weather accident problem sites	63[a]	–	–	–	–	New Jersey *(25)*	1982
	58[a]	58	57	–	–	West Virginia *(27)*	1982
	57[a]	–	–	–	–	Pennsylvania *(26)*	1982
	40[a]	–	–	–	–	Creasy and Agent *(21)*	1985
	21[a]	–	–	–	–	Creasy and Agent *(21)*	1985
	64[a]	83	–	–	75	Creasy and Agent *(21)*	1985
Two-lane rural							
All sites	25	28	20	35	20	FHWA *(22)*	1982
	30	34	22	48	24	FHWA *(22)*	1982
	25[b]	–	–	–	–	Alabama *(28)*	1979
	21	–	–	–	16	Los Angeles *(28)*	1979
	21	–	16	+8	–	Dale *(9)*	1973
	22	–	–	–	–	FHWA *(22)*	1982
	15	–	–	–	–	*(17, 18)*	1981 1985
Site with wet weather accident problem	12	–	–	–	21	Jorgensen *(13)*	1966
	21	–	–	–	–	Texas *(28)*	1979
	46[a]	36	60	–	–	West Virginia *(26)*	1981
	42[a]	–	–	–	–	Texas *(27)*	1979
Four-lane	–	–	–	–	–		
Rural	–	–	–	–	–		
Other	44	–	–	–	59	Jorgensen *(13)*	1966
Four-lane	–	–	–	–	–		
Undivided	37	43	27	–	27	FHWA *(20)*	1982
Four-lane	–	–	–	–	–		
Divided	11	8	17	–	15	FHWA *(20)*	1982

NOTE: Dashes indicate not applicable.

[a]Improvement in wet pavement accidents.

[b]Skid resistant treatment.

Projects are selected on roads that do not need widening for safety or capacity. Roadsides are improved for clear zone safety only in response to a known safety hazard with high accident experience. All shoulders are paved as part of the resurfacing. At such locations, all accidents increased 4 percent, a nonsignificant change *(14)*. No significant change was found at 33 simple resurfacing projects on more than 182 mi of road with 2 years of before-and-after data. There was no change in the number of accidents on wet or dry roads, but ice and snow accidents increased by 32 percent. Under New York's more extensive Reconditioning and Preservation program, 47 projects were resurfaced, and all accidents increased by a significant 6 percent.

Studies have been summarized in several publications, but insufficient details have been provided to enable the study methodology to be evaluated. In other cases, engineers have used their judgment to estimate the safety response immediately after resurfacing *(11, 15–23)*. A summary of the values from these studies is given in Table 1. The first such study by Jorgensen summarized results from five states, and the differences among the states were found to be great, with increases recorded at some sites and decreases at others following resurfacing *(15)*. Jorgensen's final conclusions were based on Ohio data.

Many of the results given in Table 1 are from the FHWA, which determined these estimates of accident reduction levels from data provided by the states and FHWA research *(22, 24)*. The findings are based on data from after the mid-1970s and (a) use before-after comparisons corrected for traffic exposure, (b) control sites if possible, and (c) are reported only when the total accident frequency at all treated sites is large. Eighty percent Poisson confidence limits are used; however, no information on site selection is presented.

Skid resistance is at its peak immediately following treatment and declines rapidly at a decreasing rate; roughness also increases with traffic and time at an increasing rate, and the average change over the life of a project as the pavement wears out should be about 40 to 50 percent of that recorded immediately after the resurfacing.

Considering all of the preceding information, it is concluded that the overall effect of rural resurfacing is a small immediate increase of 2 percent in total accident experience. Averaged over the life of the pavement this should be less—about 1 percent.

WET WEATHER RURAL ACCIDENT STUDIES

Although dry pavement accidents usually increase after resurfacing, large reductions in wet pavement accident experience have often been found as can be observed from Table 1. The reductions occurred particularly where wet pavement accident experience was high and made up a large fraction of all accidents before the resurfacing. This conclusion was reached before 1970 by both British investigators and Jorgensen using Ohio data *(15, 29)*. Ohio policy was to resurface such sites when the number of wet pavement accidents (WPA) exceeded 25 percent of the total and where four or more were recorded in a 3-year period *(15)*. Both Ontario and New York classify sections based on a 30 percent wet pavement accident criterion *(3, 13)*. A reasonable explanation for this result can be developed from considering the improved skid, drainage, and roughness characteristics of the wet resurfaced road *(2)*.

For large safety improvements, it appears that there has been a "need" for improved surface characteristics before the resurfacing as evidenced by "high-accident" frequencies on the wet surfaces with respect to the dry pavement rate. This is an indication of an important differential between dry and wet pavement conditions at these sites when compared with safer places on the system. Such locations are usually those where "unusual" driver control is required, such as on curves, at intersection approaches, and other places where traffic conflicts are frequent. Rizenbergs et al. reported that the wet-to-dry accident ratio was only 0.23 on tangent sections and rose to 0.55 on curves where ratios as high as 0.75 were found *(30)*. Wet pavement accidents are much more clustered than other accidents and are found where unusual driving maneuvers are more common *(28)*. These intersections, curves, and downgrades occur in different mixes and numbers on a road and hence, the wet pavement safety effect would be expected to be highly variable. Also, when resurfacing projects are not initiated to remedy skidding accident problems, safety responses are less and more variable.

Under the best conditions, the wet pavement accident rate (WAR) (wet accidents per million vehicle miles of operation on wet pavements or often per million vehicle miles of travel under all surface conditions) approaches 133 to 150 percent of the dry pavement accident rate (DAR) *(29, 31)*. Of course, wet pavement accident criteria depend on local weather conditions.

The wet pavement accident experience in the MRI data set was reviewed. MRI found a nonsignificant WAR reduction of 8 percent (from 3.39 to 3.06) for about 140 resurfaced sections and a (DAR) increase of about 4 percent *(5)*. When wet and dry pavement accidents are combined, the previously mentioned total increase of 2 percent results. The control sections recorded a nonsignificant 1 percent decrease in the WAR.

The analysis closest to current study quality expectations was reported in 1979 by Sparks and Flowers *(32)* who analyzed before-after rural Texas accident data for 11 asphaltic concrete resurfacing (ACP) projects and 44 seal coated (SC) sites. These sites had been selected on the basis of high-accident frequency analysis, but it was concluded that regression-to-the-main effects would be minimal because the accident data used to make the overlay decision were not those used in the before-period analysis. Accident data, ADT, and precipitation values were developed for one-year, before-and-after periods between 1975 and 1978. The variability in precipitation between the two periods was large as revealed by 18 percent more rainy days at the SC sites after resurfacing. There were no satisfactory control sites available, and it was believed that the wet pavement change would be the sole result of the resurfacing and that resurfacing exhibited no effect on dry pavement accidents. Therefore, the dry pavement accident experience was used as the control to estimate the effect of other unknown, time-related factors.

Ignoring the effects of traffic and rain, total wet pavement accidents at ACP sites dropped almost 70 percent, from 341 to 105, whereas dry pavement accidents increased 14 percent, from 1,036 to 1,188 after resurfacing. Following the paving, wet weather accidents decreased at 90 percent of the locations. At the SC sites, dry pavement accidents increased 5 percent, and wet weather accident frequency was less by 60 percent. Wet pavement accidents decreased at 37 of the 44 sites.

A better estimate of the effect of resurfacing was made by using a multiplicative regression model that corrected for both the number of rainy days and the ADT. The dependent variable was the cross-product ratio (CPR), the fraction of wet pavement accidents after resurfacing compared with before, corrected for the change in dry accidents. For example, the CPR value of 0.31 obtained from the model for the ACP sites indicated a 69 percent reduction in wet pavement accidents compared with what would be expected considering the increase in dry pavement accidents. This value was significant at the 10 percent confidence level. There was also a highly significant 60 percent reduction for the 44 SC sites.

Because studies show that dry pavement accidents increase after resurfacing, it is concluded that the use of the dry pavement accident experience as the control overstates the effect of the wet accident improvement and that over the life of such improvements, a 25 percent reduction in the average wet pavement accident experience would be found for projects with high safety payoffs.

The wet pavement accident experience in the MRI rural two-lane data set was analyzed similarly as a part of this review *(8)*. MRI found an 8 percent wet accident reduction on the 142-site data set *(5)*. The FHWA did not report an analysis of the wet pavement accidents for this group of sites. There were 54 sites for which data on ADT and rain were available for an analysis similar to that conducted in Texas.

The model showed that a 22 percent decrease in accidents could be attributed to resurfacing. Because dry pavement accident experience was up 7 percent, a better estimate would be a 15 percent improvement in wet pavement accidents due only to resurfacing. There were 30 sites where wet pavement accidents exceeded 25 percent of the total, 21 sites where the wet pavement accident rate exceeded 1.0 accidents per mile per year, and 18 sites where both conditions were met. A 28 percent reduction in wet pavement accidents was reported at these sites accompanied by a 17 percent increase in dry pavement accidents.

The data were also analyzed by Persaud who used a method recently developed by Hauer *(33, 34)*. He corrected for traffic and rain condition exposure and concluded that an unbiased estimate of the reduction in accidents at 28 sites with high wet pavement accident experience was 18 percent. This reduction appears to be greater for roads with 2,500 to 5,000 vehicles per day.

New York analyzed projects of this type and the effect on wet weather accidents *(13)*. At 56 locations with higher wet pavement accident experience, there was substantial improvement following antiskid grooving treatment and the placement of overlays. On 37 roads that had been grooved, the wet accident rate was down 53 percent, and all accidents were down 21 percent; these results are significant. At 19 locations where high-friction overlays were applied, the wet accident rate dropped a significant 56 percent with similar results that were not statistically significant for all accidents. At 47 sites resurfaced with high-friction overlays but where wet pavement accident experience was low, accidents dropped a nonsignificant 4 percent.

Including ice and snow accidents in the MRI data file for the 27 sites recording these types of accidents, the total number of accidents increased by 15 percent following resurfacing. In New York, snow and ice accidents increased by a nonsignificant 12 percent after resurfacing, and other New York studies have always shown an increase in these types of accidents *(13)*.

The earlier studies described did not provide detailed information on the characteristics of the resurfacing material supplied, and because there are many differences among the possible coatings, the differences could be great as indicated in the New York studies *(14)*.

It is concluded that where wet pavement accident experience is high and also a large fraction of the total accident experience, wet pavement accidents can be reduced by 45 percent in the first year and an average of 20 percent over the life of the resurfacing. All accident experience should improve at these sites, generally up to 5 percent in the first year.

URBAN EXPERIENCE

Because of speed and many other differences between urban and rural areas safety responses to resurfacing may differ in cities. The results of other estimates are given in Table 2, and there is no recent detailed information that can be evaluated to modify these judgments. Reviewing these values, urban resurfacing should result in an average 25 percent reduction in accidents over the life of the pavement.

ACCIDENT SEVERITY

Considerable information is available on the difference in severity of accidents occurring before and after resurfacing. In this section changes in fatal and injury accidents after resurfacing are expressed as the percent change in the average percent that such accidents are of the total—severity ratio. A severity ratio of 30 percent means that 3 out of 10 accidents involve a fatality or personal injury. A 10 percent increase in the severity ratio would change that value to 33 percent.

The early Jorgensen study indicated widely varying severities for rural two-lane roads *(15)*. As shown in Table 1, accident severity was reduced following resurfacing, with values ranging from 5 to 15 percent better than the reduction in accidents at rural locations where the high wet pavement accident experience conditions are met.

Larsen reported a 16 percent increase in severity per accident for 33 New York State resurfacing projects *(14)*. Accident severity also increased at 37 grooved road sites *(13)*. Severity was reviewed for 47 projects that were resurfaced in 1981 and 1982 to obtain better friction where no other improvements were made. Almost 3 years of before-and-after accident data on these projects were available. There was a 3 percent increase in fatal and injury accident severity per accident for these projects. This 3 percent increase was not significant.

Brown reported on 57 mi of improvements at 24 controlled sites in Alabama *(12)* and concluded that accident severity was reduced after the improvement, with the personal injury and fatal accident severity improving by 14 percent, whereas the property-damage-only accidents remained unchanged.

A French study indicated a 10 percent greater reduction in individual accident severity than the impressive 59 percent reduction in overall accident experience *(35)*. However, this study appears to suffer from regression-to-the-mean effects.

The FHWA safety program evaluation estimated a 10 percent reduction in fatal and injury accidents against an overall accident reduction of 22 percent, an indication of a 15 percent expected increase in severity *(22)*. The FHWA reported small increases in severity of approximately 5 percent per accident *(22)* for three states.

McFarland and Rollins recently estimated that improving a road surface would produce a small, perhaps 5 percent reduction in severity *(19)*. On the other hand, Smith concluded that resurfacing highway sections would increase severity by between 5 and 10 percent *(20)*. Smith found no change in severity with ADT on Virginia secondary roads as well as no difference in severity between wet and dry conditions.

Beatty reported a 1.3 percent increase in the severity ratio—the percent of accidents involving an injury or fatality—for each 1-mph increase in speed *(36)*.

It is concluded that severity on rural resurfacing projects is about 10 percent greater, although changes of as much as 15 percent have been recently recorded.

TABLE 2 Estimates of the Effect of Resurfacing on Urban Accident Experience

	Percent Improvement in Accident Experience						
Road Width	All	Property Damage Only	Injury	Fatal	Injury/ Fatality	Source	Year
All	42	–	46	–	–	Jorgensen *(15)*	1966
	61[a]	64	56	–	–	West Virginia *(52)*	1982
Two lanes	25	27	19	–	19	FHWA *(24)*	1982
Four lanes	20	28	–	–	–	FHWA *(24)*	1982
Undivided	52	53	48	–	47	FHWA *(24)*	1982
Four lanes Divided	17	20	10		10	FHWA *(24)*	1982
Over four lanes Undivided	52	53	48	–	47	FHWA *(24)*	1982
Over four lanes Divided	32	39	16		16	FHWA *(24)*	1982

NOTE: Dashes indicate not applicable.

[a]Improvement in wet pavement accidents.

SURFACE CONDITION AND ACCIDENT STUDIES

Skid Resistance Studies

Skid resistance is often improved on resurfacing projects. The 1976 National Highway Safety Needs Report estimated that improved skid resistance was the ninth most effective highway safety countermeasure with a potential for saving almost 3,500 lives over a 10-year period *(37)*.

Many studies have been conducted on the relationship between skid resistance and accident experience, including both cross-section studies and longitudinal studies of accident rates before-and-after skid treatments were implemented.

The MRI study measured a small decrease in the skid resistance for the resurfaced sites and, as described previously, no significant overall safety change was recorded *(5)*.

There are many studies that show a relationship between measures of skid resistance and accident experience. Jorgensen found such a result for skid resistance in data from Texas. Where the skid number (SN) (speed standardized friction coefficient expressed as a percent) was a very poor 10, the accident rate (AR) was 3.5; it decreased to 1.5 at locations where the SN was an excellent 60 *(15)*. Other studies in Texas and Arizona showed similar effects *(38, 39)*. The FHWA reported a West Virginia study of nine urban and rural projects in 1980–1981 where the SN was increased by 20 and the wet pavement accident rate decreased by 1.0 accidents per million vehicle miles *(40)*. The MRI cross-section study revealed that as the skid number increased by 10, the wet accident rate decreased by 0.5 accidents per million vehicle miles. However, the effect was less at sites where the SN was higher. This is consistent with Burchett's findings that a declining exponential function captures the relationship *(31)*.

In Germany Beckmann also found a similar relationship involving the percent of wet pavement accidents *(41)*. Rizenbergs found an exponential relation between the skid number and the percent of wet pavement accidents *(26)*. When the SN was 16, this percentage was 44 percent, and it declined exponentially to 18 percent when the SN was 58. German studies indicated that the percent of wet pavement accidents declined with increasing skid resistance, from 70 percent at $f = 0.18$ to 30 percent where $f = 43$ *(42)*.

Burchett also showed that the results were nonlinear with respect to ADT *(31)*. Using Texas data, McFarland found a much smaller decrease in wet accident rate (0.11) for the same SN change *(43)*. Levy explored the relationship between skid resistance and safety on 94 sections of Indiana highway involving 4,416 accidents between 1973 and 1975 and found that the wet-to-dry accident ratio stratified by road type related weakly to the SN *(44)*.

Rizenbergs and Burchett studied almost 8,500 accidents between 1969 and 1971, along with the friction of almost 1,500 mi of rural two-lane Kentucky highways and found a large scatter obscuring any relationship between skid number and any measure of accident experience *(26)*. However, of all the measures investigated, the results with the wet-dry accident ratio were the best. All investigators have noted that the skid number–wet accident rate relation is a very complex function of many conditions and that the variance explanation of their models was very poor, never more than 10 percent according to Sparks and Flowers *(32)*.

In an effort to take some of these differences into account, MRI related the wet accident rate to the dry accident rate and found that at sites with a high dry accident rate, the wet accident rate was more sensitive to the available SN *(5)*. The relationship was highly nonlinear. The interaction with the dry accident rate indicates the importance of the relative exposure to emergency control maneuvers and is a clear indication of the presence of other safety problems. Accordingly, this is a reasonable result for sites at which there is a need for additional friction to avoid accidents.

As speeds increase, there is a significant drop in the available skid resistance. This speed increase works against safety on resurfaced roads.

Road Roughness, Speed, and Accident Studies

The cited accident studies reveal a consistent increase in dry pavement accidents after resurfacing. The MRI found a significant 15 percent increase in the dry accident rate, from 2.35 to 2.70 *(5)*. Several researchers have concluded over many years that the increase in accident experience following resurfacing can only be attributed to driver response to the changed road and most likely to the selection of a higher speed *(5, 13, 15, 29)*. However, there is little information available to support this belief. One early British study showed a speed increase of almost 5 mph following resurfacing of a "very irregular" road *(29)*. Zegeer described the results of a Kentucky study in which the average speed before and after resurfacing of a "very rough" road increased by 8 mph *(45)*.

A possible explanation for the higher dry accident rate is that resurfacing usually makes relatively little change in the ability of the dry road to provide traction or substantial change in road roughness *(41)*. It would be expected that in cases where the desired speed is high, locations that do not have a safety problem at lower speeds would become more hazardous at higher speeds and a larger number of accidents would result.

Accordingly, additional indirect evidence supporting such an effect was sought. Speed selection depends on many elements, including the driver's desired speed, the speed limit, vehicle capabilities, enforcement level, and constraints of other traffic. Most important within the context of this study is the effect of the perceived ease of travel related to road surface roughness.

A Swedish driver attitude and operational study conducted before and after the initial surfacing of a gravel curve showed that this improved ease of travel was easily detected by the subject drivers and that they increased their speed on the curve following its surfacing *(46)*.

Speed studies over many years show little difference between wet and dry conditions. In a recent study of Illinois data, speeds on wet roads were lower but the difference was usually well under 2 mph, and never as much as 5 mph *(25)*. MRI found an average speed difference between dry and wet conditions of less than 2 mph *(5)*.

Results of cross-section studies exploring the effects of surface roughness on speed were reviewed. The most widely accepted measure of road roughness is the Pavement Serviceability Index (PSI) (also called PSR). On most highway systems in the United States, PSI values range from 2 when the surface is in very poor condition to 5 when it meets the highest smoothness standards. Roads typically have a PSI of 4 immediately after resurfacing, which declines at an increasing rate with time and traffic until additional rehabilitation is necessary. Roads are considered ready for possible resurfacing when the PSI reaches a value of 3. About 10 percent of the U.S. rural mileage has a PSI value less than 2, and only 20 percent is greater than 4 *(20, 38)*. There are no longitudinal studies documenting speed changes over time as the PSI decreases at a location. Cross-section speed-PSI studies at locations with varying roughness show higher speed with higher PSI values *(26, 27)*. Studies of rural highways in Ontario revealed about a 0.7-mph decrease in average speed per unit of PSI drop *(26)*. McFarland estimated a 5-mph drop from a smooth road speed of 25 mph and a 12-mph drop from 60 mph as the PSI changes from 5 to 2 *(43)*. Hazen concluded that speed decreases noticeably as the PSI drops below 3 *(47)*. He estimated that the average speed at a PSI of 2 is about 91 percent of that at 5 and 93 percent of that at 4. Extending Hazen's analysis, Zaniewski recently reviewed research findings and concluded that a road operating at an average speed of 49 mph when the PSI is 5 would average 45 mph when it is 2 *(27)*. The overall effect on speed therefore appears to be an increase of about 5 mph following resurfacing of a road with a PSI of 2 and raising its smoothness to 4. Numerous cross-section studies have shown that surface smoothness and skid resistance affect accidents. In all but one study the increase in road roughness or decrease in skid resistance was associated with increased accident experience. Locations at which accident experience is worse on rougher roads have been found in Jamaica, Kentucky, Kenya, Great Britain, and Ontario *(3, 29, 31, 48)*. It should be noted that these results are contrary to the assertions previously described in this paper that improved smoothness increases accidents as was found in a recent study by Zaniewski *(27)*, who studied the relationships between PSI and 1976 accident experience for 1,800 rural, primary and secondary road sections in Texas totaling 8,300 mi. The study showed that accident frequency increased slightly but significantly with a smoother pavement surface on two-lane rural roads with ADT in the 1,000 to 8,000 range. His findings indicate an increase of about 0.6 accidents per million vehicle miles as the road condition varies from a PSI of 4 to 2.

Roughness and Skid Resistance

Zegeer studied records for 2,300 mi of two-lane rural Kentucky highway analyzed for resurfacing in the early 1970s *(44)*. Roughness and skid resistance were quantified separately and regressed against accident measures developed from 2 years of accident data. The roughness rating correlated strongly with road defect accidents and somewhat with the percentage of wet pavement accidents. Rougher roads had higher road defect accident experience and a decreasing fraction of wet pavement accidents. The skid resistance rating correlated well with the wet and dry pavement accident rate, and the wet pavement accident percent correlated with both of these measures, increasing with decreased skid resistance. Accidents involving road defects did not correlate with this measure.

Contrasting these two measures, skid resistance was strongly related to wet pavement accident experience whereas roughness was not. Accidents involving road defects responded to roughness but not to the skid measure.

Janoff cited a study of the relation between roughness and wet pavement accident experience on a section where the surface was grooved, which increased its PSI from 2.1 to 3.6 *(49)*. The wet pavement accident rate decreased 15 percent whereas it increased from 35 to 82 percent on untreated control sections.

INTERACTIONS

There are so many driver-vehicle-road elements interacting in such complex and poorly understood ways that unclear and conflicting results are likely. However, the experience on previous RRR projects involving other improvements should be of value. It would be expected that resurfacing would be less effective alone than in combination with other changes. Larsen found accident decreases *(14)* in a study of 79 extensive RRR projects in New York. All accidents decreased 21 percent, property damage accidents decreased 40 percent, and injuries decreased 9 percent (a 10 percent severity increase) following the improvement. Both wet and dry pavement accidents decreased significantly—33 and 17 percent, respectively.

Sanford recently described Illinois studies involving 44 rural, two-lane, major-route RRR projects with a total length of 284 mi *(50)*. Each section was more than 2 mi long, the improvements were made between 1978 and 1981, and accidents were analyzed for 2 years before and after the improvements. There were no control sections, and two-thirds of the projects involved widening and resurfacing and one-third resurfacing with some roadside improvements plus treatment at high-accident locations. No information on site selection techniques was given. The total AR decreased 25 percent, from 2.32 to 1.73, and the fatal-personal injury rate decreased 18 percent, from 0.89 to 0.73, another 10 percent increase in severity per accident.

The magnitude of potential accident payoff at locations with wet weather accident problems where multiple improvements are made is shown in the French study summarized by Schultze *(35, 42)*. At 51 locations with high wet pavement accident experience, there was an average of 64 wet pavement accidents out of a total of 85 in 1969—a ratio of 75 percent. Following treatment, which included traffic controls and widening as well as surfacing and antiskid treatments, wet pavement accidents averaged 6.6, and the total was down to 35. The overall reduction was 59 percent (90 percent for wet pavement accidents); no significant effect on dry pavement accident experience was recorded. This study probably suffers from regression-to-the mean effects, however.

Sensitivity to Roadside Conditions

Of particular importance are interactions involving roadside hazards and road geometry—two improvement types expected to be important alternatives to or supplements in RRR projects involving resurfacing.

Two fundamental physical factors are at work when considering the interaction of roadside improvements and resurfacing: the increased energy that must be dissipated because of higher speeds, and the greater ability of a road with a higher SN to dissipate this energy. With an increase in speed, the errant vehicle is more likely to reach a roadside hazard. For the small increases in speed that have been described in this review and typical improvements in friction, calculations show that fewer accidents involving braking might be expected because the increased friction, if needed, can quickly dissipate the energy that is created by the higher speed. Also, the driver may be expected to maintain greater control after the pavement is resurfaced.

Only one study relating resurfacing and roadside hazards was found *(13)*. In New York two types of improvements were made involving 81 resurfacing projects in 1981–1982 with almost 6 years of before-after accident data available. Thirty-four of the projects involved both resurfacing and extensive roadside safety work. For these projects, all accidents were reduced by a significant 6 percent, fatal and injury accidents were reduced by a significant 10 percent, and severity was reduced by 5 percent. Wet pavement accidents were reduced by 17 percent and dry pavement accidents were reduced by 7 percent—both significant values. Snow- and ice-related accidents increased by a nonsignificant 12 percent. Considering the type of accidents, head-on collisions were reduced by a significant 63 percent, and fixed-object accidents decreased by a significant 20 percent. Left-turn collisions increased by a significant 75 percent.

The remaining 47 projects were resurfaced without roadside improvements. All accidents were up a significant 6 percent, and severity increased slightly as fatal and injury accidents increased 9 percent. Both wet and dry pavement accidents decreased a nonsignificant 4 and 2 percent, respectively. There was an 84 percent improvement in head-on collisions counteracted by increases of 22 percent in fixed-object accidents and 103 percent in left-turn collisions.

A comparison of the fixed-object accidents on these two types of sites is of particular interest because of the different treatment of these hazards. The 22 percent increase where the roadside was not improved was 90 percent of the increase at these sites. At the 34 sites where the roadside was improved, fixed-object accidents decreased 20 percent, 63 percent of the safety improvement. These reductions were concentrated among accidents involving utility poles and trees. One-third of the fixed-object accidents at the nonimproved roadside locations involved trees and utility poles.

Sensitivity to Geometric Design

Recent quantitative data on the safety effects of resurfacing as related to geometric design is almost completely lacking. In 1966 Jorgensen concluded that resurfacing roads with poor geometrics would lead to higher speeds and an increase in accident experience *(15)*. The data supporting this conclusion were not shown although the conclusions are consistent with other findings reported in this paper. Early German studies of wet weather accidents revealed the surprising importance of geometric design *(51)*. MRI could identify no relation between wet weather accident rate and geometry *(5)*. Cleveland's analysis of part of the MRI data set indicated a strong effect of intersection density and horizontal curvature on accident frequency but no effect involving resurfacing itself *(9)*.

Zegeer found a high correlation between roughness and the percent of accidents on curves with this percent increasing on the rougher roads *(45)*, but found no identifiable correlation between skid resistance and the fraction of accidents on curves.

The data sets analyzed as a part of this synthesis undoubtedly have confounding geometric effects. For example, in the analysis of the MRI rural data set, there was an indication that lane width differences among the 54 sites might be important although the results were far from statistically significant.

ANALYSIS PROBLEMS

Many problems exist in accident analysis methodologies and all of the studies reviewed have some defects. The problem is exacerbated by the complexity of the causal element interactions, the wide variability in results, and the small effects that have been found. These effects are usually far less than the uncertainty in the results. For example, the MRI data for accident rate changes at individual sites range from an improvement of 50 percent to a worsening of 400 percent, averaging the 2 percent change as described previously *(6)*.

Reference has been made to the regression-to-the-mean effect throughout this paper. Another problem is accident rate measures. An analysis of the MRI data revealed that the effects of section length and ADT could not be accounted for by the linear relation needed to use accident rate and accident frequency (accident per mile per year) rates across the range in length and traffic flow desired *(9)*. In the model, the accident rate was found to be directly proportional to ADT and varied as the square root of section length. Because the sections in the MRI study varied in length from 0.6 to 18.4 mi, the effect is substantial, and accident rate values on longer sections of this data set cannot be compared with confidence with those on shorter lengths.

Changes over time may also be important. For example, the problems of lighter automobiles are accentuated where the pavement is rough and the friction is low *(1)*. The expected continuing increase in the numbers of these vehicles argues in favor of resurfacing.

The overall effects of resurfacing may be summed up as follows by type of project. For projects selected because of structural quality or poor riding condition, all accidents increase immediately an average of 2 percent. Typically, this will be made up of a 10 percent increase in dry pavement accidents offset by a similar decrease in wet pavement accidents. For projects selected because of high wet pavement accident experience, total accident experience will drop as much as 5 percent. In the first year, dry pavement accidents will increase (up to 15 percent) but wet pavement accidents will decrease from 15 to 70 percent. Over the life of the project, wet pavement accidents probably average a 20 percent reduction.

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Future Changes to the Vehicle Fleet: Effect on Highway Safety

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Following the oil crisis of 1973–1974, a revolution took place in the automobile industry. A second "energy shock" occurred in 1979–1980. As a result, the price of fuel rose sharply. The federal government established industry fuel economy standards that became progressively more stringent through 1985.

In response to these actions, U.S. automobile manufacturers downsized their vehicles—in length and width and weight. Imports, which were generally smaller, cheaper, and more fuel efficient, captured a larger share of the market. As these trends became established in the late 1970s and early 1980s, many projected a continuation of the trends and the emergence of a substantial volume of mini- and microvehicles.

During the same period, truck size and weight laws were also changed. The 1982 Surface Transportation Assistance Act (STAA) *(1)* was passed, which forbids the states from prohibiting, on designated highways, semitrailer lengths as large as 48 ft (45 ft was the previous maximum length in common use), truck widths of 102 in. (up from 96 in.), truck weights of 80,000 lb (up from 73,280 lb), and operation of truck tractor-semitrailer-trailer combinations (doubles) with trailer lengths up to 28 ft.

In light of these changes and predictions of the future, it is appropriate to ask what effects these alterations to the vehicle fleet will have on highway safety. More specifically, what effect will the changing fleet have on relationships between highway safety and roadway features, for example, lane width and horizontal and vertical curvature.

Some of these issues are addressed in this paper, which is based entirely on the literature, and do not represent any new data collection efforts. However, the literature reviewed is quite diverse, most of it is quite recent, some of it is unpublished, and much of it is "expert opinion."

The paper is organized into three major topics: (a) a review of the vehicle and highway features of potential interest; (b) a review of recent trends in, and future projections for characteristics of automobiles and trucks; and (c) an assessment of how the relationships between highway safety and roadway features may be affected by future changes to the vehicle fleet, and therefore how resurfacing, restoration, and rehabilitation (RRR) planning might be affected.

VEHICLE AND HIGHWAY FEATURES OF INTEREST

Highway safety has been examined from the viewpoint of roadway elements and vehicle characteristics in two major syntheses. These are reviewed briefly, and the 10 roadway features of interest to the RRR study are individually addressed. [The results of the RRR study are published in TRB *Special Report 214: Designing Safer Roads (2)*.]

FHWA Synthesis

A Federal Highway Administration (FHWA) synthesis *(3)* focused on the roles of traffic control and roadway elements in highway safety. The study examined 17 subject areas, one of which was roadside features, an area that has been addressed at length in the literature. Specific findings are summarized in the section in this paper titled Application of Results to RRR Projects. The other subject area for which vehicle characteristics were believed to be of importance in the FHWA synthesis was intersections—three concerns were voiced: collisions between large and small vehicles, vision limitations for drivers of small vehicles occasioned by the presence of larger vehicles, and anticipated extra driver workload caused by the need for clutching and gear shifting and confined interior space.

FHWA Contractor Study

McGee et al. recently completed a 2-year FHWA research study, *Highway Design and Operations Standards Affected by Vehicle Characteristics (4)*. A major portion of this study was a review of geometric design and traffic control criteria that are affected by vehicle characteristics, as well as an assessment of the appropriateness of relationships between those characteristics and the geometric design and traffic operations criteria. Fifteen standards or traffic operations criteria were identified that incorporated one or more vehicle characteristics. In some instances, the characteristic is included explicitly; for example, driver eye height is a parameter used in the measurement of acceptable sight distance. In other cases, the characteristic is not explicitly stated, but is implied. An example of this is the effect of vehicle width on lane width, which is not part of the written American Association of State Highway and Transportation Officials (AASHTO) policy, but is inherent in it because the policy was originally developed based on research pertaining to vehicle width.

Twelve of the 15 standards examined relate to 4 areas of interest to the RRR study: lane width, horizontal and vertical curves, sight distance, and intersections. The McGee et al. study does not mention the other six highway features of interest to RRR, although clearly based on the literature cited, most if not all of these features were also reviewed. The study findings in this regard are presumably covered by the statement, "the absence of a particular standard indicates that it was determined that a vehicle characteristic does not influence the standard."

The vehicle characteristics found to influence one or more standards are weight, length, height, width, wheelbase, underclearance, off-tracking, acceleration ability, maneuverability, side friction factors, braking ability, driver eye height, suspension, load distribution, and headlight characteristics. Several of these involve, in turn, more specific vehicle characteristics. For example, offtracking is a function of the number of units (if a combination vehicle), the wheelbase of each unit, locations of hinge points, vehicle widths, and overhangs. Braking ability encompasses the braking system, type and condition of tires, load and load distribution, and so forth.

Roadway Characteristics of Interest to the RRR Study

Each of the 10 roadway features reviewed by the Committee for the study of Geometric Design Standards for Highway Improvements is assessed as to whether they are likely to be influenced by reasonable changes in vehicle characteristics. This assessment is based largely on the preceding two major studies, supplemented by other literature on sideslopes, roadsides, and pavement edge drop-offs.

1. *Shoulder width*: No relationship between vehicle characteristics and shoulder width is apparent in the literature. If it is implied that shoulder widths should safely accommodate parked vehicles, then vehicle width would be of concern. However, because major changes in vehicle widths are not expected, this feature will not be considered further.
2. *Shoulder type*: No relationship between vehicle characteristics and shoulder type is evident in the literature other than the obvious implication that a shoulder must have the stability necessary to sustain loads imposed by the vehicles using them. On small-radius horizontal curves where the pavement width is not adequate, truck off-tracking could lead to increased shoulder usage, and hence increased shoulder damage unless it is designed to accommodate such loads. This issue is best covered by lane width and horizontal curvature considerations, however, so shoulder type will not be considered further.
3. *Lane width*: Lane width was found to be implicitly linked to vehicle width by McGee et al. *(4)*. However, the research supporting the STAA-mandated 102-in. truck width on roads with 12-ft lane widths *(1)* did not indicate safety degradations relative to the earlier 96-in. widths. On roads with less than 12-ft lanes, the safety impacts of wider trucks should be considered.
4. *Horizontal and vertical curvature*: These features are directly related to the ability of the vehicles to stop or accelerate. Thus, vehicle characteristics such as driver eye height, braking ability, length, width, and engine performance may be important. Also, as mentioned earlier, vehicle off-tracking may be a problem on small-radius horizontal curves.
5. *Sideslope*: Although the two major FHWA studies did not identify sideslope as being related to vehicle characteristics, others suggest there is a vehicle size relationship. Woods found that the likelihood of rollover for smaller vehicles is greater than that for larger vehicles *(5)*. Woods further states that the testing that led to the guideline of 3:1 unprotected sideslopes involved only large vehicles, and that therefore, a 4:1 value should be used where practical, for the benefit of smaller cars. Unfortunately, no data exist to quantify the effect of vehicle size.
6. *Roadside*: Again, although the two major FHWA studies did not address roadside issues such as guardrails, poles, and the like, their contribution to safety has been well researched. Vehicle characteristics such as weight and bumper height are very important.

7. *Sight distance*: Stopping and passing sight distances depend strongly on vehicle braking and acceleration capabilities, and to some extent on driver eye height and vehicle length.

8. *Bridge width*: Although the FHWA synthesis *(3)* examined bridge width, it did not relate it to any vehicle characteristics. An independent review of the literature on bridge width indicated that the vehicle-width relationships have not been addressed. There is some suspicion that trucks are more of a safety problem at narrow bridges than automobiles, but issues such as 102- versus 96-in.-width trucks or narrower versus wider automobiles have not been examined. Widening of bridges is usually a matter of many feet, not inches, so it is unlikely that small changes in vehicle width will be important.

9. *Pavement edge drop-off*: Pavement edge drop-off poses a problem for vehicles whose right wheels have moved off the pavement and must therefore remount the drop-off. In attempting to remount the pavement, the driver may lose control of the vehicle, causing it to cross into opposing lanes of traffic.

Graham and Glennon *(6)* provide an extensive review of the literature on this topic, as well as new simulation results. They discuss several experimental studies using a variety of automobile sizes (including minicompacts), drop-off heights, speeds, and maneuvers. The experimental studies indicated only small differences as a function of vehicle size, up to the drop-off height tested (~4 1/2 in.), so their authors tended to merge results across vehicle sizes. The simulation results also revealed that, "responses to the drop-off were nearly identical for a mid-sized and a compact automobile." They therefore recommended maximum drop-off heights based on criteria other than vehicle size.

10. *Intersections*: Although interchange design has been found to involve many vehicle characteristics such as vehicle length, deceleration, and acceleration capability, most of these are not expected to change enough to affect geometric design. The exception is the intersection return radius, which is strongly affected by vehicle off-tracking characteristics.

PRESENT AND FUTURE DISTRIBUTIONS OF VEHICLE CHARACTERISTICS

Trucks

Fewer data are compiled on vehicle characteristics of trucks than of automobiles, with the exception of weight data. And essentially no formal studies have been conducted of projected distributions of truck characteristics. However, this situation is likely to change.

The Surface Transportation Assistance Act of 1982 *(1)* made a number of changes in allowable truck specifications and mandated a number of studies, including the RRR study *(2)*. Another STAA study, the "Double Trailer Monitoring Study," was conducted by the Transportation Research Board, and the results are published in *Special Report 211: Twin Trailer Trucks (7)*. Although it focused on double trailer configurations, other truck issues were examined as well. The findings on current and projected usage, although largely based on expert opinion, are perhaps the most definitive in existence.

At present, the principal data on truck characteristics deal with truck weights and, to a lesser extent, vehicle classifications. Such data are obtained annually, on a voluntary basis, from the states by FHWA for the Annual Truck Weight Study. However, they do not represent a statistically valid sample. The Census Bureau conducts a Truck Inventory and Use survey every 5 years of truck owners of a sample of registered vehicles. The most recent survey was in 1982; the results have not yet been published.

It is likely that changes in the distributions of truck characteristics will occur. The impetus behind the changes is the STAA, which, as noted earlier, mandated that the states could not prohibit trucks with certain characteristics from the Interstate system or other designated routes. Some of these designated routes are likely to be highways of interest to the RRR process. The changes to be expected are longer (48-ft) semitrailers, wider (102 in.) trucks, heavier (80,000 lb) trucks, and doubles replacing a portion of the tractor-semitrailer population.

It is not possible at this time to estimate the magnitude of the changes, or the timetable over which they will take place. Some have advocated predictions based on experiences of the western states, which historically have been more liberal in their legal limits on truck sizes and weights. Indeed, substantial data have been obtained in research studies, such as that obtained by Vallette et al. *(8)*. Unfortunately, the data are unlikely to be representative or predictive of the rest of the country because of methodological flaws in the study and because the data are predominantly from California *(9)*. For example, the majority of the doubles in California are tankers, flatbeds, or bulk commodity trailers (e.g., rock, gravel), as opposed to the enclosed van trailers typically expected to be used by general commodity carriers.

Beyond these changes, what else might be predicted for the future? Based on the evolving history of trucks in the United States, this author believes that no other major changes will occur by 1990, but perhaps by the year 2000, particularly in the western states.

One change, which will occur gradually, will be the expansion of the designated network. Trucks now largely excluded from two-lane roads will become more frequent users of such facilities. In some states, most or all of the primary system is already "designated." The concept of "access" to designated routes can be expected to gradually increase the roadway mileage used by larger trucks. Similarly, "illegal" use of some roadways will undoubtedly occur, as enforcement is very difficult.

A second gradual change likely to be seen, at least in some areas, is increased use of semitrailers longer than 48 ft. The latter dimension is the minimum ceiling a state can impose. The majority of the states already allow up to 53-ft lengths. Although 48-ft trailers are presently becoming the industry standard, use of 53-ft trailers will probably increase, and off-tracking will then become a greater problem.

Other changes can be predicted based on existing configurations presently operating in limited areas of the country. These include triple trailer combinations and so-called turnpike doubles in which each trailer is up to 48 ft long. Some increase in the weight ceiling may also occur, perhaps in concert with a revised "bridge formula." However, it is doubtful that future trucks will be much wider, as the investment in the infrastructure is too great, and industry pressure for this type of change is weak. The implications of changes in truck characteristics are discussed in the section on Applications of Results to RRR Projects.

Automobiles

The ultimate goal of this subsection would appear to be a set of curves or tables detailing the projected distributions of the vehicle characteristics noted earlier. Indeed, if this paper had been written in 1981, that could have been done. Since then it has become gradually apparent that the projections of that time would not be realized. It was only toward the end of 1984 that the impact of changes in the world economy, new technology, and other pressures on automobile marketing became evident. This dramatic turnabout is not widely appreciated and not yet broadly discussed in the literature. Reasons for these changes from the earlier projections are discussed next, followed by a more qualitative (but quantitative where possible) update on projections. This subsection then concludes with thoughts on the longer-term outlook.

Early Predictions

Following the petroleum energy shortages of the mid- and late 1970s, there was a two-pronged response in the automobile marketplace. First, the industry designed and marketed more fuel-efficient vehicles primarily through "downsizing," accomplished largely by manufacturing front-wheel drive designs. Second, the public sought out more fuel-efficient vehicles, most notably Japanese imports and diesels.

The federal government set standards for corporate average fuel economy (CAFE) that mandated progressively improving fuel economy that would reach 27.5 mpg by 1985. These standards were developed "hand in hand" with the industry in hopes that the standards would be achievable. Projections of automobile sales for 1981 through 1984 by size and weight class were made by the National Highway Traffic Safety Administration (NHTSA) in the mid-1970s and published in 1977 *(10)*.

Glauz et al. used these projections to predict the vehicle-mile-weighted characteristics of automobiles on the roads *(11)*. Examples of these projections are given in Table 1. These projected impacts were fairly large at first, but only modest changes would occur after 1985. Deducting the vehicle loads from the weights given suggests average curb weights of about 3,200 lb in 1985 and 3,000 lb in 1995.

TABLE 1 Projected Average On-Highway Passenger Vehicle Characteristics *(11)*

Year	Inertial Weight[a] (lb)	Engine Displacement ($in.^3$)	Engine Net Horsepower
1978	3,880	297.3	143.2
1981	3,732	259.8	125.9
1985	3,508	227.8	109.4
1990	3,377	213.4	103.3
1995	3,352	211.1	102.6

[a]Empty weight plus fuel and coolant plus 300 lb (500 lb for light trucks).

During the late 1970s and early 1980s, most predictions were more extreme. For example, Figure 1 shows a 1985 weight projection prepared by NHTSA in 1981, as quoted by Viner *(12)* and others. It suggests a median weight of about 2,300 lb and a practical maximum of 3,000 lb for automobiles sold in 1985. In 1981 General Motors (GM) predicted that nearly 20 percent of sales would be diesels and 60 percent would be four-cylinder gasoline engines *(13)*. GM also predicted *(14)* that vehicle lengths would decline from 1985 to 1990 (Figure 2) and that tread widths would decrease from

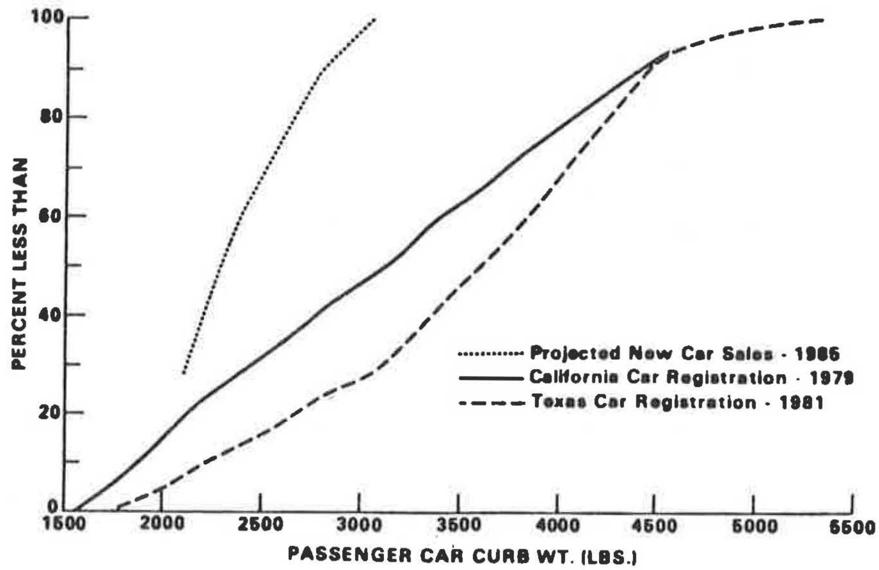

FIGURE 1 Earlier weight projections (12).

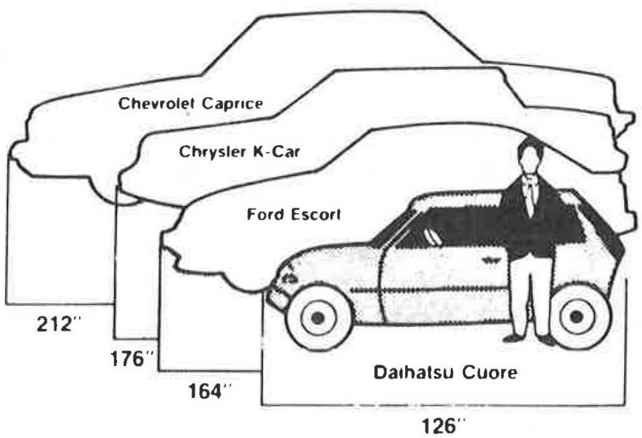

FIGURE 2 Micro vehicle size example (15).

1978 to 1985 but then stabilize (Figure 3). McGee et al. *(4)* compiled data and projections made in the 1980–1982 period, which suggest vehicle lengths will continue to decline from 1970 through 1990, closely in agreement with GM projections.

The most extreme of the earlier predictions are those dealing with mini- or micro-vehicles, alternatively called urban cars, city cars, or "Kei" cars. One example of such a vehicle is the Daihatsu Cuore shown in Figure 2 *(15)*. Among those projecting their substantial impact in the United States were Lave et al. *(16)*, Sparrow and Whitford (15), and Woods and Ross *(17)*. The work of all of these authors was done mostly in the 1982–1983 time period. In 1983 Woods *(5)* predicted that "all the major automobile manufacturers" would introduce vehicles in the 1,000 to 1,500 lb weight range by the 1985 or 1986 model years. Projections of the market share of these vehicles ranged from 6 to 9 percent *(16)* to as high as 60 percent *(15)*.

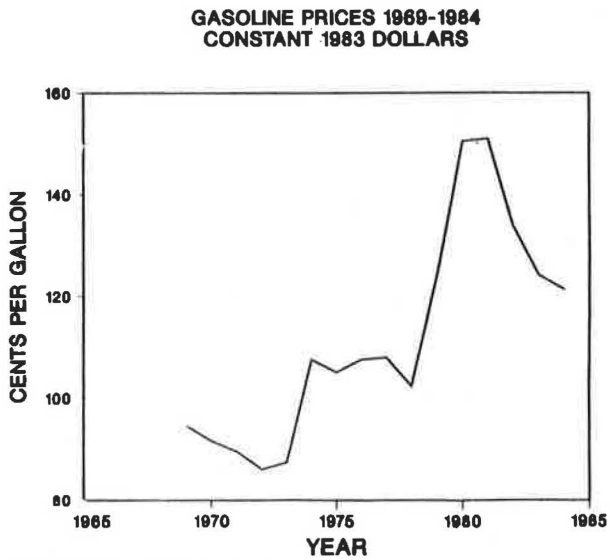

FIGURE 3 U.S. fuel prices (18).

Why Have Opinions Changed?

Current projections represent a significant change from earlier thinking. It is appropriate to examine possible reasons for this near turnabout because these same reasons may dominate longer-term projections.

One reason, of course, is that the marketplace has already responded differently than projected. The public has not demanded the small cars, and manufacturers' downsizing was not as dramatic as many had expected. The Japanese, faced with "voluntary" import quotas to the United States, have concentrated on their larger, more luxurious and profitable models.

Hemphill *(16)* reported that the three major criteria people used in choosing a vehicle in 1980 were, in order, (a) fuel economy, (b) low purchase price, and (c) quality and dependability. In 1981 fuel economy had dropped to third on the list. Greene et al. *(18)* reported the dramatic changes in fuel prices (see Figure 3). Since their report, world crude prices have been cut in half, and gasoline was selling in early 1986 for prices at or below those of the early 1970s.

Technology advances in the industry have resulted in improved fuel economy independent of vehicle downsizing *(19–21)*. The biggest technological impact has been the rapid growth in the use of electronics and microcomputers. Altshuler and Roos *(19)* point out that microprocessors were added to engines beginning in the mid-1970s, to transmissions in the early 1980s, and are now available for some suspension systems. Further, the use of microprocessors and electronic controls can significantly reduce the demands of auxiliary systems on the power system. A current example is the electric radiator fan, which has become nearly universal and operates only when auxiliary cooling is required. Microprocessor-controlled power steering is available in some Japanese models; it provides maximum assist in very low speeds, such as when parking, and very little assist at highway speeds. Mercedes Benz and some top-of-the-line Ford products now offer microprocessor-controlled antiskid power brakes *(22)*. Most other auxiliary systems are likely to come under computer control in the future.

Other technological improvements discussed by Altshuler and Roos *(19)* include engine improvements such as the four-valve-per-cylinder engine, turbocharging, and electronic fuel injection, which is now becoming quite common *(21)*. Advanced research and development is presently underway on adiabatic engines with ceramic liners and other ceramic parts. Such engines can run at much higher temperatures, and thus convert a greater proportion of the heat produced into useful energy. In fact, such engines would not need a cooling system. Continuously variable transmissions, when coupled with microprocessor control and advanced materials, will enable the engine to always run at its optimum speed, regardless of vehicle speed. Carbon fiber composites provide strength-to-weight ratios far in excess of metals. Although not yet generally cost-competitive with metals, composites are expected to be used increasingly. For example, the Chevrolet-Corvette uses composite materials in its springs.

Another major advance is in improved aerodynamics. The drag coefficient (C_d) of the average vehicle on the road is about 0.5. The average C_d of vehicles presently being marketed is about 0.4. Some production vehicles (e.g., the Audi 100) have C_d values as low as 0.3. There are prototypes as low as 0.15, and researchers are hoping to achieve values below 0.1. Within 20 years, it is projected that the average new automobile will have a C_d of 0.2 or less *(18)*, which, when compared with the vehicles now being marketed, would have up to 25 percent better fuel economy at highway cruising speeds because of that factor alone.

In summary, fuel economy is no longer the pressing issue it was in the mid- and late-1970s when most projections were made. Federal fuel economy standards have been relaxed. Rapid technology advances have enabled fuel economy savings beyond that offered by downsizing. The purchaser can obtain reasonable fuel economy without buying a small car. Moreover, manufacturers now push the more expensive (and profitable) larger cars at a sacrifice in fuel economy.

Current Data and Revised Projections

Recent data show that the earlier predictions are not proving accurate—they greatly overestimate the amount or rate of change that would take place in vehicle characteristics. Taylor *(21)* noted that the average weight of American automobiles for model year 1980 was about 3,200 lb, and that has not changed appreciably in the 4 years hence. The federally mandated CAFE of 27.5 mpg was not met in 1984 and was relaxed to 26.5 mpg in 1985. The sales of domestic minicompacts declined from nearly 5 million in 1978 to zero in 1982 (see Figure 4). Sales of domestic subcompacts have declined appreciably from 1980 onward. The growth has been in sales of the compact-sized vehicles and, to some extent, in large vehicles. A similar story is true for imported vehicles (Figure 5).

The University of Michigan has conducted biannual Delphi surveys of automobile industry forecasts of more than 100 automotive industry experts *(23)*. Examples of how drastically opinions have changed are given in Table 2, and predictions for 1990 model year vehicles are compared.

The preceding discussion focuses on automobiles. However, the purchaser of a vehicle for personal transportation has other options, such as pickup trucks, vans, and special purpose vehicles (e.g., "jeeps"), which, collectively, are termed "light trucks" by NHTSA. In 1984 the latter accounted for 25.8 percent of all light-duty vehicles sold, the result of a fairly consistent increase from 20.7 percent in 1978 *(24)*.

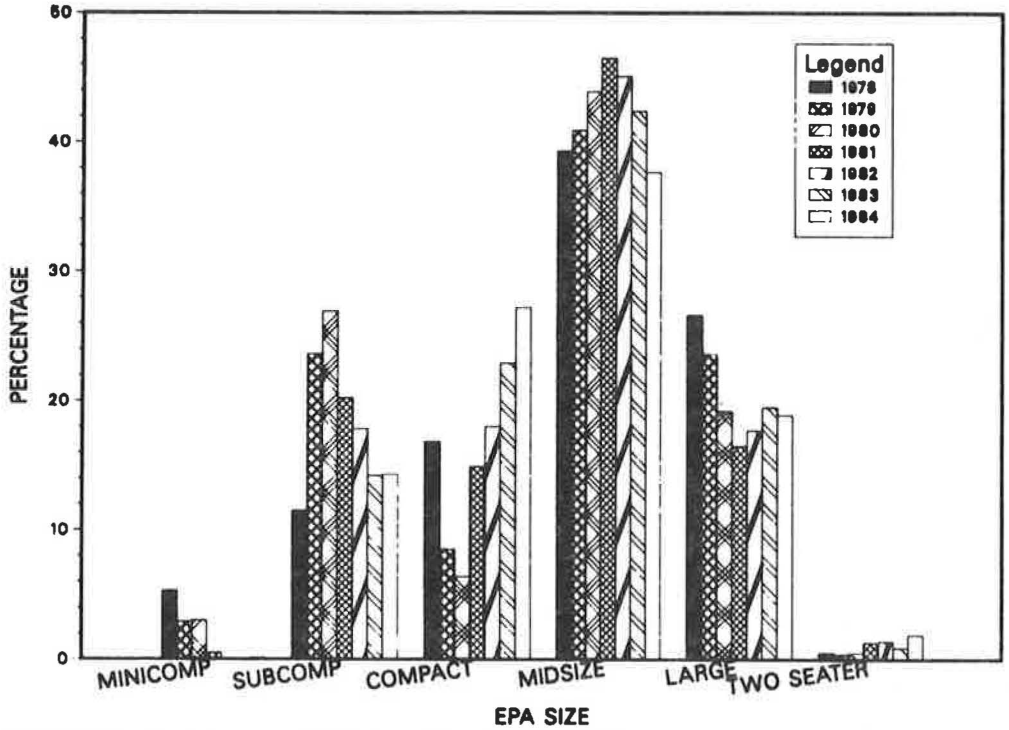

FIGURE 4 Market shares of U.S. domestic automobiles (24).

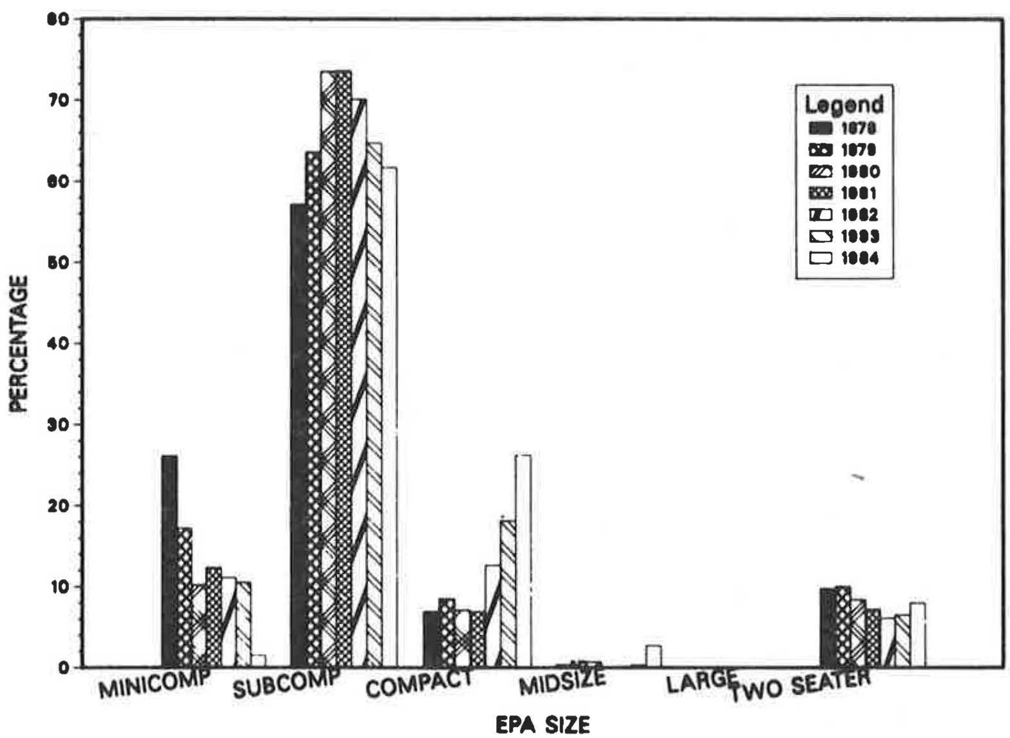

FIGURE 5 Market shares of U.S. import automobiles (24).

TABLE 2 Automotive Industry Opinions

Topic	Year of Survey 1979	1981	1983
Price of gasoline in 1990 (1983 $/gal)	3.45	3.30	1.60
1990 fuel economy (mpg)			
Domestic	30	35	30
Japan	NA	36–39	32
1990 median vehicle weight (lb)	2,515	2,250	2,419
Percent of 1990 domestic sales accounted for by			
8-cylinder engines	5	3	10
6-cylinder engines	25	20	35
Diesel engines	25	20	10
Full size plus intermediates	30	24	36
Mini-subcompacts plus commuters	NA	15	4

NOTE: NA = not applicable.

Revised projections of most other vehicle characteristics have not appeared as yet in the literature. It is clear that the process of downsizing, and possible future "upsizing," involves a number of characteristics such as length, width, weight, interior volume, and the like. Moreover, the correlations between these characteristics are not perfect *(25)*. Nevertheless, it is likely that the authors who overestimated the future decrease in vehicle weight, for example, also overestimated the decrease in length, width, and the like.

The one vehicle-related characteristic that has been addressed by highway safety experts more than any other, aside from weight, is driver eye height. Recent studies by Farber *(26)*, Khasnabis et al. *(27)*, Olson et al. *(28)*, and Weaver et al. *(29)* have been reported. They generally agreed that the lower bound on driver eye heights has not changed appreciably for several years and is not likely to change in the future.

The Longer-Term Outlook

It is likely that the basic characteristics of size and weight will not change drastically in the next 15 years.

At the end of 1982, the world's proven petroleum reserves represented a 34-year supply at current production rates, the same as it was in 1969–1971. Moreover, the fraction of the petroleum reserves available for automobiles will increase dramatically in the years ahead. In the last 10 years, there have been substantial movements toward alternative energy sources for space heating, electricity generation, industrial process energy, and agriculture. Vehicles are much more fuel-efficient now than 10 years ago (twice as efficient in the United States). For all of these reasons, there should be no long-term shortage of gasoline in the next few decades.

Speed limits are likely to increase, at least on rural Interstates *(30)*. As a result, there will be some increase in demand for more power and comfort, further suggesting a decline in the downsizing mode, and probably a return to larger cars.

The vehicles now being sold will still be in use in large numbers by the year 2000. The life cycle of an automotive design is on the order of 25 years *(19)*. It takes approximately 5 years to bring a new design to market; it then typically continues in production for 6 to 8 years. The vehicles continue to be driven for 12 or more years.

APPLICATION OF RESULTS TO RRR PROJECTS

Trucks

1. *Low-speed off-tracking.* Increased off-tracking is experienced by longer vehicles—especially those with large spans between successive axles. Of the trucks expected to be frequently encountered in the near future, the 48-ft semitrailer is of greatest concern. Redesign of intersections and widening of sharp curves may be required to eliminate encroachment on the opposing or adjacent lane, on curbs or medians, or on the shoulder.

As an illustration of the effect of configuration on the amount of off-tracking, consider the simple case of a constant radius curve *(31)*. The data in Table 3 show the amount of off-tracking (i.e., the offset between the paths followed by the front wheels and the rear wheels) for a number of configurations on a 200-ft radius curve. The 48-ft semitrailer would encroach on either the shoulder or the adjacent lane by more than 1 ft; the fairly common 53-ft semitrailer would encroach by nearly 2.5 ft. In this case, the lane may require a greater width at this location to accommodate such trucks. (Note that the twin trailer combination off-tracks substantially less.)

TABLE 3 Illustrative Off-Tracking Amounts

Configuration	Offtracking (ft)	Lane Width[a] (ft)
17-ft car	0.25	–
30-ft single unit truck	1.00	9.50
40-ft bus	1.57	10.07
Tractor/40-ft semitrailer	3.40	11.90
Tractor/48-ft semitrailer	4.87	13.37
Tractor/53-ft semitrailer	5.96	14.46
Tractor/twin 28-ft trailers	2.54	11.04

[a]Required to accommodate an 8.5-ft-wide vehicle.

2. *High-speed off-tracking.* This phenomenon requires higher speeds *(32)*, and could be a problem on curves or ramps. The off-tracking magnitudes are usually not large, but are greater for multiple-unit vehicles. It could lead to overturn of the rear trailer if the rear wheels contact an obstacle such as a curb.

3. *Vehicle width.* Research studies have found no significant problems induced by the added 6 in. in width. Lane width suitable for 96-in. widths are generally also acceptable for 102 in. Off-tracking will be greater (by 6 in.), but this is usually a small fraction of the total off-tracking problem.

4. *Vehicle weight.* Collisions between automobiles and heavy trucks have always been a concern, and will continue to be so. The allowable weight increase from 73,280 to 80,000 lb is not of great concern as this is only a small increment over the already great differential with, for example, a 3,000-lb automobile. However, as truck volumes increase, the potential for automobile–truck collisions becomes greater. Also, there is concern about the ability of heavier trucks to maintain speed on grades. The resulting speed differentials between vehicles suggest a higher accident probability at such locations.

Roadside hardware is usually designed for automobile impacts. It generally will not redirect 73,280-lb vehicles, let alone 80,000-lb vehicles. This fact, coupled with increased truck volumes, suggests that substantial work is needed to design and install hardware that will safely accommodate both trucks and light automobiles.

5. *Rearward amplification.* In a severe lateral movement, such as in an evasive lane change, the second trailer in a doubles combination will be subjected to an amplified lateral acceleration and displacement *(33, 34)*. If severe enough, it could lead to rollover of the rear trailer. It is not clear how highway design can affect this phenomena—it is more a question of vehicle design.

6. *Braking.* Properly designed and adjusted brakes provide comparable stopping distances for the newer configurations so this should not be considered a new issue *(34)*.

7. *Overall safety of doubles.* It is unlikely that twin trailer combinations will be found to be appreciably less safe than semitrailers *(34)*.

Automobiles

Despite a great diversity of opinion concerning future vehicle characteristics, most researchers do not anticipate the need to revise highway design standards to any significant degree *(4, 17)*. Those areas or standards that have been studied are briefly examined next.

1. *Lane width.* A few authors *(5, 17)* have suggested that narrower lane widths would be acceptable, based on present and projected automobile widths. However, they also point out that if trucks and buses are allowed to use these lanes, no changes in the standards should be made.

2. *Vehicle length.* Even though present and future automobiles are somewhat shorter, no changes in standards are recommended because they are so weakly dependent on vehicle length *(4)*.

3. *Driver eye height.* The present design height of 42 in. could be reduced to 39 or 40 in. However, this change would have minimal effect on sight distance *(26, 27)*.

4. *Underclearance.* Several authors *(4, 5, 28)* have noted that underclearances of 4 in. are not uncommon. McGee et al. *(4)* report that the median underclearances for 1983 automobiles were about 4.9 in. (domestic) and 5.2 in. (foreign); about 10 percent of domestic and foreign vehicles had underclearances of 4 in. or less. If the present design object height of 6 in. is based on underclearance, it should be reduced.

5. *Stopping sight distance.* In addition to driver eye height and object height, stopping sight distance depends on stopping ability. There is some criticism of present AASHTO standards relative to stopping ability *(4, 28)*, claiming that real drivers may require more distance than the standard assumes. This has nothing to do with changes in vehicle characteristics. However, to the extent that future vehicles use antiskid brakes, stopping distances should decrease, perhaps counterbalancing these criticisms. (See also Item 9.)

6. *W-beam guardrail.* Smaller automobiles tend to have lower bumper heights, and some may have a tendency to submarine under W-beams set at the present standard height of 27 in. *(5, 17)*. On the other hand, if the height were lowered, some larger vehicles may be prone to vaulting the barriers *(35)*. It appears that further research is required in this area.

7. *Sideslopes.* Burtch et al. *(36)* found that smaller vehicles are not more likely to overturn on sideslopes than larger vehicles; Woods *(5)* suggested otherwise. Further research may be required in this area, also.

8. *Roadside hardware.* The safety issue that may be most affected by smaller (lighter) vehicles is roadside hardware or fixed-object collisions *(5, 12, 17, 36).* Present design standards for utility and luminaire poles are probably inadequate for vehicles weighing substantially less than 2,000 lb—for example, microvehicles in the 1,200 to 1,600 lb range. Nevertheless, such vehicles are not presently anticipated to be a significant fraction of the highway traffic mix. Sign supports, on the other hand, may not be designed appropriately for any vehicles under 2,000 lb *(5).* Because several present vehicles weigh less than this amount, this design standard might reasonably be reviewed.

9. *Automobile braking ability.* As an increasing number of automobiles use antiskid braking systems—a development just now beginning with certain Mercedes and Ford products—stopping distances will decrease, especially on wet pavements. This could affect design standards such as stopping sight distance. However, the effect on safety is likely to be mixed. If all vehicles had such brakes, many current accidents would be converted to "near misses" or accidents of less severity. On the other hand, if some vehicles have substantially shorter stopping capabilities than others, an increase in rear-end accidents would be expected.

ACKNOWLEDGMENTS

The author would like to acknowledge the thoughtful suggestions of one of the reviewers. A valuable reference on some of the truck issues is *Transportation Research Record 1052: Symposium on Geometric Design for Large Trucks,* 1986. The symposium was held in August 1985. The Record contains a number of very good papers, although much of the material was available to the author previously, in less convenient form. Unfortunately, the symposium occurred about 6 months after this paper was prepared. It is also noteworthy that NCHRP Project 22-6, which deals with small vehicle issues, was initiated in mid- to late-1985. The interested reader should anticipate the completion of that work, presently scheduled for the spring of 1988.

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