SYNTHESIS OF SAFETY RESEARCH RELATED TO TRAFFIC CONTROL AND ROADWAY ELEMENTS VOL. 1

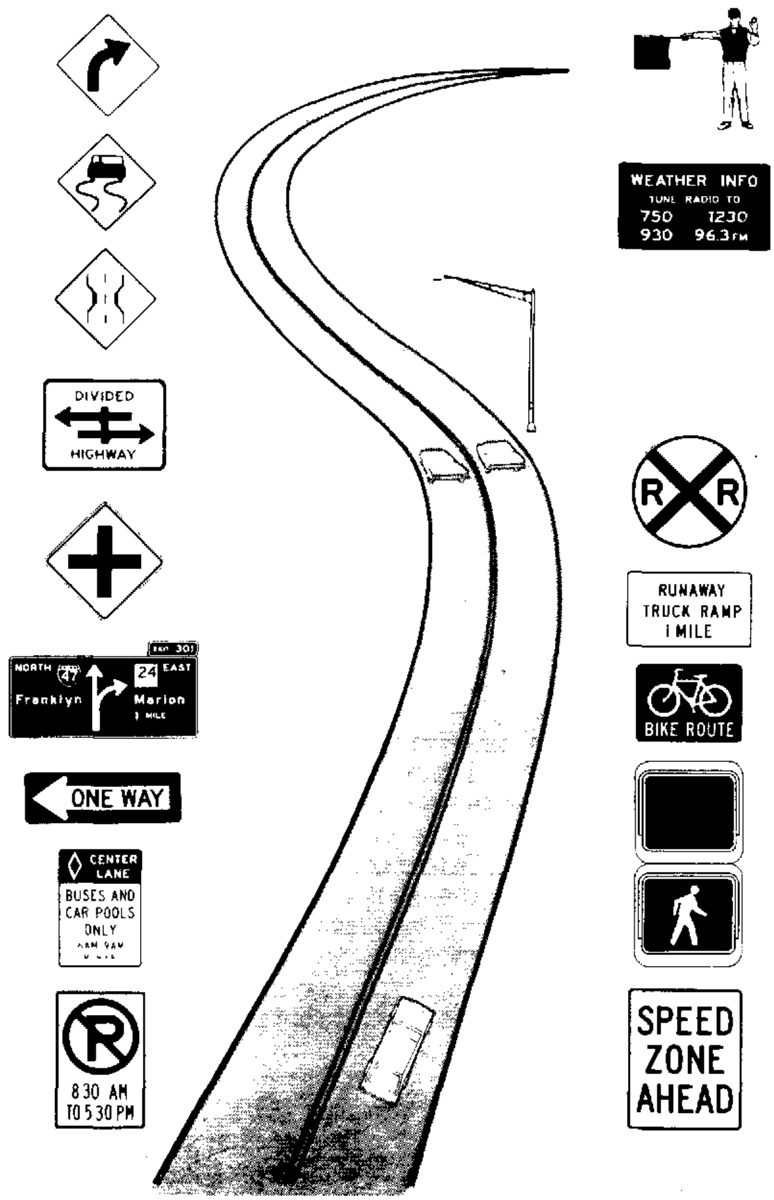
Offices of Research,
Development, and
Technology
Washington, D.C. 20590

FHWA-TS-82-232

DECEMBER 1982

U.S. Department
of Transportation

Federal Highway
Administration



FOREWORD

This "Synthesis of Safety Research Related to Traffic Control and Roadway Elements" is the second update of "Traffic Control and Roadway Elements - Their Relationship to Highway Safety." The original synthesis was published by the Automotive Safety Foundation in 1963. The first update was published as a series of 12 individually printed chapters. These chapters were issued beginning in 1968 by the Automotive Safety Foundation and continued through 1971 by its successor, the Highway Users' Federation for Safety and Mobility. This second update is published in two volumes by the Federal Highway Administration (FHWA).

The development of this updated safety synthesis was initiated with a contract with Texas A&M University, Texas Transportation Institute. In this contract, extensive literature reviews were made and 17 draft chapters prepared, each covering a specific subject area. Staff from many FHWA Offices participated in an extensive review and revision to each draft chapter; the final editing and report production was accomplished by the FHWA Offices of Research, Development, and Technology.

The emphasis for this safety synthesis is to present safety research results reported since the previous update was published, and other older research results as appropriate. The synthesis provides public officials, highway administrators, engineers, and researchers with factual research findings on safety effects of specific design and control features to guide and support engineering decisions. Findings are reported objectively based on the contributing authors' critical analyses of pertinent research studies. Reference lists are provided for those needing more detailed information on the subjects cited.

Stanley R. Byington
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Traffic Operations Research and
Development

NOTICE

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA-TS-82-232		PB85 199297/AS

4. Title and Subtitle
Synthesis of Safety Research Related to Traffic Control and Roadway Elements - Volume 1

5. Report Date
December 1982

6. Performing Organization Code
HSR-30

7. Author(s)
See individual chapters for principal contributors

8. Performing Organization Report No.

9. Performing Organization Name and Address
Federal Highway Administration
Off. of Saf.& Traf.Oper. R&D
Tra. Contr. & Oper. Div.
Washington, D.C. 20590

Texas A&M University
Texas Trans. Inst.
College Station, TX
77843

10. Work Unit No. (TRAIS)
31A3-044

11. Contract or Grant No.
DOT-FH-11-9625

12. Sponsoring Agency Name and Address
Federal Highway Administration
Office of Safety and Traffic Operations R&D
Traffic Control and Operations Division
Washington, D.C. 20590

13. Type of Report and Period Covered
Final Report
TTI - Aug.1979 to June 1981
FWHA-Apr. 1981 to Oct. 1982

14. Sponsoring Agency Code
T-0493

15. Supplementary Notes
Mr. Merton J. Rosenbaum of the FHWA, Traffic Control and Operations Division, HSR-30, provided the prime direction of the synthesis and Dr. Charles Pinnell was the principal investigator for TTI. Mr. Willard J. Kemper was in charge of report production.

16. Abstract
This synthesis is published in two volumes. Each of the 17 safety research subject areas is presented as an individual chapter. Subject areas included in Volume 1 are: roadway cross section and alinement; pavement surfaces; roadside features; access control and driveways; intersections; interchanges; one-way streets and reversible lanes; priority for high occupancy vehicles; and on-street parking.

Volume 2 subject areas include: construction and maintenance zones; adverse environmental operations; roadway lighting; railroad-highway grade crossings; commercial vehicles; bicycle ways; pedestrian ways; and speed zoning and control.

An overall 17-chapter subject index is included in both volumes of the synthesis for finding specific areas of interest.

17. Key Words
Cross Section, Alinement, Pavement, Roadside Features, Access Control, Driveways, Intersections, Interchanges, One-Way Street, Reversible lanes, High Occupancy Vehicles, On-Street Parking

18. Distribution Statement
No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	193	

METRIC CONVERSION FACTORS

APPROXIMATE CONVERSIONS FROM METRIC MEASURES

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
in	inches	2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
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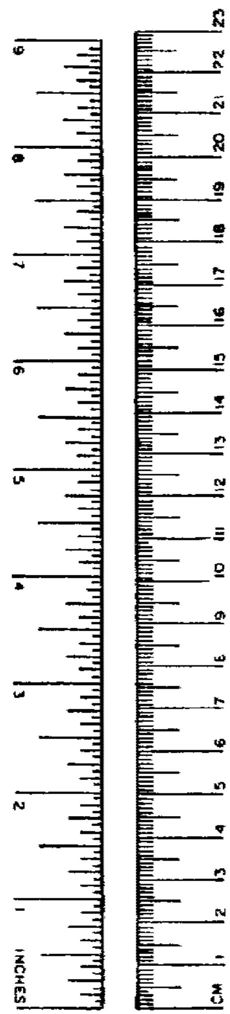
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C



APPROXIMATE CONVERSIONS FROM METRIC MEASURES

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		LENGTH		
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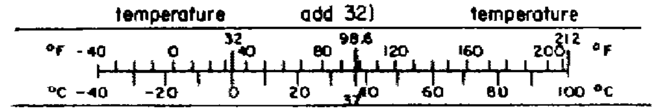
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	36	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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CHAPTER 1 - ROADWAY CROSS SECTION AND ALINEMENT

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INTRODUCTION

The fundamentals of highway design include the selection of physical dimensions that will appropriately serve the expected flow of traffic with consideration to the ground topography along the designated route. The basic geometric dimensions involve the roadway cross section and the alinement. Roadway cross section includes the width of the traveled way (pavement), the width of shoulders, the width and shape of median sections if the highway is divided, the cross slope of the pavement and the slope of embankments, and types of draining ditches.

The roadway alinement along the selected route includes: straight sections (tangents), horizontal and vertical curves, and roadway grades.

Research has been conducted to evaluate the safety of these roadway elements by analyzing accident data on existing roadways.

The American Association of State Highway and Transportation Officials (AASHTO) has set geometric standards for various roadway types. These standards are presented in such publications as A Policy on Geometric Design of Rural Highways-1965 (1) and A Policy on Design of Urban Highways and Arterial Streets-1973 (2). Some of the standards set by AASHTO are based on past research while others are from rational analyses and expert opinions. This chapter summarizes the relationships between safety, as measured by accidents, and the geometric cross section and roadway alinement elements.

When the roadway alinement or changes in the cross section provides some type of hazard to the motorist, traffic control devices are installed to warn drivers of possible hazards and/or to modify the vehicles' operation. The standards for these traffic control devices are presented in the Manual on Uniform Traffic Control Devices for Streets and Highways (MUTCD) (3), published by the Federal Highway Administration (FHWA) in 1978. Research has been conducted on some of these traffic control devices to relate their effects on accident experience and/or vehicle operations.

CROSS SECTION

The cross section of a roadway is made up of the traveled lanes (pavements), shoulders, medians, and roadside slopes and ditches. The cross section can be represented as if a slice were made across the roadway, perpendicular to its alinement so that a profile is shown along the cut-line. See Figure 1.

The widths and slopes of component portions of the cross section affect vehicle operations and safety. Studies have been conducted relating to the different component elements of the cross section as well as combinations of the various elements.

Five recent studies have been conducted to determine the relationship of cross section features with accident data.

1. Dart and Mann (4) reported in 1970 on the research conducted in Louisiana to determine accident rates related to rural highway geometry. They collected accident data for 246 sections of rural roadway where sections varied from 1 to 17 miles. They reviewed over 6,000 accident reports from 1962 to 1966.

2. Foody and Long (5) reported their accident study for Ohio in 1974. From the 13,962 miles of two-lane State highways a representative sample of 1,400 miles was studied in detail. Traffic accident records from 1969 and 1970 were used to identify the single vehicle accident for the sample. The effects of roadway width and shoulder widths on accident rates were determined as well as studying the interaction of shoulder quality and the recovery area.

3. In Jorgensen's (6) NCHRP Project 3-25, reported in 1978, the data bases from the States of Maryland and Washington were combined to study the safety effect of various highway geometric features. The data base included 1,937 miles of two-lane rural roads in Maryland and 2,010 miles in Washington. The combined number of study sections was over 34,000 for the two States.

4. Zeeger, Deen and Mayes (7) reported in 1980. Their analyses of accidents related to lane and shoulder widths for two-lane rural roads in Kentucky. They sampled 15,994 1-mile sections and 16,760 accidents reported in 1976. They controlled for classification variables and eliminated non-homogeneous sections.

5. The Arizona Department of Transportation (8) evaluated shoulder improvements in a 1978 report. Shoulder improvements on 111.8 miles of two-lane highways were done by maintenance forces in 1974 and 1975. Accidents for one year before totaled 98 and 1 year after the improvement totaled 92. The accident rate decreased from 1.81 accidents per million vehicle miles (A/MVM) to 1.30. Run-off-the-road accident rates went from 0.81 A/MVM to 0.58. For 74 miles of pavement widening projects done in 1974-1976, the accident rates fell from 1.89 A/MVM to 1.14 and the run-off-the-road accidents went from 0.94 to 0.49 A/MVM.

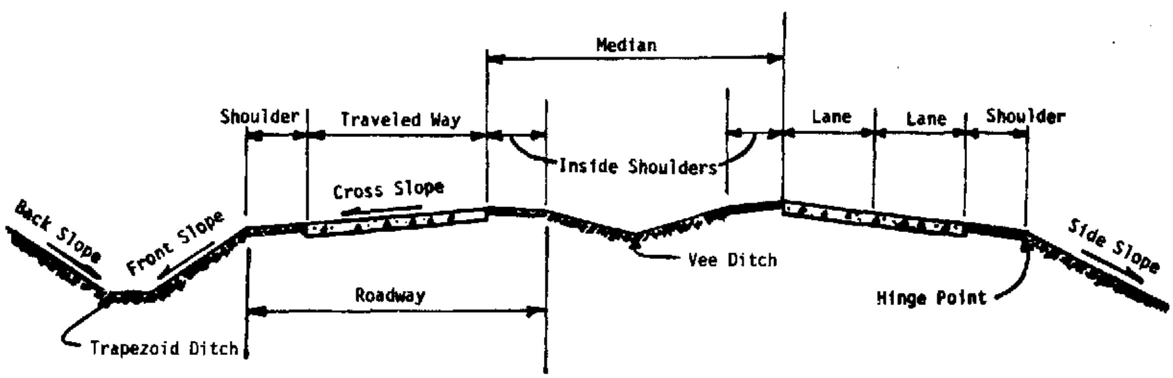


Figure 1. Typical Cross-Section of a Divided Highway

Lane Width

Research studies have generally shown that the accident rates decrease with an increase in the width of the traffic lane (4, 5, 6, 7, 8). The information in Figure 2 is for two-lane rural highways. Zegeer et al. concluded that widening a lane beyond 11 feet is not cost effective (7).

The Alabama Highway Department (9) studied the effects of widening lanes on two-lane rural highways. They had 17 sites where lanes were widened from 9- and 10-foot lanes to 11- and 12-foot lanes. They also had control sections and parallel study sections. The results showed a lane width increase reduced the injury-fatality accident rate significantly (22%) and caused a decrease in the total accident rate.

Most high volume roadway facilities were built with standard 12-foot lanes. However, a number of freeway sections were becoming congested during peak traffic periods. In order to increase the capacity through the congested bottleneck section, the pavements were restriped to add another lane by using narrower lanes and reducing shoulder widths. Table 1 shows the safety results of a number of freeway sections that were modified to increase capacity (10).

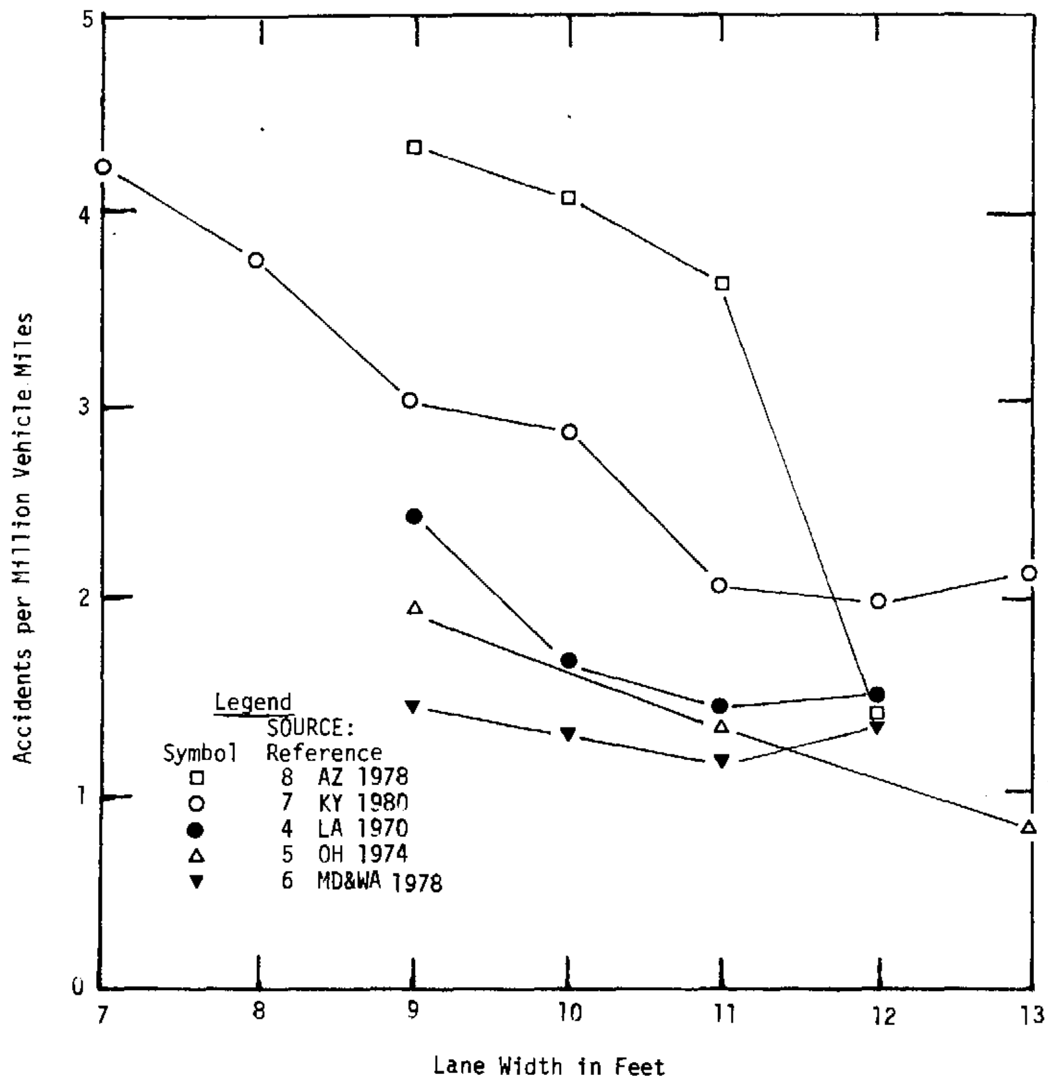


Figure 2. Accident Rate Based on Lane Width for Two-Lane Rural Roads

TABLE 1 - Increased Capacity Through Freeway Lane Width Reduction

Location City & Facility	Length Miles	Before Cross Section Lt.Sh.	Before Cross Section Lanes	Before Cross Section Rt.Sh.	After Cross Section Lt.Sh.	After Cross Section Lanes	After Cross Section Rt.Sh.	Accident Rate Accidents/MVM Before	Accident Rate Accidents/MVM After
L.A.,CA I-5	2.63	6'	3-12'	8'	6'	4-11'	0'	1.38	0.90
L.A.,CA I-5	7.6	8'	5-12'	8'	8'	6-11'	2'	1.73	1.47
L.A.,CA I-5	2.2	5'	4-12'	8'	3'	1-12',4-11'	0'	2.39	2.17
L.A.,CA. I-10	1.4	11'	4-12'	10'	11'	5-11'	3'	1.45	1.25
	0.9	11'	5-12'	8'	11'	6-11'	2'		
L.A.,CA I-10	.28	8'	4-12'	8'	8'	5-12'	0'	3.4	3.0
L.A.,CA LA-60	1.7	8'	4-12'	8'	8'	1-12', 4-11'	0'	1.18	1.13
LA-60	2.8	11'	4-12'	8'	3'	5-11'	8'	1.41	1.03
US 101	3.0	8'	4-12'	8'	2'	5-11'	10'	1.86	0.78
Houston,TX US 59	3.1	10'	3-12'	10'	10'	4-10.5'	4'	3.68	2.90
		10'	4-12'	10'	10'	5-10.5'	5.5'		
Nashville,TN. I-65,265	1.0	6'	3-12'	10'	4	4-11'	4'	1.91	0.91
E. Hartford,CT I-84,86	2.6	4'	2-12'	10'	2'	3-11'	3'		9.58
Welhersfield,CT,I-91	0.5	6'	2-12'	10'	2'	3-12'	2'		0.31
W. Hartford,CT. I-84	0.3	4'	2-12'	10'	1'	3-12'	1'		2.05
Portland, OR. I-5	0.2	0'	2-12'	8'	0'	2-11 1/4; 1-9.5'	0'	4.69	1.30

SOURCE: Reference 10

Shoulder Width

Research on the width of shoulders related to traffic accident rates has had mixed results. Early research had indicated that accidents increase with increasing shoulder width. In 1954, Belmont (11, 12) in California tested three ranges of shoulder widths against total accident frequency which included 1,300 accidents on 533 miles of roadway. He concluded that accident rates were significantly lower with paved shoulders of 6 feet than with wider pavement shoulders. Blensley and Head (13, 12) in 1960 in Oregon studied 346 miles of rural two-lane tangents and concluded accident frequency increased with increasing shoulder widths for all volume ranges studied.

More recent studies have generally shown that the accident rates have been reduced as shoulder widths increase (5, 6, 7). These results are shown in Figure 3.

Zegeer et al. (7, 12) conducted an economic analysis of shoulder widening which showed the benefit-cost ratio is more then one for widening narrow shoulders on sections of two-lane rural roads having six or more run-off-the-road and head-on accidents per mile per year. Shoulder widening would not be cost effective for low-volume roads with less than 1,000 vehicles per day having low accident frequencies. No additional benefit is obtained on rural, two-lane roads by widening shoulders to more than 9 feet. Accident reductions from the Zegeer study for shoulder widening are shown in Table 2. Higher priority should be given to shoulder widening on horizontal curves and winding sections than to straight level tangents.

TABLE 2 - Reduction in Accident Rates from Shoulder Widening on Two-Lane, Rural Roads

Shoulder Width in Feet Before	Shoulder Width in Feet After	Reduction in Run-off-Road & Opposite Direction Accidents in %
None	1-3	6
None	4-6	15
None	6-9	21
1-3	4-6	10
1-3	7-9	16
4-6	7-9	8

SOURCE: Reference 7

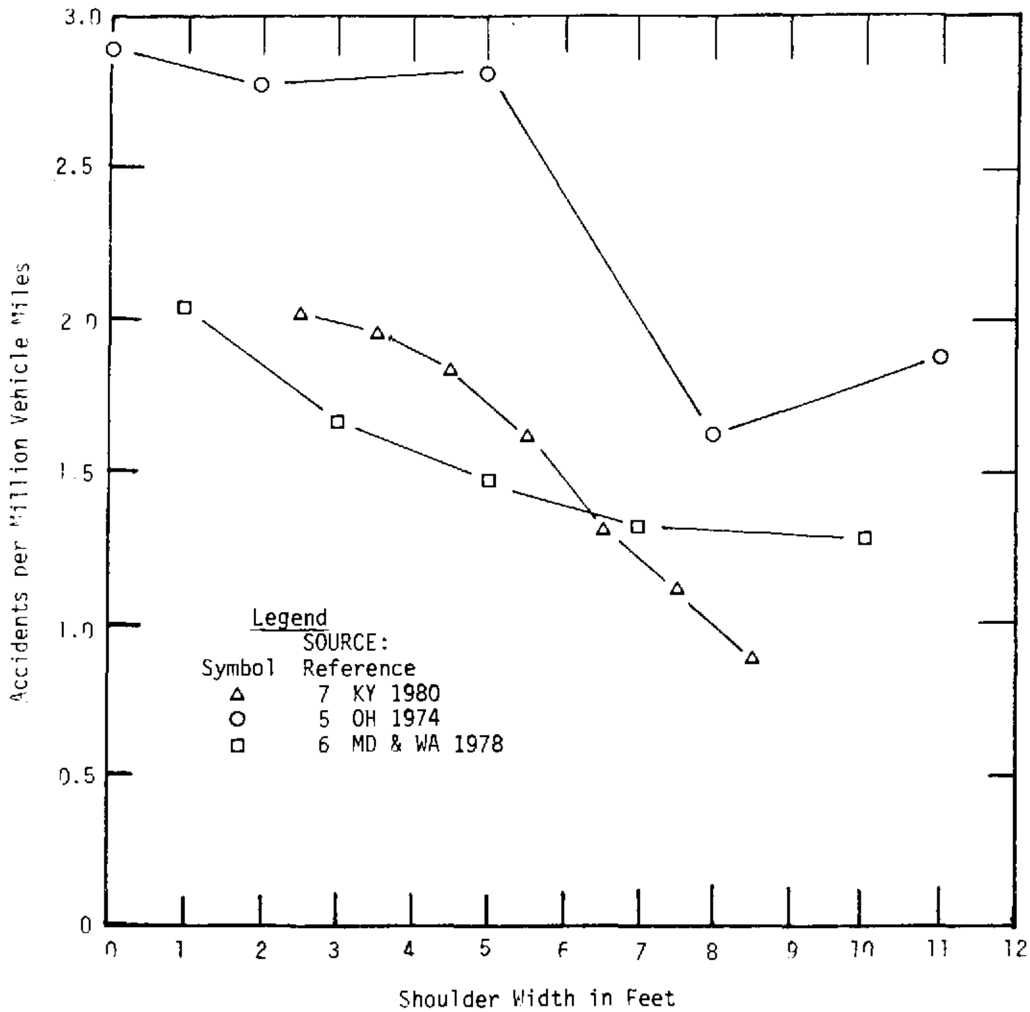


Figure 3. Accident Rate Based on Shoulder Width for Two-Lane Rural Roads

SHOULDER SURFACE TREATMENT

Foody and Long (5) concluded in their 1974 Ohio study that it is more cost effective for Ohio to improve (stabilize) two-lane rural road shoulders than to widen the pavement or to clear the roadside. This was based on their analysis that unstabilized shoulders have 30 to 50 percent higher accident rates than stabilized shoulders and the costs to stabilize shoulders are much less than pavement widening and/or roadside clearance.

Heimbach, Hunter, and Chao (14) studied the accident experience in North Carolina for paved versus unpaved shoulders and reported their results in 1974. Based on a sample of 3,054 individual roadway sections (including two-, four-, and six-lane highways) a significantly lower accident experience (about 20%) was observed for all types of highways with paved 3- to 4-foot shoulders as compared to identical highways with unpaved shoulders.

Arizona DOT (8) evaluated a 6-inch wide (3/4-inch deep) 45° diagonal grooving of the right shoulder at 100-foot intervals on a 10-mile section of Interstate 8. They found that run-off-the-road accidents were reduced 61%. The cost of shoulder grooving the 10 mile section was $2,000. They estimated the grooves saved 13 single vehicle accidents in 3 years. Considering a 10-year life of the grooving, the benefit-cost ratio for the project was 108. A similar study was conducted to evaluate painted diagonal striping of the shoulder. The painted shoulder did not lower the accident rate.

Roadway Width

The roadway consists of both the traveled way and the shoulders on two-lane, two-way roads. Motorists generally consider that the right half of the roadway is for use by vehicles traveling in one direction. The right lane is for the movement of vehicles and the shoulder is for stopped vehicles. The combination of the traveled way and the shoulder affect the safety of a roadway facility. Research results from several studies were summarized by Leisch (15) in a figure recommending roadway widths for two-lane rural highways in Minnesota. Figure 4 from Leisch's report relates total shoulder width and pavement widths to expected accident rates.

In 1981, Frambro et al. (16) reported the safety of three different types of rural highways in Texas. These highway types were two-lane without paved shoulders, two-lane with paved shoulders, and undivided four-lane without paved shoulders. Both a comparative analysis and a before and after technique were used to determine the safety benefits. The results are shown in Figure 5.

Accident data were collected from 1975 through 1977 for 85 sites. A total of 16,334 accidents were included as the study base, 8,815 of these accidents were non-intersection accidents. Based on this study, the researchers concluded that two-lane roads with shoulders were safer than four-lane roadways without shoulders (termed "poor boys"). They note that when paved shoulders are converted to travel lanes, the immediate recovery zone is removed and fixed objects are nearer the traveled lane. If safety is a major consideration, they say consideration should only be given to using "poor boy" highways for ADT's above 7,500.

Fisher (17) reported in 1977 on street widening in Los Angeles. For 40 projects consisting of 31.5 miles of arterial street widening, the analysis of 4,035 reported accidents showed that total accidents were reduced by 21% and injury plus fatal accidents were reduced by 22%. The primary purpose for the street widening was to increase capacity. Most streets were widened to have two lanes in each direction with continuous median or left turn channelization with parking allowed. Roadway "Jut-Out" sections were eliminated to provide standard widths on 13 major secondary streets. (A "Jut-Out" is an abrupt change in the pavement width across a few lots along a street.) The accident analysis consisted of counting the number of accidents for 24 months before and 24 months after the widening. The analysis was based on the actual number of accidents and not the accident rates even though traffic volumes increased during the after periods because of the increased capacity due to widening.

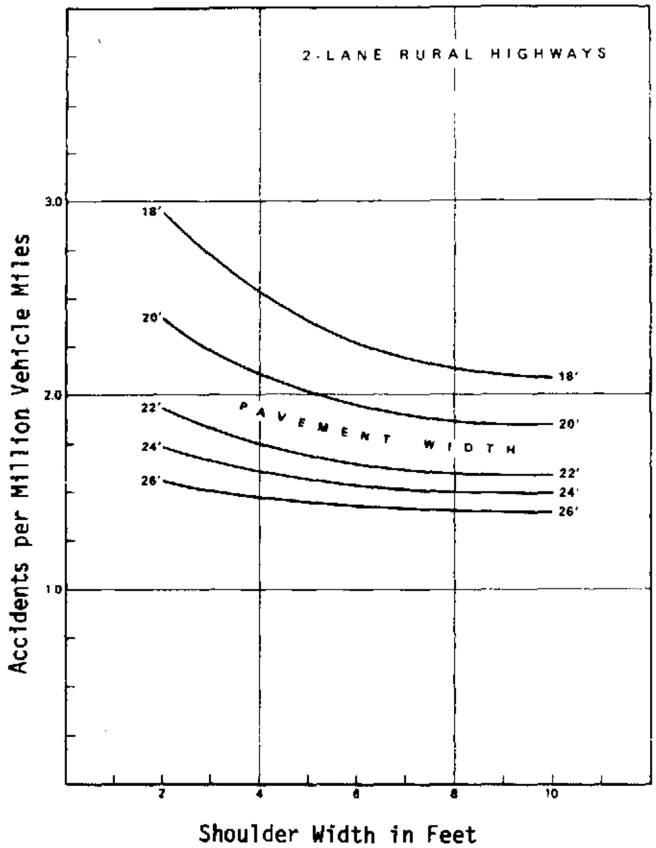


Figure 4. Accident Rate for Variable Pavement and Shoulder Widths on Two-Lane Rural Highways

SOURCE: Reference 15

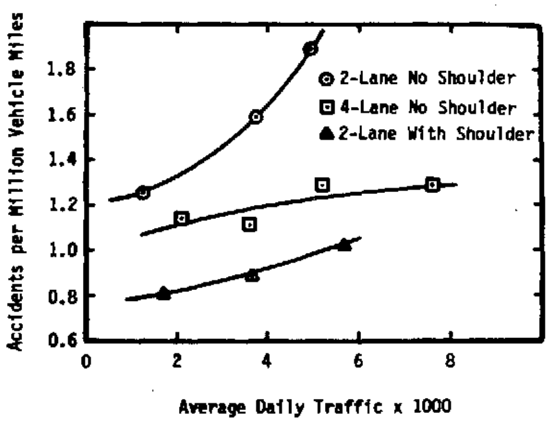


Figure 5. Accident Rates for Different Classes of Texas Highways

SOURCE: Reference 16

Bridge Width

Jorgensen-Westat (18) in 1966 indicated safety on bridges has been shown to be related to the width of the bridge and the width of the roadway approach (travelway plus the shoulder). Table 3 provides a comparison of accident rates and the related bridge and roadway width differences.

From information contained in Table 3, McFarland (19) in 1979 presented the expected effectiveness from bridge widening in Figure 6.

Research reported in 1976 by Woods, Bohuslav and Keese (20) showed remedial treatment on the approach to more than 50 narrow bridges (26-feet wide or less) reduced the number of accidents on these bridges from 20 in a 22-month period to 4 in a 17-month period while the ADT increased from 4,780 to 5,690. The treatment consisted of the following: placing diagonal shoulder markings 2-feet wide at 45 degrees starting 225 feet from the structure, placing raised jiggle bars on every fourth shoulder marking, providing a continuous guardrail from 225 feet before the structure offset 8 feet, tapering to the bridge and continuing on across, and post mounted delineators placed behind the guardrail.

Medians

The divided highway consists of two roadways separated by a median. Research reported in 1973 by Garner and Deen (21) conducted in Kentucky involved studying a variety of median types on 420 miles of toll road and interstate system opened prior to 1966. This research has shown that both the total median accident rate and the accident severity rate decline with increasing median width. A breaking point or "leveling off" seems to occur for median widths between 30 and 40 feet. See Figure 7.

TABLE 3 - Safety of Narrow Bridges.

Bridge Width-Roadway Width in Feet	-6	-4	-2	0	2	4	6	8	10	12
Accidents per 100 Million Vehicles	120	103	87	72	58	44	31	20	12	7

SOURCE: References 18 and 19

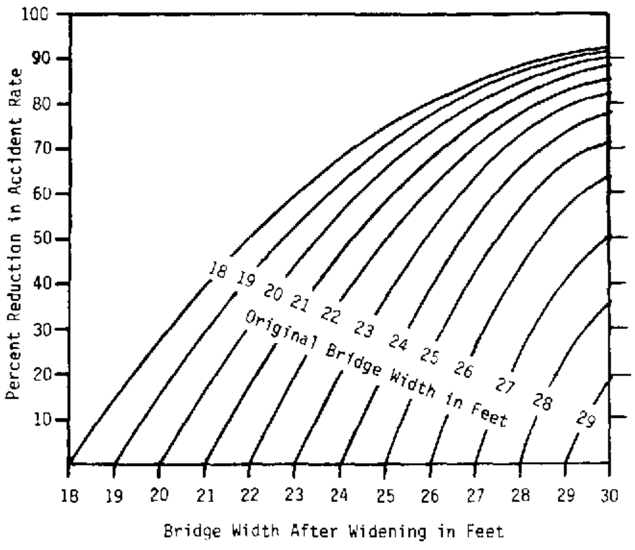
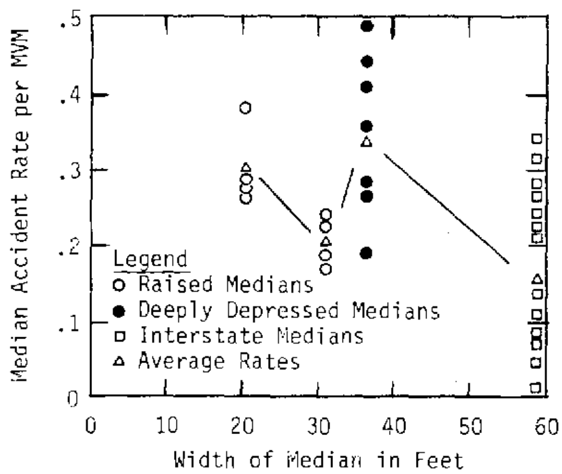


Figure 6. Percent Reduction in Accident Rate Associated With Increases in Bridge Width

SOURCE: Reference 19



MVM - Million Vehicle Miles

Figure 7. Median Accident Rate versus Median Width

SOURCE: Reference 21

They found, however, other elements of the median, such as cross slopes and presence of obstructions and irregularities, can have a greater effect on safety of the median than width. The beneficial effect of wide medians can be completely negated by steep slopes. The Garner and Deen study in Kentucky showed that 4:1 and 3:1 cross slopes of the 36-foot deeply depressed medians have high median accident rates. The cross slopes of the 20-, 30-, and 60-foot medians were relatively mild when compared to the 36-foot medians. The steep slopes do not provide reasonable recovery areas and are often a hazard in themselves.

Foody and Culp (22) reported in 1974 on their study of the safety benefits associated with 84-foot-wide medians as to mound type (raised) versus swale type (depressed) for interstate highways in Ohio. About 130 miles of each median type for four-lane divided highways were studied and the accident data from 1969 through 1971 were analyzed. The results indicated that either type provides a generally adequate recovery area for encroaching vehicles although the swale median appears to provide more opportunity for encroaching vehicles to regain control and return to their roadway. The swale type median had 8:1 slopes to a 4-foot-deep ditch in the center. The mound type had 8:1 slopes down to 1.6-foot ditch with a 30-foot-wide, 5-foot-high mound in the center which had 3:1 slopes.

More information on the safety of medians with regard to openings and barriers is presented in Chapters 3 and 4 on "Roadside Features" and "Access Control and Driveways."

Pavement Cross Slope

Roadway pavements are generally designed to slope from the centerline toward the edges to accommodate drainage during wet weather. Flat cross slopes on flat roadway sections cause water to accumulate on the pavement surface during heavy rains and vehicles are more likely to be involved in hydroplaning accidents. Dart and Mann (4) determined that in Louisiana, which gets 60 inches of rainfall a year, roadways with relatively flat cross slopes are more accident prone than those with steeper slopes. See Figure 8.

According to research by Gallaway et al. (23), the most critical location for hydroplaning is sag-vertical curves where the bottom of the curve is subject to flooding. Pavement cross slope is a dominate factor in removing water from the pavement surface and a minimum cross slope of 2 percent (0.02 ft/ft) was recommended as a remedial treatment for these locations.

Side Slope and Ditches

Weaver et al. (24) conducted a series of computer simulation studies using the Highway Vehicle Object Simulation Model (HVOSM) to evaluate the effects various side slopes and ditch configurations would have on vehicles which run off the road. The simulation results were later verified with field studies. They found that the hinge point, the location where the shoulder meets the side slope, produced no critical adverse effects for side slopes of 3:1 to 10:1.

Return maneuvers can be accomplished without vehicle rollovers on smooth, firm embankments of 3:1 or flatter at speeds up to 80 mph and encroachment angles of 15 degrees. However, to permit recovery, a coefficient of friction of 0.6 must be available. Almost no returns can be executed when the coefficient of friction is as low as 0.2 (a more probable value than 0.6).

The trapezoidal ditch configuration represents the most desirable cross section from a safety standpoint, particularly for ditches wider than 8 feet. The use of front slopes (the slope from the shoulder to the bottom of the ditch) steeper than 4:1 is not desirable because their use severely limits the choice of back slopes to produce a safe ditch configuration. Ditch evaluation curves for roadside slope combinations are shown in Figure 9.

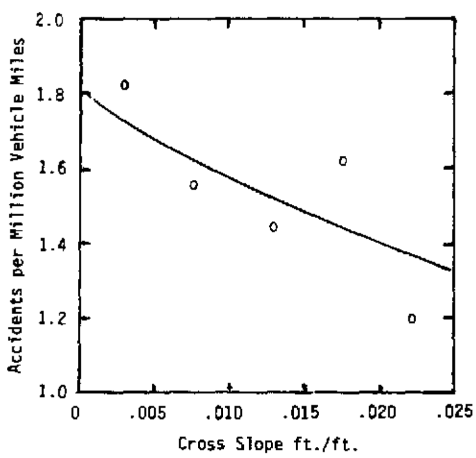


Figure 8. Accident Rate versus Pavement Cross Slope

SOURCE: Reference 4

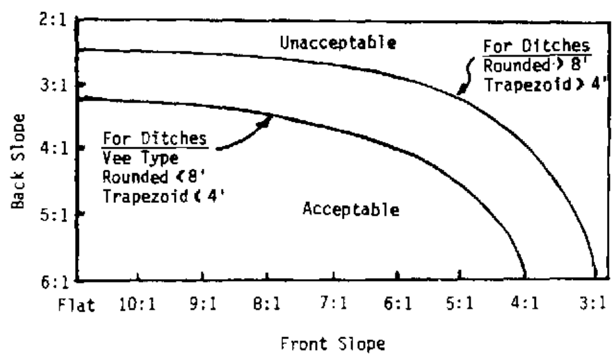


Figure 9. Ditch Evaluation for Roadside Slope Combinations

SOURCE: Reference 24

ALINEMENT

The alinement of a roadway includes both the horizontal line of the roadway such as straight sections (tangents) and curves as well as the vertical line (profile) such as grades and vertical curves (sags and crests). An estimate of the total number of accidents occurring on these various alinement features for 1979 and 1980 is presented in Table 4 (25).

Although most accidents occur on straight, level ground, the curves and grades are more hazardous, as the accident rates are higher for these features as will be shown in the following sections.

HORIZONTAL CURVES

Past research generally shows that as the degree of curve increases, the accident rate increases. (The degree of curvature is the central angle subtended by an arc of 100 feet.) In his 1953 classic study, Raff (26) reported on how accident rates on main rural highways are affected by design features. Fifteen States provided information covering 1 year's accident experience (16,421 accidents) on about 5,000 miles of highway. Factors studied included number of lanes, average daily traffic volume, degree of curvature, pavement and shoulder widths, frequencies of curves and other sight-distance restrictions. Two-lane, three-lane, and four-lane divided and undivided roadways were analyzed. Adjustments were considered for differing reporting requirements but the analyses using unadjusted accident rates appeared to be most reasonable. Raff shows that for all types of roadways the sharper the curve, the higher the accident rates. See Table 5.

Dunlap et al. (27) reported in 1978 the results from NCHRP Project 1-14 which studied accident records for the Pennsylvania and Ohio Turnpikes. They reviewed 9,822 mainline accidents covering 2-1/2 years starting in 1966 for the Pennsylvania Turnpike and 5,553 accidents covering 4-1/2 years starting in 1966 for the Ohio Turnpike. The study analyzed the effects of horizontal and vertical alinement on accident rates. On the Ohio Turnpike, they found no significant accident dependence on either grade or curvature, except that a 1° curve on a 3 percent downgrade had a very high accident rate. The data showed the Pennsylvania Turnpike accident rate was not dependent on grade, but it did increase with increasing curvature. Figure 10 was developed using the Pennsylvania Turnpike data.

TABLE 4. Percent of Total Accidents by Alinement and Profile Features in the United States for 1979 and 1980 (Total Number of Accidents = 6,773,000).

	Single Vehicle Accidents		Multi-Vehicle Accidents		Totals
	Curved	Straight	Curved	Straight	
On Grades	3.0	3.6	1.5	7.7	15.8
Sag or Hillcreast	0.2	0.5	0.4	0.8	2.0
Level Ground	3.9	23.2	3.6	51.4	85.2
TOTALS	7.1	27.4	5.6	59.9	100.0

SOURCE: Reference 25

TABLE 5 - Accident Rates on Curves, by Degree of Curve and Roadway Type

	Two-Lane Roads		Three-Lane Roads		Four-Lane Roads					
					Undivided		Divided		Controlled Access	
Curvature	Number of Accidents	Per Mil. Vehicle-Miles	Number of Accidents	Per mil. Vehicle-Miles	Number of Accidents	Per Mil. Vehicles-Miles	Number of Accidents	Per Mil. Vehicle-Miles	Number of Accidents	Per Mil. Vehicle-Miles
0 - 2.9°	504	1.6	11	1.7	98	1.9	95	1.8	180	1.6
3 - 5.9°	596	2.5	11	2.8	90	2.6	65	2.4	162	2.3
6 - 9.9°	338	2.8	6	3.5	16	3.3	5	3.1	38	4.5
10° or more	354	3.5	11	7.3	3	1.2	12	6.7	0	-
Tangents	6,474	2.3	227	2.5	1,348	2.7	982	2.9	774	1.7

SOURCE: Reference 26

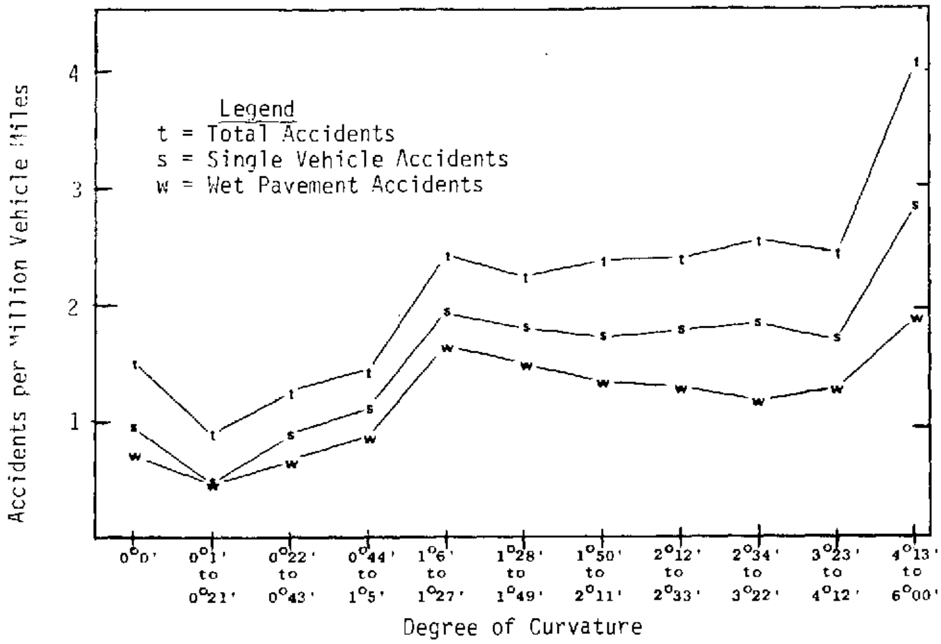


Figure 10. Expected Accident Rate Using Pennsylvania Turnpike Data for Grades of ± 0.6 Percent

SOURCE: Reference 27

In 1981, Smith et al. (28) analyzed three accident data bases to quantify safety performance of rural two-lane highways. These data bases were collected for prior FHWA research studies. The data bases were: "Skid-Reduction" data base, developed by Midwest Research Institute; the "Delineation" data base developed by Science Applications, Inc.; and the "Calspan" data base developed by Calspan Field Services, Inc.

The "Skid Reduction" data base consisted of accident rates, skid numbers, ADT's and related geometric data for two 1-year periods on 455 sections of two-lane rural highways, including 2,212 miles in 15 States. Before accidents totaling 7,157 were for 1-year periods for the years 1970 through 1973. The 1-year-after period occurred during 1974 and/or 1975 for 363 of the 455 sections, which included 1,758 miles experiencing 6,032 accidents.

The "Delineation" data base contained data on accidents, geometrics, traffic controls and traffic volumes on 320 roadway sections and 194 horizontal curves. The 320 roadway sections had 12,414 recorded accidents and the curves 5,022 accidents.

The "Calspan" data base provides 6,651 single-vehicle accidents that occurred on two-lane rural roads during 1975 through 1977 in six States. The data provided 375 items for each accident which included roadway geometric data, traffic volume, etc. rather than the number of accidents which occurred on specific roadway sections. Thus, accident rates as such were not recorded.

The 1981 Smith study (28) shows in Table 6 that accident rates generally increase with the degree of horizontal curvature for the Delineation data base, but have no apparent trend in the Skid Reduction data base. The accident rate for tangents for the Skid Reduction data base is slightly less than the overall accident rate for horizontal curves. The accident rates for the Delineation data base are significantly higher than for the curves in the Skid Reduction data base. The Delineation data base may contain a bias due to the selection of horizontal curves to receive delineation treatment on the basis of high accident experience.

TABLE 6 - Accident Rates for Tangents and Horizontal Curves for Two-Lane Rural Roads

Degree of Horizontal Curvature	Accidents per Million Vehicle Miles	
	Skid Reduction Data Base	Delineation Data Base
0 (tangent)	2.199	-
Less than 1.55	2.252	-
1.55 - 3.25	2.503	4.590
3.25 - 5.50	2.319	5.960
Over 5.50	-	7.718
All Degrees	2.329	6.196

SOURCE: Reference 28

The Calspan data base (28) provides insight into the characteristics of single vehicle accidents on two-lane rural curved versus tangent highways. Table 7 provides an analysis of single vehicle accidents by route familiarity by the driver. It can be noted that while 23 percent of drivers who had accidents on tangents were first time or rarely traveled the roadway, 31 percent of the accidents on curves involved such drivers. Thus, although curve warning devices may not have an effect on the local driver, there may be substantial benefits to drivers who rarely frequent that section of highway.

Departure direction for right and left curves by degree of curvature was also developed by Smith (28) from the Calspan data. See Figure 11. The percentage of departures on the outside of curves increases as the degree of curvature increases for both right and left curves. For tangents, vehicles are twice as likely to depart on the right as on the left, presumably because they are closer to the right-hand roadside and they have more recovery room on the left or may become involved in a multi-vehicle accident before departing on the left.

Raff (26) in 1953 reported on accident rates on two-lane curves as a function of curve frequency. Table 8 shows that the rates are higher when there are more curves per mile except for the sharp curves. When there is a curve of 10 degrees or more after a long straight tangent, the rate goes up.

TRANSITION CURVES AND SUPERELEVATION

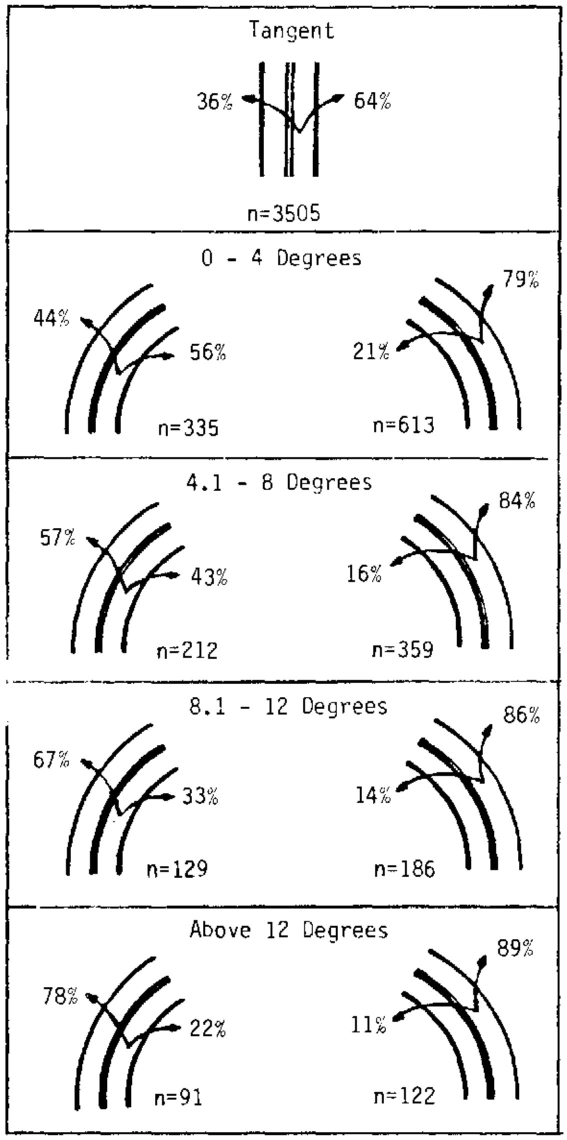
Transition curves are often used to connect the tangent section of the roadway with the horizontal curvature of a roadway. For curves with long radii (small degrees of curvature) transition curves are often not used. For sharper curves either compound curves (i.e., two simple curves are used to connect with the main curve) or a spiral curve (i.e., a curve with an increasing radius from the tangent to the main curve) are used to connect with the tangent section. Although no accident studies have been reported on the application of transition curves, a study by Segal and Ranney (29) reported in 1980 analyzed the vehicle dynamics for transition curves. They used the computer simulation model, Highway-Vehicle-Object Simulation Model (HVOSM) for the analysis of 8^o, 31^o, and 38^o curves. They found that vehicles' lateral acceleration with no transition curve was as much as 50% greater than the steady-state acceleration, while the spiral transition simulated less than 10%. They concluded no transition was the worst case, compound transitions better, and the spiral

TABLE 7 - Single Vehicle Accidents by Horizontal Alinement by Route Familiarity

Familiarity	Tangent		Left Curve		Right Curve	
	N	%	N	%	N	%
Daily	1,066	35.1	425	29.0	276	30.2
1+ Week	786	25.9	375	25.6	211	23.1
1+ Month	472	15.6	206	14.0	146	16.0
Rarely	464	15.3	281	19.2	165	18.0
First Time	245	8.1	180	12.2	117	12.8
Total	3,033	100.0	1,467	100.0	915	100.0

N = Number of Accidents

SOURCE: Reference 28



Right Curve Left Curve

Figure 11. Direction of Departure by Degree of Curvature for Two-Lane Highways

n = Number of Departures

SOURCE: Reference 28

TABLE 8 - Accident Rates on Two-Lane Curves, by Degree of Curvature and Frequency of Curves

Degree of Curvature	$0 - 2.9^{0}$		$3^{0} - 5.9^{0}$		$6^{0} - 9.9^{0}$		10^{0} or more	
Frequency of Curves	Number Accidents	Per Mil. Vehicle-Miles	Number Accidents	Per. Mil. Vehicle-Miles	Number Accidents	Per. Mil. Vehicle-Miles	Number Accidents	Per. Mil. Vehicle-Miles
Number per mile								
0 - 0.9	128	1.4	110	2.7	13	2.0	31	4.3
1.0 - 2.9	178	1.4	163	2.1	96	2.9	53	2.6
3.0 - 4.9	125	1.9	223	2.5	170	2.9	139	3.4
5.0 - 6.9	75	3.1	100	2.9	59	2.6	130	3.9

SOURCE: Reference 26

allowed the easiest path to follow. Segal and Ranney (29) also studied superelevation effects for the three curves and concluded that superelevation does not appear to play a significant role in affecting transient vehicle dynamics on curves but does influence the steady-state steer characteristics of the vehicle. The greater the superelevation, the less the vehicle understeers.

Dunlap et al. (27) studied in detail the causes for a high accident rate (55 accidents in 6 years) on a 1^{0} curve and 2 percent downgrade on the Ohio Turnpike. Of the total accidents, 67 percent involved skidding during wet weather. This curve had a superelevation of only 0.0156 ft/ft. They concluded that the lateral acceleration for this curve is relatively large and the water depth during rain storms would be greater than on smaller radius curves having larger superelevations. They recommend on long radius curves, higher superelevations be used to compensate for the increased drainage path length. Increasing the superelevation from 0.0156 to 0.06 will reduce the water depths by about one third and thus reduce the wet weather accident rate.

Delineation Treatments

One countermeasure to make horizontal curves safer involves delineation treatments along the roadway. Stimpson, McGee, Kittelson, and Ruddy (30) reported in 1977 on delineation treatments on two-lane highways. Their study reviewed accidents that might possibly be related to delineation treatments or the lack thereof. Their data base was discussed before as the "delineation" data base. The delineation related accidents were considered to be those accidents that did not involve snowy or icy pavements, collisions with an object on the pavement, defective roadways or vehicles, or improper maneuvers on roadways. They determined from their data base that the portion of delineation related accidents for three alinement situations was as follows:

- o Tangent - 68 percent
- o Winding Roads - 80 percent
- o Horizontal Curves - 74 percent

Stimpson et al. tested 18 delineation treatments ranging from just a solid painted centerline to the centerline marked with retroreflective raised pavement markers, edgelines with raised pavement markers and post mounted delineators. These treatments were compared to the standard centerline and edgeline treatments designated by the then current MUTCD. An accident potential model was developed and validated to test the various treatments. This model involved measurements of vehicles centrally indexed (CI) the mean location of the vehicles in relation to the center of the lane, and the difference in lateral placement variance (OPV) which is the variance of the location of the vehicles within the lane divided by the lane width. The accident rate (AR) is the number of nighttime, delineation-related, non-intersection accidents per million vehicle miles (dry pavement conditions).

$$AR = -0.22 + 1.15\ CI + 25.3\ DPV$$

The 18 treatments were applied to eight test sites (four tangent sections, two winding roadways and two horizontal curves) and measurements of speed as well as CI and DPV were made to estimate the safety potential of the various treatments. The study concluded that several less paint-intensive delineation systems performed as well or better than the more expensive base (the then MUTCD standard) condition. This included the centerline skip ratio of 10-foot paint stripes and 30-foot gaps, (which was later adopted in the 1978 MUTCD) rather than the 15-foot and 25-foot spacings. This provided an estimated 4 percent cost savings. It was recommended that edgelines from 2-3 inches wide could also be used. This would save costs of an additional 12 percent, if adopted. Where severe visibility conditions occur due to frequent fog or blowing sand, the researchers recommended retroreflective raised pavement markers be considered at 80-foot intervals where passing is permitted and 40-foot intervals where passing is prohibited. Where raised pavement centerline markers cannot be applied because of snowplowing, post mounted delineators should be installed at 400 to 528-foot intervals on tangents and the MUTCD recommended spacing for curves of various radii. The post mounted delineators on tangents had a negligible effect on lateral placement, but did affect speeds and placement variance on curved sections. They concluded that further research is needed on the use of a single solid centerline in no-passing zones. The use of reflective raised pavement markers on both the centerline and edgeline showed a 68 percent reduction in potential hazard, but costs about 900 times the standard painted markings and it was considered to be too expensive for general use.

Niessner (31) reported in 1982 on the results from eight States that participated in studies to evaluate 13 types of post mounted delineators. Although flexible posts cost twice as much as the standard U-channel type, it would be cost effective to use the flexible posts in areas subject to numerous impacts if the flexible posts can survive two or more hits. The accident data collected by the States indicates a trend toward reducing run-off-the-road accidents where post mounted delineators are installed. Montana Department of Highways (32) reported a 30 percent reduction of run-off-the-road nighttime accidents at delineator test locations (primarily curve and narrow bridge sections) with the larger delineator types being more effective than the small type.

Curve Warning Signs

In 1980, Lyles (33) reported on a study conducted at two sites on the Maine Facility which examined the effectiveness of several alternative sign configurations (both warning and regulatory) for warning motorists of a hazardous horizontal curve ahead in a rural two-lane situation. He found in spite of relatively large decreases in speed in the vicinity of the curve, no sign configuration was found to be consistently more effective in reducing speeds than another. The study did not test, however, the curve sites without the standard curve warning sign. The sign configurations included the standard curve warning signs alone, with advisory speed plates, with "Reduced Speed Ahead" sign and a regulatory speed limit sign, and with a "Maximum Safe Speed 35 MPH" sign. Also a test condition involved changing the placement of the curve warning signs in advance of the curve from 500 feet to 700 feet.

Grades

The vertical alinement of a highway is generally a compromise between a desire to allow the highway profile to conform with the terrain, safety and construction costs. The vertical alinement is a combination of straight roadway sections at a set slope (grades are expressed as the percentages of rise to horizontal distance) and vertical curves (usually parabolic) to connect the slopes in crest or sag curves.

The 1953 Raff study (26) found that grade alone did not have any particular effect on accident rates for tangent sections on any type of rural highway. The combination of grade and horizontal curvature did show steeper grades increase the accident rates for two-lane rural curved sections with average daily traffic volumes (ADT's) between 5,000 and 9,900 vehicles per day. See Table 9.

TABLE 9 - Accident Rates on Two-Lane Curves for Traffic Volumes of 5,000 to 9,900 for Grades Above and Below Three Percent

Curvature Degrees	Grades Less than 3%		Grades More than 3%	
	No. Acc	Acc/MVM	No. Acc	Acc/MVM
0 - 2.9	86	1.9	22	2.9
3 - 5.9	117	2.8	55	4.1
6 - 9.9	51	2.6	22	3.1
10 or More	27	2.5	22	3.9

SOURCE: Reference 26

An NCHRP Study by St. John and Kobett (34) reported in 1978, analyzed the safety effects of long steep grades on two-lane rural highways by using a computer simulation model to determine traffic speed distributions and then estimated accident rates by using Solomon's (35) 1964 report on "Accidents on Main Rural Highways Related to Speed, Driver and Vehicle" where accident involvements are presented as a function of the deviation from the mean speed. Accident estimates were made for a variety of terrains. In flows up sustained grades of 4-8 percent, vehicle populations with many recreation vehicles (26%) and a few trucks (3%) have accident involvement rates that are about 133 percent of the rates of flows involving only passenger vehicles. With 20 percent low performance trucks, the involvement rates on long 4-8 percent grades increase to 175 to 250 percent of the passenger car only rates. On severely rolling terrain, the accident rates are expected to be slightly increased by the presence of recreation vehicles and trucks. On long steep downgrades, greater than 4 percent, trucks using crawl speeds to maintain control increase the accident rates.

Bitzel (36) reported in 1957 a study of accident rates on German expressways. He studied 25,500 accidents on 2100 km (1,300 miles) of expressways for the years 1953 through 1955. He found the accident rate increased as the grade increased as is shown in the Table 10. The table shows steep grades of 6-8 percent produce over four times the accidents as gradients under 2 percent.

TABLE 10 - Accident Rates Related to Grades on German Expressways

Roadway Grade %	Accident Rate Acc/MVKm	Acc/MVM
0 - 1.9	0.46	.74
2 - 3.9	0.67	1.08
4 - 5.9	1.90	3.06
6 - 8	2.10	3.38

SOURCE: Reference 36

Bitzel also found the combination of grades and horizontal curvature were at high accident locations. The superelevation in combination with the grade produced oblique gradients of over 8 percent. Skidding accidents occurring during wet weather caused vehicles to either slide off the road or collide with vehicles they were passing. The results are shown in Table 11. As the grades become steeper and the degree of curvature increases, the accident rate increases.

TABLE 11 - Influence of Curves on Gradients on Accident Rates for German Expressways

Curvature in Degrees	Accidents/Million Vehicle Miles Gradients in Percent 0-1.99%	2-3.99%	4-5.99%	6-8%
>0.48	0.45	0.48	1.69	2.13
.48-0.54	0.68	0.40	2.09	2.50
0.54-0.80	0.64	0.32	2.42	2.74
0.80-1.61	0.81	1.13	2.98	3.22
1.61-4.02	1.18	1.71	3.09	3.75

SOURCE: Reference 36 (Revised to show Accidents /MVM and Curves in Degrees)

Vertical Curves

Vertical curves are installed to connect grades of different slopes. The lengths of the vertical curves are usually based upon the difference between the grades and the required stopping sight distance for the design speed of the roadway. For crest vertical curves, the sight distances are determined for drivers to see over the top of the hill to objects on the other side. For sag vertical curves the sight distances are determined for drivers seeing at night from the vehicles' headlights. The crest vertical curve is one of the primary features of the roadway which limits sight distance.

In the 1953 Raff (26) report an analysis of sight-distance restrictions on tangents was conducted for two lane rural roads. For the study, a sight distance of less than 600 feet for flat or rolling terrain or less than 400 feet for mountainous terrain was considered to be a sight restriction. The results are shown in Table 12. The accident rate rises as the restriction frequency increases from zero to about three restrictions per mile.

TABLE 12 - Accident Rates on Two-Lane Tangents by Frequency of Sight Distance Restrictions

Frequency of Restrictions Per Mile	No. of Accidents	Accidents per MVM
0 - 0.9	3,472	2.0
1.0 - 1.9	1,061	2.5
2.0 - 2.9	891	3.1
3.0 - 3.9	648	3.0
4.0 - 4 9	354	3.0
5.0 - 5.9	12	2.7

SOURCE: Reference 26

In 1961, Mullins and Keese (37) investigated 10,000 accidents on 54 miles of freeways in the five largest Texas cities covering from 2 to 5 years of data. They found a concentration of accidents at crest and sag vertical curves. Rear-end type collisions comprised 70 percent of all accidents as a result of following too close. The study showed unfavorable sight conditions were present at the high accident frequency crest and sag locations. The results are shown in Table 13.

TABLE 13 - Freeway Accidents and Vertical Curvature

Type of Vertical Curve and Position	Accidents/ MVM
CRESTS (General)	2.02
On upgrade of crests	2.33
At peak of crests	1.96
On downgrade of crest	1.92
SAGS (General)	2.96
On downgrade of sags	3.57
At bottom of sags	2.45
On upgrade of sags	2.39

SOURCE: Reference 37

Limited Sight Distance Controls

In 1981, Christian, Barnack and Karoly (38) evaluated the "Limited Sight Distance" warning signs in New York. Spot speed studies were taken at 14 locations in five counties with and without the warning sign and its accompanying advisory speed panel. They recorded speeds at the crest of the vertical curve. The results indicate the warning signs with advisory speed panels had no affect in slowing the speed of vehicles, in fact, at five sites, the speeds decreased when the signs were removed.

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CHAPTER 2 - PAVEMENT SURFACE

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INTRODUCTION

The subject chapter is a synthesis of safety research related to the interactions of pavement surface texture, vehicle tire treads, and moisture on the pavement. The research involves the reduced safety caused by loss of friction -- resulting in skidding -- due to too smooth pavement surface, inadequate tires, and wet pavement.

Water is the most frequent pavement contaminant, thus, wet pavement safety will be the major concern of this chapter. The problems of ice and snow are treated in Chapter 11 of this synthesis, "Adverse Environmental Operations." Drainage problems due to cross slopes and vertical alinement are discussed in Chapter 1, "Roadway Cross Section and Alinement."

Friction coefficients of dry pavements (ratio of friction force to wheel load) may be 0.8 or more. Wet pavements may have friction coefficients ranging from 0.1 to 0.6. If the originally high skid resistance of a pavement surface has been worn away by traffic, it may become extremely slippery when water lubricates the tire-road surface contact area. If the driver of a vehicle attempts to brake, turn, or accelerate, the tire-road contact friction may be inadequate creating a serious accident potential.

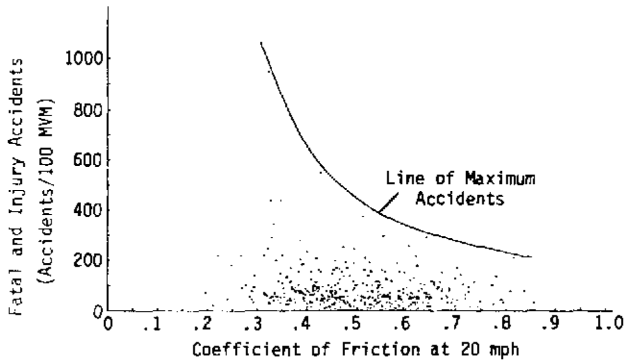
Where a layer of water stands on the pavement, the tires of a vehicle may become separated from the road surface resulting in a loss of vehicle control. This phenomenon, termed "hydroplaning," is another area of concern regarding wet pavement safety.

SKID RESISTANCE AND ACCIDENTS

A considerable amount of research has been directed toward the establishment of a specific skid number as a criteria for skid resistance requirements. One of the major drawbacks in comparing studies of the relationship between wet weather accidents and tire-pavement friction involves the number of different techniques used to determine tire-pavement friction. These methods vary from the predominant United States technique of measuring the skid number at 40 miles per hour (SN40), the ratio of skid resistance on wet pavement to wheel load times 100 being SN40, to the widely varied usage of different test methods by other countries.

Ivey and Griffin (1) reviewed a number of studies of accident rate and pavement friction related to a skid number at 40 miles per hour. Studies conducted in Texas by McCullough and Hankins (2) and in Kentucky by Rizenbergs et al. (3) illustrate the high degree of variability of accident data when related to pavement friction as shown in Figures 1 and 2. The accidents are influenced by a number of factors such as geometrics, weather conditions, and driver behavior in addition to pavement friction.

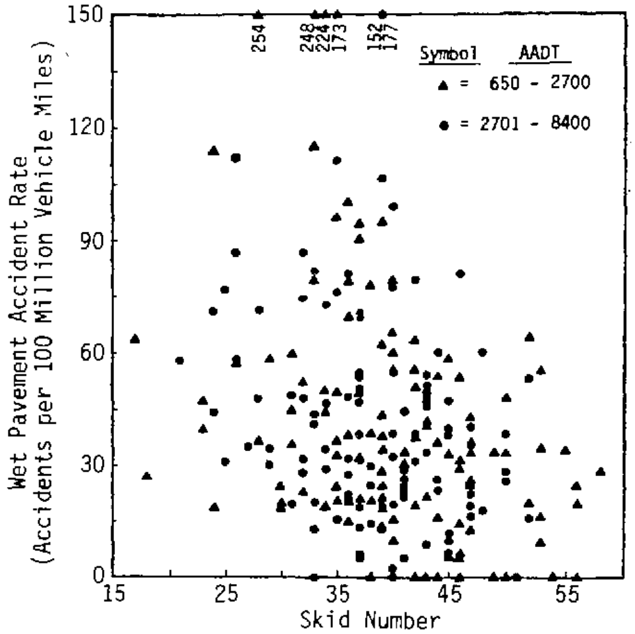
Based on studies in Great Britain, Giles (4) reported the curve shown in Figure 3. An estimate of SN40 relationship to Side Force Coefficient is shown by the SN40 ranges added along the abscissa of Figure 3 as provided from the work of Ivey and Griffin (1). If the relationship between SN40 and Brake Force Coefficient is reasonable, the curve illustrates a rapid increase in the "Liability to be Skid Accident Site" when SN40 is reduced somewhat below a value of 40.



MVM - Million Vehicle Miles

Figure 1. Comparison of Accidents and Coefficients of Friction

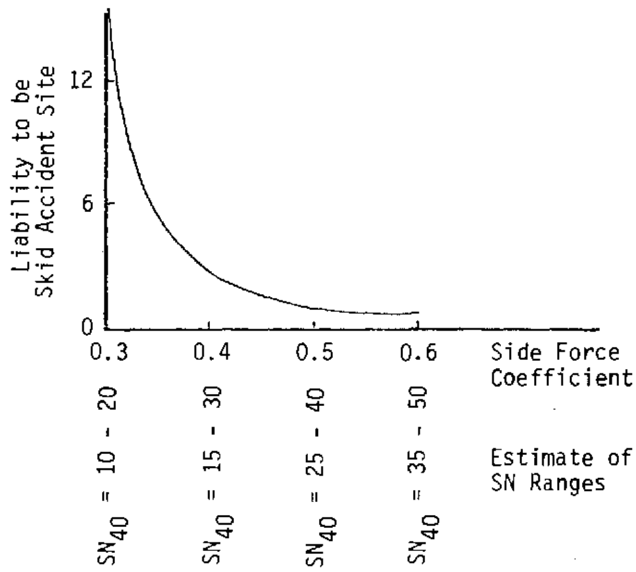
SOURCE: Reference 2



AADT - Average Annual Daily Traffic

Figure 2. Wet Pavement Accident Rate versus Skid Numbers

SOURCE: Reference 3



SN - Skid Number

Figure 3. Estimate of the Liability to be a Skid Accident Site

SOURCE: References 1 and 4

A German study by Schulze (5) related the percentage of wet weather accidents to a locked wheel brake force at 50 mph as illustrated in Figure 4. As in Giles work (4), wet weather accidents are increasingly sensitive to the decreasing friction value.

Rizenbergs et al. (3) conducted an extensive study of tire-pavement friction and wet weather accidents in Kentucky. The data were stratified on both traffic volume and road class. The usual data trend as interpreted by Rizenbergs is illustrated in Figure 5. This data trend indicates little sensitivity to friction when SN40 is greater than 40 and becomes increasingly sensitive as SN40 is reduced below 40. Some researchers indicate there may not be statistical justification for the curvilinear versus the linear interpretation.

Another report from Kentucky studies by Havens et al. (6) shows, in Figure 6, a relationship for rural, four-lane roads between SN40 and wet-surface accident rate.

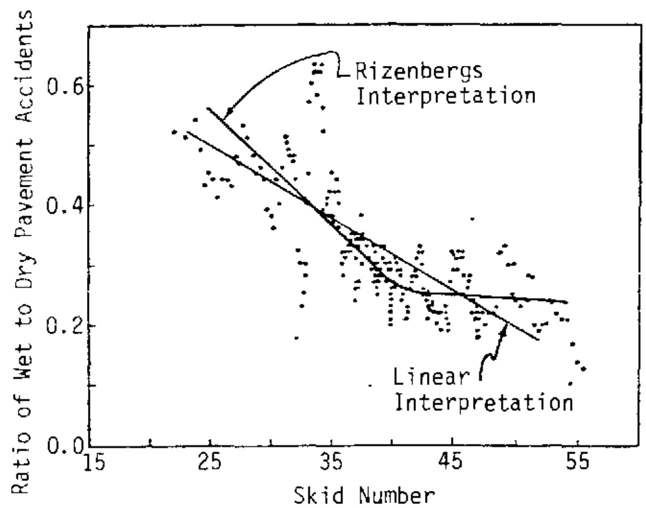
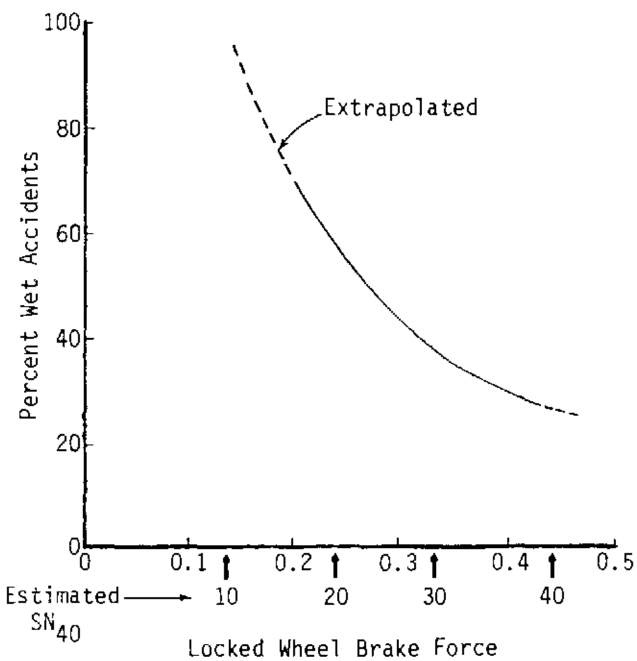


Figure 5. Ratio of Wet to Dry Pavement Accidents versus Skid Number

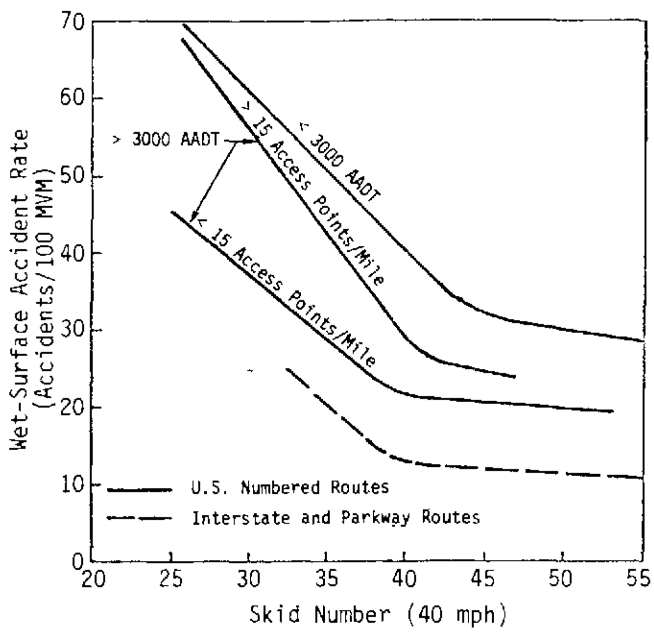
SOURCE: Reference 3



SN - Skid Number

Figure 4. Wet Pavement Accident Sensitivity to Pavement Friction

SOURCE: Reference 5



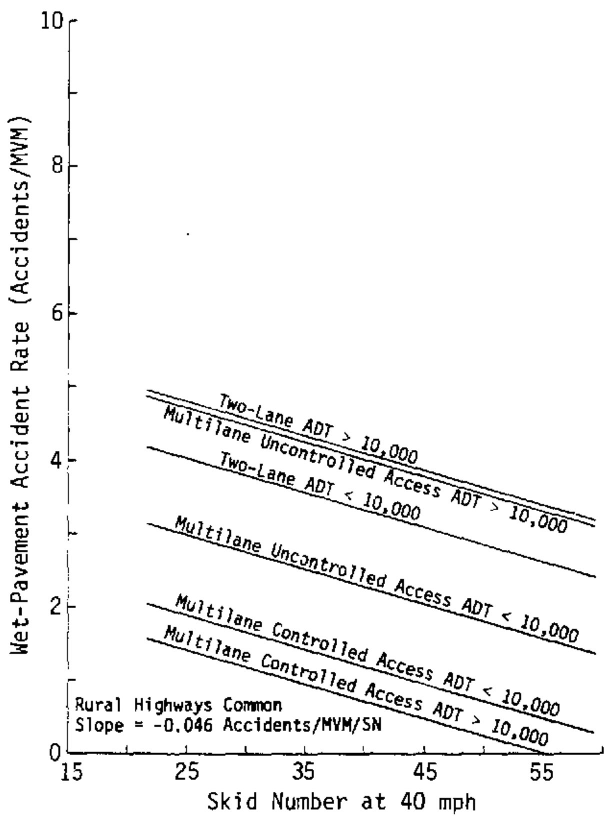
AADT - Average Annual Daily Traffic
MVM - Million Vehicle Miles

Figure 6. Relationship Between Wet-Surface Accident Rate and Skid Number

SOURCE: Reference 6

Blackburn et al. (7) conducted an extensive before and after study to determine the relationship between wet-pavement accident rate and skid number for different highway classifications. A statistically significant inverse relationship between skid number and wet-pavement accident rate was found for both rural and urban highways across various ADT levels. These effects are shown in Figures 7 and 8. A relationship between dry-and wet-pavement accident rates was also established and is shown in Figure 9.

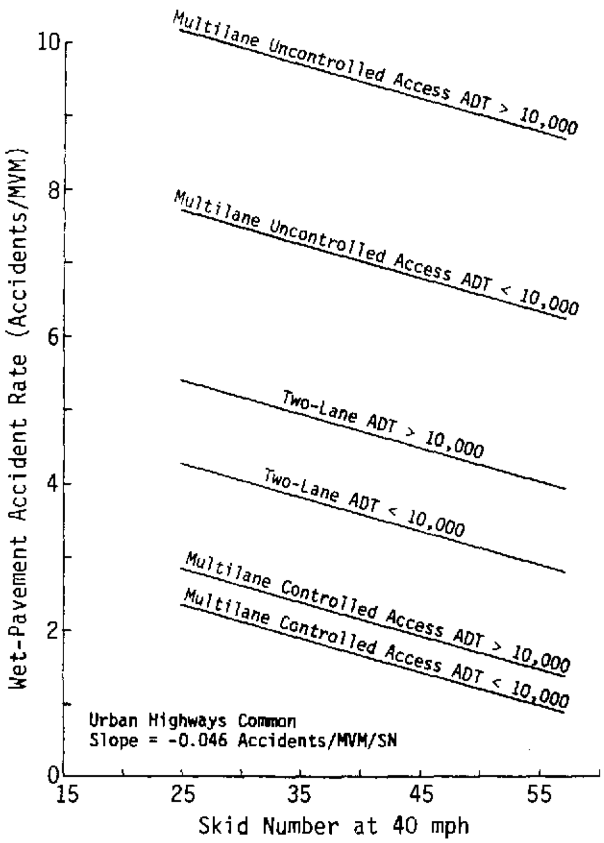
Holbrook (8) developed a model to estimate wet weather accidents from a review of precipitation data at 120 stations and approximately 40,000 accidents at 200 intersections in Michigan. In general, the fit of the model indicated no skid number hazard threshold. The wet pavement accident incidence increases monotonically and continuously as the skid number deteriorates. This relationship appears strongest for bituminous aggregate surfaces including asphalt concrete pavements. For all levels of wetness and all surfaces, a skid number less than 30 is accompanied by an accelerating increase in wet pavement accidents. The model was used to predict before and after accident differences for 30 resurfaced intersections. The model results were consistent with estimated results from police records as shown in Table 1.



ADT - Average Daily Traffic
MVM - Million Vehicle Miles
SN - Skid Number

Figure 7. Relationship Between Wet-Pavement Accident Rate and Skid Number at 40 mph for Rural Highways

SOURCE: Reference 7



ADT - Average Daily Traffic
MVM - Million Vehicle Miles
SN - Skid Number

Figure 8. Relationship Between Wet-Pavement Accident Rate and Skid Number at 40 mph for Urban Highways

SOURCE: Reference 7

Rice (9) investigated the seasonal variations in pavement skid resistance research by various States and the British Road Research Laboratory. Study variables included skid number, inches of rainfall, number of wet-dry pavement accidents, and percent accidents involving skidding. Figure 10 shows the annual cycle of pavement skid number change. Figure 11 gives the ratio of wet to dry pavement accidents to skid number. Findings of the report are as follows:

1. For asphalt surfaces, minimum levels of skid resistance are generally observed in the late summer and early fall, with maximum levels occurring in spring.

2. Short term variations are attributable to external factors such as amount and timing of rainfall, and the possibility of contamination from oily films, drippings, detritus, and other deposits on the surface.

3. In addition to real changes in pavement surface characteristics, temperature changes affect the properties of the tires involved in the skid resistance measuring system.

4. The measured skid resistance of a given surface can vary on the order of 10 to 20 or more skid numbers.

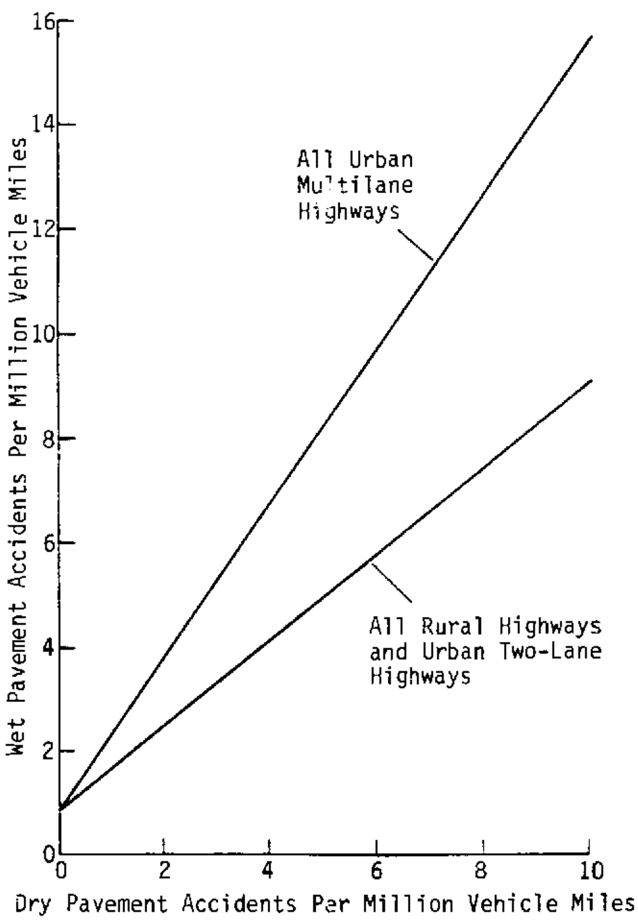


Figure 9. Relation Between Dry-Pavement and Wet-Pavement Accident Rates

SOURCE: Reference 7

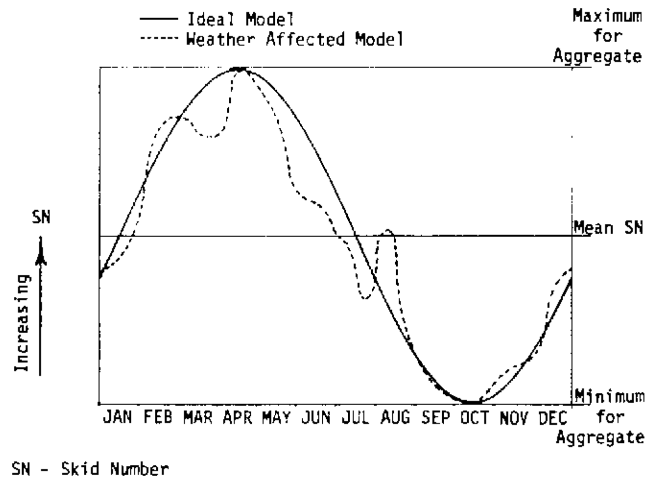


Figure 10. Annual Cycle of Pavement SN Changes

SOURCE: Reference 9

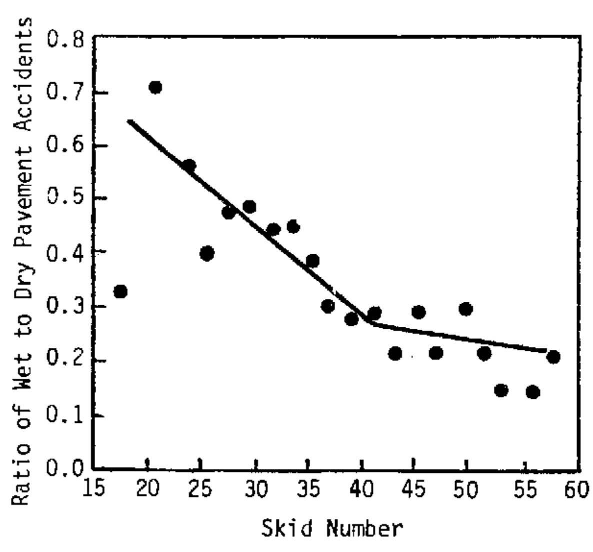


Figure 11. Ratio of Wet to Dry Pavement Accidents of 230 Test Sections in Kentucky -- Grouped by Skid Number -- versus Skid Number

SOURCE: References 3 and 9

TABLE 1 - Wet Surface Accident Prevention Benefits with Intersection Resurfacing

Basis of Computations	Number Wet Pavement Accidents		Location Months		Wet Pavements Accidents Per Location Per Month		Wet Pavement Accident Rate Decline	Estimated Number of Wet Pavement Accidents Prevented During the First Year After Resurfacing
	Before	After	Before	After	Before	After		
Police Files	556	590	384	526	1.45	1.12	-23%	120
Model predictions	618	635	384	526	1.61	1.21	-25%	144

SOURCE: Reference 8

The previous studies indicate there is a definite relationship between tire-pavement friction and wet-weather accident rates. There is, however, no overall agreement on the definition of a desired minimum skid resistance standard. Kummer and Meyer (10) provide recommendations for minimum skid resistance requirements as presented in Table 2.

TABLE 2 - Tentative Interim Skid Resistance Requirements For Main Rural Highways(a)

Traffic Speed (mph)	Recommended Minimum SN(b) Measured At Traffic Speed	Measured At 40 MPH
30	36	31
40	33	33
50	32	37
60	31	41
70	31	46

(a) From Table 18, NCHRP Report 37. These values are recommended for main rural two-lane highways. For limited-access highways lower values may be sufficient, whereas certain sites may require higher values.

(b) SN - skid number, measured according to ASTM Method E274

SOURCE: Reference 10

AUTOMOTIVE HYDROPLANING

The following section has been prepared from a paper by Balmer and Gallaway (11) dealing with automotive hydroplaning, the separation of a moving-vehicle tire from a solid pavement surface caused by the presence of a fluid on the surface. Vehicle operation may involve partial hydroplaning when a significant amount of water is present.

Horne (12) divided hydroplaning into three categories for pneumatic-tired vehicles:

1. Dynamic hydroplaning results from uplift forces acting on a moving tire from tire-fluid-solid interaction. Partial dynamic hydroplaning may occur at ordinary speeds. Uplift forces are not great enough to develop full dynamic hydroplaning without substantial vehicle speed and a significant water film thickness.

2. Viscous hydroplaning occurs when tire-fluid-solid interaction encounters cohesive forces in the thin film of water between a tire and a solid (pavement surface) that separates them. Viscous hydroplaning may occur at any speed. It is most prevalent on surfaces with insufficient microtexture to penetrate and diffuse the fluid film. An example is a tire on a wet, smooth pavement.

3. Tire-tread-rubber reversion hydroplaning results from tire-pavement interaction causing the rubber to melt and return to an uncured state. This phenomenon occurs from frictional heat for high speed vehicles such as aircraft, but is uncommon for passenger cars.

The probability of full dynamic hydroplaning on the highway is low, because the likelihood of the combined occurrence of factors causing full hydroplaning is small. For example, high intensity rainfalls are rare. It is unlikely many vehicles travel at high speeds during such rainfalls.

Extensive investigations by Horne (12), Gallaway et al. (13, 14), and Yeager (15) of hydroplaning have been conducted in the laboratory and the field. Analyses of these data show that hydroplaning can be decreased by application and/or consideration of the following:

1. A pavement cross slope of 2.5 percent will facilitate surface drainage, reduce tire hydroplaning, and improve traction during wet weather travel, while not being objectionable for vehicle steering or lane changing.

2. A pavement texture depth of 0.06 in. or greater will improve wet-pavement skid resistance and the cornering slip number, decrease hydroplaning tendencies, reduce splash and spray, and diffuse headlight glare, especially on high speed highways. There are usually small increases in tire rolling resistance, tire wear, and tire-pavement interaction noise as the texture depth increases. An open-graded asphaltic friction course will generate less tire-pavement noise than most other surfaces. Less texture depth is acceptable for low speed roadways and city streets.

3. Transverse pavement finishes or grooves permit reduced braking distances as compared with longitudinal grooves. Traffic may decrease large texture depths 25 percent or more during the first 6 months. The wear rate varies with the pavement type and texture characteristics. Smooth pavement or dense graded asphalt surfaces will not wear as rapidly.

4. Pavement maintenance or resurfacing is needed when rut depths exceed 0.24 in. on pavement cross slopes of 2.5 percent, if water ponding is to be avoided. Less rut depth can be tolerated for smaller cross slopes.

5. The pavement surface water layer thickness, which increases hydroplaning and decreases skid resistance, can be minimized by roadway design, construction, and rehabilitation. The water thickness depends upon the pavement cross slope, texture depth, rainfall intensity, and the pavement surface drainage path length. The drainage path length, which should also be minimized, is a function of the number of lanes and other roadway geometrics.

6. Drainage facilities should be provided to collect and rapidly remove water from sag-vertical curves to reduce hydroplaning susceptibility and improve traction.

7. Tire tread pattern depths should be greater than 2/32 in. for wet weather travel.

8. Vehicle tires should be inflated to the maximum design pressure to minimize hydroplaning on wet pavements.

9. Radial tires incur less rolling resistance than belted bias ply tires and will more than offset the increase in rolling resistance due to pavement texture increase. Less rolling resistance decreases vehicle fuel consumption.

10. An analysis of weather data shows that high intensity rainfalls are rare and of short duration. Hydroplaning or partial hydroplaning may occur from ponded water.

11. Speed should be reduced below 50 mph on wet pavement to decrease the probability of dynamic hydroplaning and to experience better skid resistance.

RESTORATION OF PAVEMENT

Renewal of existing pavement surfaces to restore wet frictional resistance involves either:

1. Modifying the existing pavement surface, or

2. Providing a new pavement layer over the existing pavement.

MODIFICATION OF EXISTING SURFACE

A number of procedures have been developed and employed to improve the coefficient of friction of existing pavement surfaces.

1. Grooving by sawing or grinding transverse or longitudinal grooves into the pavement surface.

2. Heat planing used to correct low friction due to bleeding of the binder at high volume intersections or other appropriate locations on bituminous surfaces.

3. Bush hammering, using percussion, to scour portland cement concrete pavement.

4. Milling the pavement with hard steel discs.

5. Spreading hydrochloric acid causing a chemical reaction to remove cement mortar exposing aggregate.

6. Sand blasting by abrasive jet.

7. High temperature flame scouring to spall the surface of portland cement concrete.

Studies revealing accident reduction from the use of grooving have been found and are being cited in this section. No such studies are known to exist for procedures 2 through 7. Studies regarding the applications of these techniques are listed under "Additional References."

GROOVING EXISTING SURFACE

The most prevalent method of restoring texture to the pavement surface is by grooving. Rotary saws with diamond tipped blades (or other extremely hard abrasive blades) saw or grind grooves of specific cross section and spacing in the pavement surface. Grooving in relation to the pavement centerline may be either longitudinal or transverse. Illustrations showing the various features of groove type, geometry (cross section), and patterns are presented in Figure 12 (16, 17).

Grooving drastically improves the macrotexture of the pavement surface which in turn enhances the drainage capacity. The NCHRP Synthesis 14 (16) describes grooving as a technique of altering a pavement surface to greatly increase its texture, thereby facilitating the displacement of water by the tires.

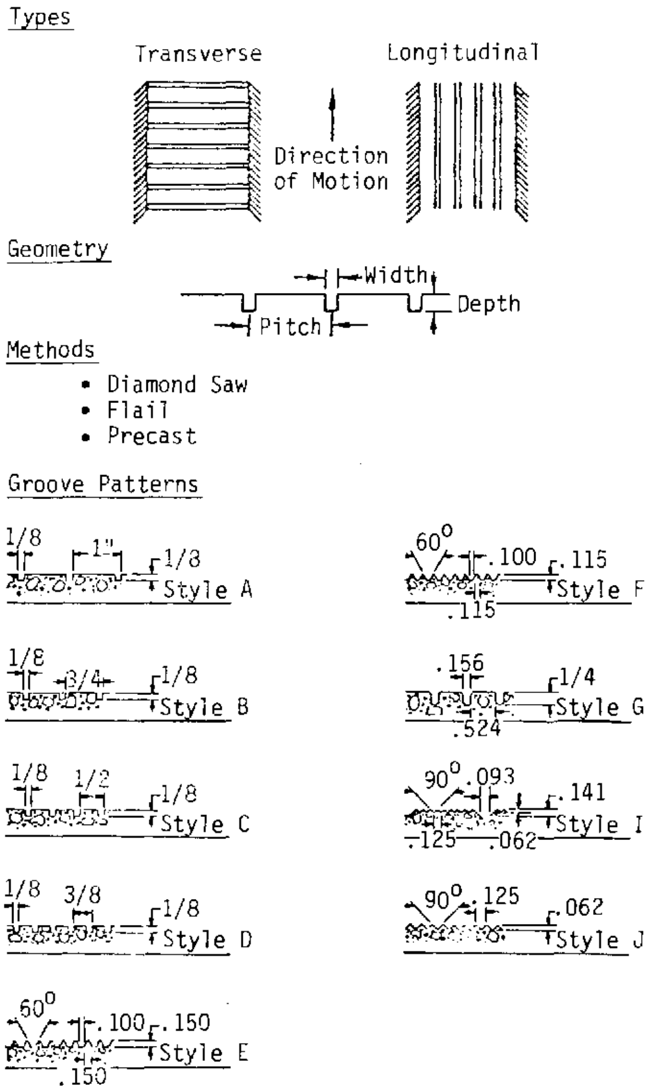


Figure 12. Pavement Groove Types, Geometry, and Patterns

SOURCE: References 16 and 17

Concerning the question of grooving direction, Beaton (18) notes all grooving on airport runways has been in a transverse direction, which, based on cross slope and water movement, etc., is the most efficient direction. Both transverse and longitudinal grooving for highways has been used with success.

Pavement grooving as a correction for wet pavement accidents has been used at many locations. Table 3 presents a summary of results from 14 pavement grooving locations in the Los Angeles, Calif., area as reported by Farnsworth (19).

Sections of the pavement on the Jones Falls Expressway (I-83) in Baltimore, Md., were grooved in the fall of 1970. As reported by Beck (20), the average daily traffic for this facility ranged from 43,000 vehicles per day in 1968 to 55,100 in 1973. In 1969, before the grooving, the average skid resistances (SN40) were 38.4 for rigid pavement longitudinal grooved sites, 41 for rigid pavement transverse grooved sites, and 45.5 for flexible sites. After grooving the skid numbers were 47.0, 49.6, and 58 for the three grooving types. In 1974, these skid numbers were reduced to 39.3, 41.0, and 55 which indicated that the skid resistance was reduced to about the same levels existing prior to grooving. The accident experience continued to hold the reduced rate through the years as shown in Table 4. The actual groove depths wore down over the years. These depths are shown in Table 5.

There have been questions about the adverse effect grooved pavement has on motorcycle safety. Smith and Elliott (21) found no indication grooving causes motorcycle accidents from a study of 23 projects totaling 322 freeway miles of grooving in California. There were five wet pavement motorcycle accidents during the before 2-year period and two during the after 2-year period on the Los Angeles grooved sections. There were a total of 114 dry pavement motorcycle accidents before grooving and 102 after. The reduction of dry pavement accidents is significant since motorcycle travel probably increased in proportion to the 14.5-percent increase in motorcycle registrations during the study period. The California study also reported grooving produced an average 69-percent reduction in wet pavement accident rates while dry pavement accidents did not change. Sideswipe and hit object accidents had the largest reductions during wet weather.

In Ohio, ramps at interchanges with I-275 in Hamilton County were longitudinally grooved in 1969 because of high wet pavement accident experience. Skid measurements were taken before and after grooving which showed the SN40 ratings on two ramps changed from 22.4 to 30.4, and from 27.4 to 37.7. The SN20 rating on a third short-radius-loop ramp changed from 30.0 to 31.4. The study, reported by Long (22), showed a 136-percent increase in dry pavement accidents and a 50-percent decrease in wet pavement accidents on the grooved sections.

Walters and Ashby (23) report a grooved concrete roadway on Interstate 12, in Baton Rouge, La., experienced a 27-percent reduction in wet weather accidents, no motorcycle accidents due to grooving, and a 12 percent or 0.013-inch loss of groove depth.

NCHRP Synthesis 14 (16) indicates that reduction in accident rates attests to the effectiveness of grooves in pavements on which water depth tends to be excessive during heavy rains. The skid numbers, as measured by locked-wheel testers with a water rate as applied by ASTM E 274, do not show a significant increase, indicating grooving is not a remedy for inadequate characteristics per se. However, lateral skid resistance is improved.

TABLE 3 - Summary of Accidents at Pavement Grooving Locations in Los Angeles

Pavement Location & Type				Grooving Pattern (Inches)			Before			After		
Route	Milepost		Pavement Type					Accidents			Accidents	
	From	To		Depth	Width	Pitch	Yrs.	Dry	Wet	Yrs.	Dry	Wet
LA - 5	78.6	78.9	PCC	1/8	1/8	1/2	2	2	7	7	32	0
Ora- 5	23.3	23.6	PCC	1/8	1/8	1/2	3	17	55	4	22	6
LA -405	2.1	2.6	PCC	1/8	1/8	1	1	10	20	4	34	1
LA -405	4.9	6.1	PCC	1/8	1/8	3/4	1	41	61	3	123	8
LA -101	0.5	0.8	PCC	1/8	1/8	3/4	1	28	23	2	42	3
LA - 5	29.5	30.5	PCC	1/8	1/8	3/4	1	10	12	3	20	5
LA -102	8.9	9.3	AC	1/4	1/4	1	1	55	139	3	116	26
LA -101*	7.7	8.9	AC	1/8	1/8	3/4	2	110	89	1	47	14
LA - 10	22.6	22.8	PCC	1/8	1/8	3/4	1	17	26	4	23	5
LA - 10	44.9	45.6	PCC	0.095	1/8	3/4	2	79	35	1.5	62	3
Ven-101	27.0	27.6	PCC	1/8	1/8	3/4	3	16	8	2	10	1
Ven-191	29.0	29.7	PCC	1/8	1/8	3/4	3	20	16	2	9	1
LA - 5	75.0	75.5	AC	1/8	1/8	3/4	3	12	14	1	3	0
Ven-101	10.9	11.2	PCC	1/8	1/8	3/4	1	3	10	2	8	3
TOTALS							25	420	515	39.5	551	76

* - Southbound only
PCC - Portland Cement Concrete
AC - Asphaltic Concrete

SOURCE: Reference 19

TABLE 4 - Wet Surface Accidents On The Jones Falls Expressway

	Before Grooving		After Grooving				
	1968	1969	1971	1972	1973	1974	1975
Grooved Sections(G)	54	57	20	31	14	25	22
Non Grooved Sections(NG)	86	62	82	118	105	55	63
Ratio G/NG	0.63	0.91	0.24	0.26	0.13	0.45	0.36

The actual groove depths wore down over the years. These depths are shown in Table 5

SOURCE: Reference 20

TABLE 5 - Average Groove Depths in Inches

	1970	1971	1972	1973	1974
Longitudinal Grooves in PCC Pavement	.123	.088	.076	.075	.066
Transverse Grooves in PCC Pavement	.093	.055	.043	.039	.034
Bridge Deck AC Pavement	.156	.106	.093	.088	.069
Bridge Apron AC Pavement	.175	.094	.065	.061	.029

PCC - Portland Cement Concrete
AC - Asphaltic Concrete

SOURCE: Reference 20

RESURFACING PAVEMENT

Resurfacing of existing pavements by the addition of new material may restore the desired wet frictional resistance. These may include the use of hot mix, hot mix cold lay, open graded hot mix, chip seal, slurry seal, and plant mixed seal. Other options include the use of epoxy resin modified binder chip seals and hot mix asphaltic concrete pavements. The additional references list includes studies related to the design and application of the many types of resurfacing. Very little safety related research has been conducted showing improved accident rates that may possibly be obtained through resurfacing.

Kamel and Gartshore (24) conducted evaluations of Ontario, Canada's "Wet Pavement Accident Reduction Program." He identified 461 highway locations including 46.1 km of pavement with an excessive rate of wet pavement collisions. Three or more wet pavement accidents and a ratio of wet-to-wet-plus-dry accidents equal to or greater than 30 percent was termed excessive.

Remedial measures used involved laying an "open friction course" on the surface of the pavement on urban freeways where no drainage problems existed. A dense friction course mixture was used on one freeway site and on all sites other than freeways. The accident experiences at eight freeway sites and five intersection resurfacing applications are shown in Table 6. For the five intersections resurfaced, the skid resistance measurements showed the average skid numbers increased from 29 to 45 after resurfacing.

Blackburn et al. (7) studied resurfacing of 130 test sections located in 12 States. The mean skid number of these test sections was 48.64 before resurfacing and 47.57 after. The reasons for resurfacing these sections were not necessarily because of low skid numbers, but primarily to maintain the structural integrity of the pavement and/or to improve the rideability. No significant before-after effects were found for wet-pavement accident rates for the resurfaced sections.

TABLE 6 - Ontario's Wet Accident Reduction Program Results

			Wet Accidents			Total Accidents		
Type Roadway	Number Sites	Years	Before	After	Change	Before	After	Change
Freeways	8	1976-78	257	118	-54%	742	524	-29%
Intersections	5	1977-78	35	10	-71%	71	38	-46%

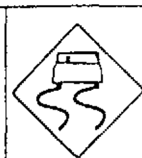
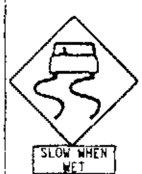
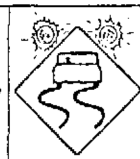
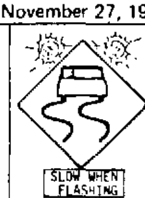
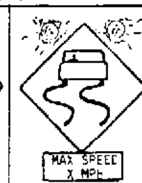
SOURCE: Reference 24

SKID WARNING SIGNS AND LIABILITY

Hanscom (25) reviewed the highway agency concern regarding the presence of skid warning signs increasing susceptibility to liability suits. A search for legal cases addressing this issue found most documented legal opinions approve the use of signs as an interim warning for slippery conditions, in liability suits. The warning signs, however, should not be considered as a permanent remedy. Cost, personnel requirements, time, and other constraints are recognized as preventing the best practices from being put into effect in all circumstances. The proper placement of the symbolic "Slippery When Wet" sign with the possible addition of an advisory speed plate provides adequate warning to the motoring public. See Manual on Uniform Traffic Control Devices, W8-5 and W13-1 (26).

Hanscom (25) treated three curved highway sections with five experimental signing conditions. Comparisons between "all signs" and "no signs" conditions were made for wet and dry pavements. Table 7 is typical of the results showing the highest quartile speed group (fastest 25 percent) of vehicles arriving at the advance speed sign. Note the little difference in speed between "no sign" and the use of the "symbolic sign" alone. Improved responses were obtained for the high levels of conspicuity and specificity obtained by the addition of flashing lights and the advisory speed plate. The sign conditions with the flashing lights were effective in reducing highest quartile mean speeds below the critical safe wet pavement speed based on roadway geometry and surface condition. Questionnaire results indicated 60 percent of the interviewed motorists saw and properly interpreted the more conspicuous warning signs. No accident studies have been found in the literature related to the placement of these warning signs.

TABLE 7 - Differences in Highest Quartile Speeds Between Normal, No Signing, Dry Pavement Conditions and Experimental Signing, Wet Pavement Conditions at Site 1

	November 5, 1973				November 27, 1973		
	No Sign				No Sign		
200' Advance	1.8	.6	1.6	4.8	2.3	5.7	7.1
Enter Curve	1.2	1.4	3.2	4.9	3.1	6.8	7.8
Tight Curvature	1.3	2.3	3.7	4.3	3.1	5.4	6.4
Leave Curve	2.7	1.9	4.6	4.5	3.0	5.9	7.1

SOURCE: Reference 25

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CHAPTER 3 - ROADSIDE FEATURES

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INTRODUCTION

Roadside accidents are a serious safety problem. Of the 45,271 fatal accidents resulting in 51,077 fatalities in 1980, approximately 40 percent occurred off the roadway width (1). Excluding pedestrian fatalities, there were 21,531 fatalities resulting from fixed-object and rollover accidents. Drivers sometimes leave the roadway unintentionally for various reasons (driver error, falling asleep, intoxicated, etc.) and are thus subjected to roadside hazards. Although the run-off-the-road maneuver may be unavoidable, the penalty should not be death or serious injury. Drivers also become unintentionally involved in on-roadway mishaps sometimes for the same reasons as those drivers who leave the road. In some of these mishaps they may be able to regain vehicle control but in others they may not. Considerable research has been devoted in the last two decades to developing roadside designs and systems that maximize highway safety and provide a "forgiving highway." The purpose of this synthesis chapter is to describe (1) the extent and nature of roadside accidents, and (2) those approaches which have helped to reduce the fatality, injury, and property damage accidents both on and off the roadway.

Extent of Problem

The development and implementaton of roadside safety features is no easy task. The 3.2 million mile highway system is spread across all kinds of topographic and environmental features and various kinds of functional roadway systems. The kinds of roadside objects that can be hit are numerous. For example, Table 1 identifies 16 objects or features which were associated with the 12,929 run-off-the-road fatalities found in the 1981 Fatal Accident Reporting Systems (FARS) study (2). These fatalities are for single vehicle fixed object and rollover accidents excluding pedestrians.

Review of Table 1 shows that the distribution of fatal accidents varies according to the functional system of highways. This is not surprising since the geometric design and type of roadside features vary by the type of functional system. For example, a much lower percentage of fatal accidents resulting from collisions with trees or utility poles would be expected on the interstate system than on other systems because of its clear roadside requirements. However, fewer guardrail fatal collisions would be expected on local roads than arterial systems because guardrail is not as prevalent on local roads. As can be seen from Table 1, the types of objects or features that can sometimes result

TABLE 1 - Percent Distribution of 1981 Run-Off-the-Road Fatal Accidents by Functional System

Type of Fatal Accident	Functional Systems				
	Interstate	Arterials	Collectors	Local Roads	Total
1. Overturn	56.7	41.9	46.5	39.6	42.3
2. Tree/Shrubbery	4.4	19.5	23.5	27.1	20.9
3. Utility Pole	1.4	11.5	7.4	10.5	9.9
4. Embankment	3.4	4.1	5.2	3.7	4.2
5. Culvert/Ditch	3.1	3.7	5.0	3.5	4.2
6. Guardrail	12.8	3.9	2.1	1.1	3.7
7. Bridge-Passing Over	3.5	2.5	3.6	2.7	3.0
8. Other Fixed Objects	1.5	1.9	2.0	2.9	2.2
9. Curb or Wall	2.4	2.2	0.6	2.5	2.0
10. Building	0	1.3	1.0	1.7	1.2
11. Bridge-Passing Under	4.2	1.6	0.5	0.8	1.4
12. Light Support	1.4	2.1	0.1	0.6	1.2
13. Fence	0.6	1.0	1.1	1.5	1.1
14. Sign Post	2.1	1.1	0.6	0.9	1.1
15. Other Poles/Support	0.4	0.9	0.8	0.8	0.9
16. Divider	2.2	0.8	0	0.2	0.7
TOTAL					
Percent	100.0	100.0	100.0	100.0	100.0
Accidents	1267	5378	2719	2570	12929

SOURCE: Reference 2

in fatal accidents when hit are numerous. The presence of some of these objects can be minimized or eliminated from the roadside environment. When a potentially dangerous object cannot be removed from the roadside then such objects need to be designed or placed so they will not inflict needless injury to persons and damage to vehicles colliding with them. The severity of accidents resulting from vehicle collisions with roadside objects is not always the same for the same kind of objects. Vehicle size also has a direct impact on system performance and expected severity levels (3). Small, lightweight passenger cars perform differently from heavier vehicles. Severe vehicle damage, rollover, and snagged support posts are examples of crash properties experienced by mini-sized vehicles. In recent years the weight differential between cars and trucks has widened and according to Figure 1 it will continue to do so (4).

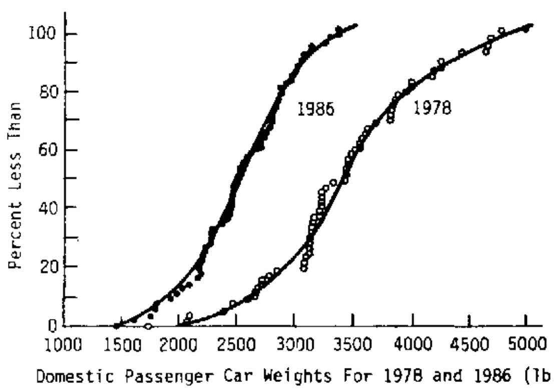


Figure 1. Domestic Passenger Car Weights For Model Years 1978 (Actual) and 1986 (Projected)

SOURCE: Reference 4

Research Approach

Solutions to minimize or reduce the effects of roadside hazards have been and are being studied. Accident studies, full-scale crash tests, and simulation have all been used to examine the hazard (or safety) of particular roadside features and objects. Crash tests and simulation studies are useful in examining the relative safety effectiveness of various features and objects. Accident studies will determine their absolute effectiveness but are unsuitable for detecting small or subtle effects. Accident studies, full-scale crash tests, and simulation all have advantages and limitations. Special accident studies are expensive, and it is difficult to obtain the level of detail necessary to answer questions beyond the overall hazard of a feature or object. The relative hazard of roadside features must be defined in terms of measure of accident exposure or opportunity. While full-scale crash tests can be carefully controlled their expense limits the various combinations of vehicle type, speed, and angle that can be examined (5, 6). Simulation is relatively inexpensive and, like crash tests, may detect small difference, but it does not totally account for the complexity of the real world (7 - 20). In discussing the various roadside problems and types of solutions each of these three study approaches (accident studies, full-scale crash tests, and simulation) are at times discussed in this synthesis.

Treatment Approach

The approaches typically used in eliminating or reducing roadside safety problems are to eliminate the hazard, relocate the hazard, make the hazard breakaway, and redirect or attentuate the vehicle (21).

An initial priority is to eliminate as many hazards as possible. Examples are the elimination of unnecessary signs, flattening of roadside slopes, and modifying drainage facilities to remove culvert headwalls or tabletop drainage inlets. If the hazard cannot be eliminated, then it may be possible to locate it longitudinally or laterally where the likelihood of a fixed-object collision will be minimized. An example would be the placement of signs on overpasses or behind protective traffic barriers. If a fixed-object hazard cannot be eliminated or relocated, then the use of a breakaway technique should be considered. This applies to such items as sign supports and luminaire supports.

Many hazards along a roadway cannot be eliminated, relocated, or made breakaway. Examples are steep slopes, natural streams, rockface cuts, opposing traffic, bridge piers, elevated gores, and bridges. Hazards which cannot be removed are shielded by traffic barriers to intercept and redirect or attenuate out-of-control vehicles.

GEOMETRIC AND CROSS SECTION FEATURES

Clear Zones

A number of studies have sought to investigate the characteristics of the roadside accident and to determine the limits of a recovery area that should be provided to prevent or minimize this type of accident. Pioneering studies of roadside encroachments and accidents were conducted by General Motors (22). Due to the abundance of driving activity on the roadways of the General Motors Proving Grounds, it was possible to compile data on run-off-the-road accidents in conjunction with normal Proving Ground activities.

The researchers compiled data on the distribution of lateral and angular vehicle encroachments. A brief review of the 1963 Proving Ground analysis was recently reported by Jones et al. (23). Based largely on these data, 98 feet of clear roadside is now a Proving Ground standard. Jones et al. (23) also reported the findings of Skeels (24) that demonstrated elimination of serious driver injury after roadside improvements have been made. American Association of State Highway and Transportation Officials (AASHTO) states that studies indicate 80 percent of the vehicles leaving high speed highways out of control can recover less than 30 feet from the pavement edge (25).

Huelke and Gikas made a study (26) of 111 fatal automobile accidents in and about Washtenaw County, Mich. They found that 67 or 60 percent of the accidents were single car, off-road collisions occurring near the roadway. Single vehicle accidents along the roadside were also studied by the Northwestern University Traffic Institute (27). Approximately 80 percent of 939 off-road, fixed-object, and overturned vehicle accidents studied on U.S. Route 676 were of the "single vehicle" type. Review of Table 1 shows that about 50 percent of single vehicle accidents result in overturns which are typically more severe than non-rollover accidents. As shown in Table 2, Hosea (28) also found a similar percentage of overturn accidents on completed sections of interstate roads.

Once the vehicle departs from the roadway the potential for collision with a roadside object logically increases. Maximizing the clear zone width is therefore considered a viable improvement. Since any real clear zone is likely to start at the road edge, Perchonok, et al. (29) calculated the percentage of vehicles getting away per clear zone width. Figure 2 depicts their findings. The initial intercept starts at zero feet with 17.7 percent of these vehicles getting away without an incident with no clear zone. In considering the effectiveness of clear zones, it must be kept in mind that vehicles departing from the roadway tend to travel until they hit something. Also, ostensibly clear zones sometimes have features which can induce rollover accidents.

A study conducted by Wright and Robertson (30) analyzed more than 300 fatal accidents in Georgia which involved roadside objects to determine correlating conditions within 528 feet of the collision site. It was found that over one-half of the collisions with roadside objects occurred at or near horizontal curves greater than six degrees. The study also reported that 98 percent of the objects struck were within 50 feet of the pavement edge. Hall et al. (31) also studied the nature of single vehicle accidents involving fixed objects along the roadside and found that these accidents occurred most frequently during darkness and/or adverse weather, on poor pavement, and on horizontal curves. An Australian study (32) of roadsides concluded that maintaining a clear recovery area of at least 30 feet would permit a large majority of the vehicles to leave the roadway and recover safely.

TABLE 2 - Characteristics of Off-the-Road Fatal Accidents on Completed Sections of the Interstate System

Type of Accident	No.	%
Total Accidents, All Types	1462	100.0
Struck Fixed Object:		
Total	1208	82.6
Overturned	480	32.8
Overturned Only	245	16.8
All Overturns	725	49.6
Off-the-Road Only	9	0.6

SOURCE: Reference 28

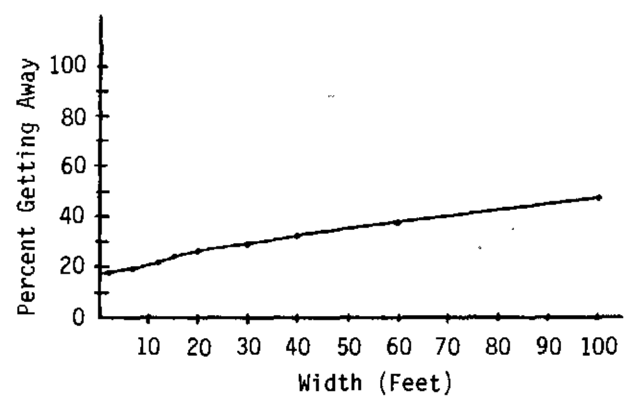


Figure 2. Percent of All Departing Vehicles Getting Away as a Function of Clear Zone Width

SOURCE: Reference 64

Hutchinson and Kennedy (33) studied the problem of roadway departures; they investigated vehicle encroachment into median areas and developed distributions for angular departure from the roadway, as shown in Figure 3. Based upon these findings less than 10 percent of the vehicles leave the roadway at an angle greater than 25 degrees. Garrett and Tharp (34) also developed a similar distribution. Perchonok et al. (29) further documented the characteristics of road departures in a study of 8,000 accidents on rural roads. Although departure has been traditionally characterized in terms of the departure angle and speed, the analyses in their study (29) is characterized in terms of departure angle and the departure attitude. For each departure the attitude of the vehicle was recorded as "tracking" (rear wheels in line with front wheels) or "not tracking." Since it is generally considered that a nontracking vehicle is out of the driver's control, departure attitude was considered as an indication of loss of control, and nontracking vehicles were more likely to rollover than were tracking vehicles. The major findings are summarized as follows:

- Overall, right side departures were more prevalent than left side departures. Left side departures involved larger proportions of nontracking vehicles and larger departure angles than did right side departures. The overall mean departure angles for right and left departures were 13.5 degrees and 18.6 degrees, respectively.

- Overall, approximately 70 percent of the accident vehicles were tracking at the point of departure. The overall mean departure angle associated with tracking vehicles was 14.3 degrees and 22.8 degrees with nontracking vehicles.

- The proportion of departures to the outside of curves increased with degree of curvature.

- With regard to point of departure along a horizontal curve, departures at the very end of curves were overrepresented. This was true only for shorter curves on undivided roads, thereby suggesting problems originating at the beginning of the curve.

- Seventy-four percent of the sampled accidents involved only a single departure. When more than one departure was involved, the most frequent configuration was a double departure, with the vehicle departing once from each side of the road.

Perchonok et al. (29) reviewed accidents on divided and undivided roadways according to horizontal alinement. As shown by Table 3, on undivided highways approximately 44 percent of the accidents occurred on horizontal curves. As curves undoubtedly represent less than 44 percent of the roads in the study, the accident

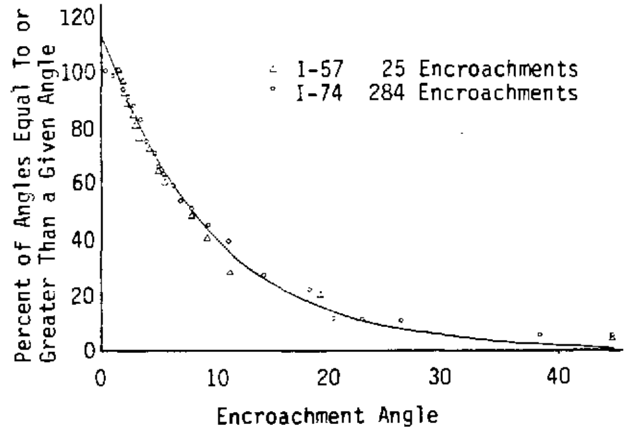


Figure 3. Distribution of Encroachment Angles

SOURCE: Reference 33

rate was higher on curves than on tangent sections. The data also show that on undivided roads there were more left curve accidents than right curve accidents. Because it can be assumed that left and right curves experience equal vehicular travel, this implies higher accident rates on left curves. Figure 4 shows the departure locations by horizontal alinement on undivided roads.

TABLE 3 - Accident Frequencies for Horizontal Alinement

	Undivided Road		Divided Road	
Horizontal Alinement	Number of Accidents	%	Number of Accidents	%
Tangent	3,663	56	847	76
Left Curve	1,751	27	111	10
Right Curve	1,089	17	151	14
Total	6,503	100	1,109	100

SOURCE: Reference 29

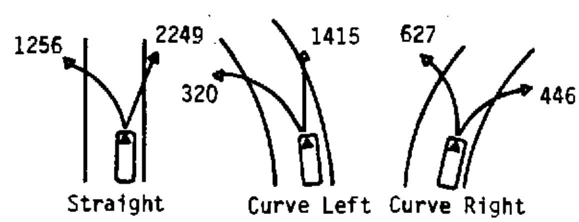


Figure 4. Departure Location by Horizontal Alinement Frequencies (Undivided Roads)

SOURCE: Reference 29

Perchonok et al. (29) also found there was a pronounced tendency for vehicles to depart the right side of the road. A reasonable explanation is that if a vehicle leaves the travel lane to the left, the adjacent lane often provides room for recovery. Results also show nearly three-fourths of single vehicle accidents on curves involved departure on the outside of the curve, which means that vehicles tended to not turn enough rather than turn too much.

The effect of alinement on accident occurrence was also studied by computing accident frequencies for equally spaced intervals after curves. Table 4 shows a peak accident frequency immediately after horizontal curves followed by decreasing accidents downstream. This phenomenon can be explained by accidents occurring after vehicles leave a curve as a result of problems originating on the curve or in transition from curve to tangent section. A similar effect is noted for vertical curves in Table 5.

TABLE 4 - Accident Frequency in Relation to Distance from Previous Horizontal Curve

Distance From Curve, Feet	Number of Accidents	Percent
0 - 200	457	34
201 - 400	416	31
401 - 600	214	16
601 - 800	149	11
801 - 1000	112	8
Total	1,348	100

SOURCE: Reference 29

TABLE 5 - Accident Frequency in Relation to Distance from Previous Vertical Curve

Distance from Curve, Feet	Number of Accidents	Percent
0 - 200	514	32
201 - 400	348	22
401 - 600	302	19
601 - 800	239	15
801 - 1000	193	12
Total	1,596	100

SOURCE: Reference 29

Perchonok et al. (29) showed in Table 6 that downgrades (tangents and crests) were overrepresented as accident sites. Because upgrades should have as much vehicular traffic as downgrades, the accident rate for downgrades is 63 percent higher than for upgrades. Combinations of vertical and horizontal alinements were examined and, not surprisingly, left curves on downgrades were overrepresented as accident sites as shown in Table 7.

Some associations between alinement and injury were also found. Table 6 shows percent injured was lowest for level roads. For vertical curves percent injured was higher for drivers having accidents traveling down than traveling up the curve. In respect to horizontal alinement, Table 7 shows percent injured was significantly higher for accidents on curve rather than tangent sections, particularly on left curves.

TABLE 6 - Accident Frequency and Severity by Vertical Alinement

Vertical Alinement	Number of Accidents	Percent Accidents	Percent Injured	Percent Killed
Level	1,951	34	54	5
Downgrade	1,503	26	58	5
Upgrade	937	16	56	4
Down on Crest	457	8	63	6
Up on Crest	368	7	60	6
Up on Sag	256	5	58	6
Down on Sag	206	4	62	7
TOTAL	5,678	100		

SOURCE: Reference 29

TABLE 7 - Severity of Injury by Horizontal Alinement

Alinement	Injury Type: None	Nonfatal	Fatal	Total	Percent Injured	Percent Killed
Tangent	2,008	2,259	203	4,470	55	5
Left Curve	715	1,017	107	1,839	61	6
Right Curve	523	652	64	1,239	58	5
Total	3,246	3,928	374	7,548	57	5

SOURCE: Reference 29

Roadfill, Side Slopes and Ditches

Roadside geometry influences the behavior of a vehicle when it leaves the roadway. Vehicle behavior is important in that (1) injury rates are much higher for rollovers than for nonroll impacts, and (2) no impact in the first road departure is a necessary condition for the avoidance of any impact at all.

Perchonok et al. (29) found rollovers are more likely to occur from accidents on roads built on fill than in cuts. Among nonroll impacts, it has been found that ditches, embankments, and culverts are overrepresented. Increased height of fill and depth of ditch are conducive to rollovers. Rollover rates begin to increase when fill exceeds 2 feet, and reach a plateau for fills greater than or equal to 4 feet. Rollover rates jump markedly for ditches 4 feet to 5 feet deep, but beyond 5 feet rollovers decrease as nonroll impacts with ditches increase.

The slope of fill and ditches primarily affects the proportion of departures having no impact. For fill, the increase in nonroll impacts appears as a step function with the increase occurring for slopes steeper than 3:1. The increase in rollover and nonroll impacts occurs in two steps--one increase for slopes steeper than 4:1, and another for slopes steeper than 2:1.

For ditches, the decreasing proportion of rollovers corresponds with the initial increase in nonroll impacts for slopes steeper than 4:1. Reduction in nonimpact departures does not occur until the ditch slope exceeds 3:1.

A separate analysis of ditch depth and injury showed a 20-percent higher injury rate for deep ditches. Ditches over 2-feet deep were both struck more often and conducive to a greater likelihood of injurys. It was also shown that part of the increased injury rate associated with accidents on roads with deep ditches was due to higher impact speeds. In comparing injury experience for roads built on fill and in cuts, some similarities were found by Perchonok et al. (29). On fill sections, as the slope became steeper or the fill higher, the injury rate increased. For cut roadways, the injury was small for shallow ditches; it increased in the middle range, then dropped down, and increased again for the deepest ditches. The authors (29) indicate that both slope and depth, therefore, had real effects on injury rates.

Three regions of the roadside are particularly important when evaluating its safety aspects: the top of the slope (hinge point), the front slope, and the toe of the slope (intersection of the front slope with the ditch or back slope). A study (35) sponsored by the National Cooperative Highway Research Program (NCHRP) addressed these regions and developed design criteria for roadside geometrics. Effects of rounding the hinge point and the ditch bottom have also been studied (36). It was concluded that rounding will enhance roadside safety by affording an errant motorist more control in terms of steering and braking.

In a more recent study, Graham and Harwood (37) studied single-vehicle run-off-the-road accidents relative to three clear zone policies, namely 6:1 clear zone, 4:1 clear zone, and nonclear zone. Highways constructed under the 6:1 policy typically have foreslopes of 6:1 or flatter within 30 feet of the traveled way. For the 4:1 policy, the foreslopes are typically 4:1 or flatter within 30 feet of the traveled way. In the nonclear zone policy, the slopes are typically dominated by sections with 3:1 and 2:1 embankment slopes and with little or no control of unprotected fixed objects. Table 8 and Figure 5 show the results of their study.

Curbs

Curbs are used in highway design to control drainage, deter vehicles, delineate the edge of the roadway, present a finished appearance, and aid in orderly roadside development. Curbs are usually classified as barrier or mountable. Barrier curbs are designed to inhibit drivers from leaving the roadway but have limited redirective capabilities. A considerable amount of research has been conducted on the design and use of curbs as they affect traffic safety.

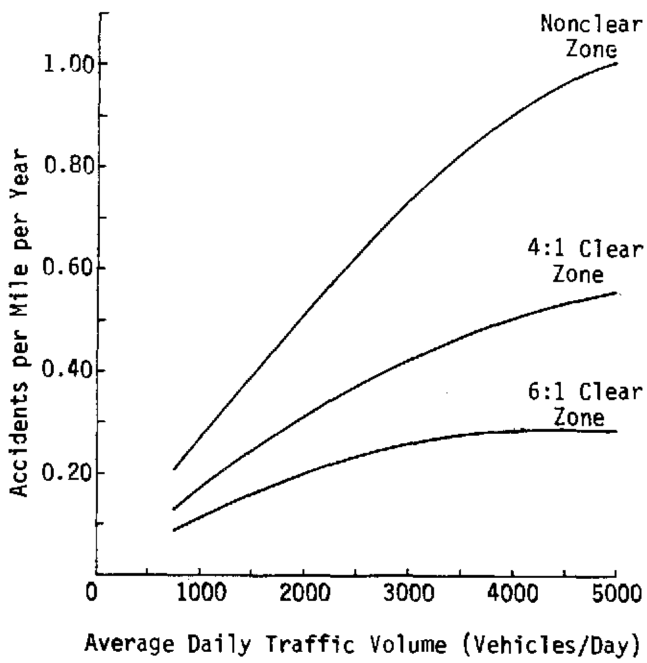


Figure 5. Relationship Between Single-Vehicle Run-Off-Road Accidents per Mile per Year and ADT for Two-Lane Highways

SOURCE: Reference 37

TABLE 8 - Adjusted Mean Single-Vehicle Run-off the-Road Accident Rate per 100 Million Vehicle Miles by Highway Types and Roadside Policy

Highway Type	Roadside Policy			Policy Differences	
	6:1	4:1	NCZ1/	6:1 vs 4:1	4:1 vs NCZ
Two-Lane	25.4	40.3	68.0	Sig.2/	Sig.
Freeway	18.2	28.9	40.73/	Sig.	---
Four-Lane Divided (Nonfreeway)	15.5	31.9	60.7	Sig.	Sig.

1/ Nonclear Zone
2/ Statistical Significance at 95 Percent Confidence Level
3/ Estimated

SOURCE: Reference 37

The California Division of Highways conducted some of the initial research (38) on curb mounting and redirection. This study consisted of 149 full-scale impact tests on the 11 types of curb sections. As a result of these tests, four basic designs were developed for further testing (39). Conclusions from these two test series were that an efficient barrier curb should be at least 10 inches high, be undercut, and have a moderately smooth surface texture.

Research on vehicle-curb impacts demonstrates the inability of a curb to redirect vehicles and raises the question as to the basic function of the curb on some roadways. A series of field tests in Washington (40) showed that a mountable curb on a median did not produce redirection of a speeding vehicle. This finding was also substantiated by full-scale tests conducted in California (41).

An evaluation of the curbs was also conducted in a research study (42) sponsored by NCHRP. The research approach utilized a combination of simulated impacts and full-scale testing. It was found that at low to moderate speeds and impact angles, the curb designs offered little path redirection. The vehicle's trajectory after curb impact was also analyzed and significant ramping problems were observed. The vehicle attitude after impact could influence the severity of a secondary impact with a traffic barrier or breakaway support.

Conclusions drawn from the NCHRP study are summarized as follows:

- Curbs offer no safety benefit on high-speed highways from the standpoint of vehicle behavior following impact.
- Omission of curbs along high-speed highways will enhance safety.
- Curbs may be desirable for drainage, but this can be achieved in other ways on high-speed facilities.
- When barriers are required to protect an errant vehicle, a full height barrier should be considered, such as the configuration employed in the New Jersey concrete median barrier.

An extensive study of barrier curbs was conducted at Wayne State University (43) for the Michigan State Highway Commission (MSHC). The results of this study are in basic agreement with the NCHRP study. Of five MSHC curbs tested, only the higher curbs had a significant influence on vehicle path with the greatest effect being noted in low speed, low angle cases. The Elsholz curb developed in West Germany and modified from an earlier California Division of Highways design, was found to be the most effective in redirecting vehicles. This curb is undercut and 10.6 inches high. Beaton and Peterson (38) and Dunlap (44, 45) have also studied curb redirection and conducted vaulting analyses.

Escape Ramps

Escape lanes or ramps have been tested and constructed on two-lane and multilane highways so that runaway vehicles, mostly trucks, on long, steep grades can stop safely (46, 47). Baldwin (48) described the use of a 13-foot-wide, 2,480-foot-long escape lane in Utah which utilized 12 inches of pea gravel to slow and stop vehicles. Fifteen vehicles successfully used the lane during the first 15 months of operation.

Erickson (49) described the experience of Colorado with runaway vehicles. Between 1976 and 1979, there had been 152 truck accidents on grades. Fifteen people were killed and another 81 people were injured. The total economic loss was over $5 million. During this period, Colorado built six escape ramps. The most successful ramp showed a 400-percent reduction in accidents and a benefit-cost ratio of 10:1. One of the vehicles was a school bus with 33 passengers which entered the ramp at 60 mph. No vehicle occupants were injured.

OBJECTS OFF ROADWAY

Once vehicles leave the roadway, they are susceptible to hitting various kinds of objects, such as trees, buildings, fences, signposts, utility poles, luminaire supports, drainage facilities, bridge abutments, etc. Impacts with different kinds of objects vary in terms of severity as measured by fatalities, injuries, and property damage accidents. As shown by Table 1, the percent of fatalities varies with type of functional system. Outside of overturning accidents, the highest percentages of fatalities occur when vehicles collide with trees, utility poles, embankments, culverts/ditches, and guardrails. Gennarelli (50) reported the use of a severity injury scale known as Abbreviated Injury Scale (AIS). Weaver et al. developed a severity index and related it to accident cost (51).

Sign Supports

It is necessary to supply information to drivers through the use of roadside signs. Sign supports must be recognized as fixed roadside hazards and either located, designed, or protected consistent with recognized safety standards. There are three types of signs: roadside, overhead, and structure-mounted. Roadside signs should be designed to "breakaway" or "yield" when struck by a vehicle. Structures for the overhead signs are either located a safe distance from the travel way or shielded by a traffic barrier. Structure-mounted signs present no safety problems

SMALL SINGLE-POST SIGNS

According to Ross (52) the most widely used designs, in order of use, are (1) steel U-posts, (2) wood posts, (3) standard steel pipe, and (4) square steel tubing. Breakaway bases are used on a small percentage of the total small sign installations.

In the past, small single-post sign installations were not a significant hazard because large vehicles made up the majority of the traffic stream. The trend, however, is to smaller vehicles for economy and fuel efficiency and even the small sign installations pose a significant hazard to small-vehicle occupants. In high speed impacts, a 1,940-lb. subcompact automobile was found to sustain a change in momentum 13 percent higher than a 2,270-lb. compact vehicle. The change in velocity was 33 percent higher. Ross summarized the crash test performance of widely used single support systems, including steel U-posts, flanged channels, wood posts, steel tubing, aluminum posts, and steel W-sections with breakaway slip bases.

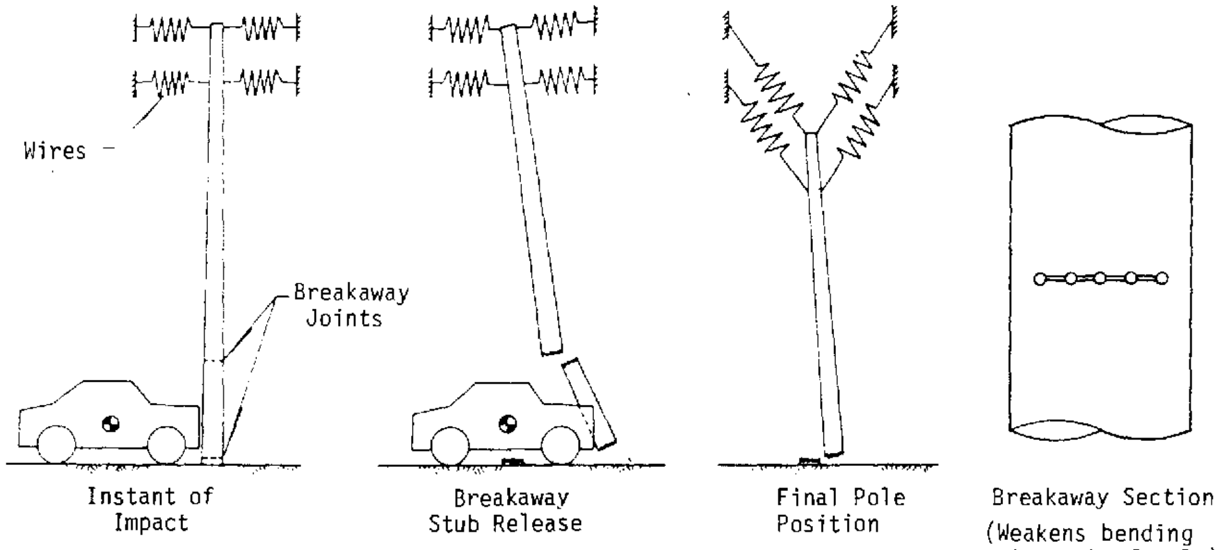
These systems have been evaluated in terms of current safety performance criteria (53) and guidelines (6), and found to be satisfactory for single-post installations (52). Research reports on small single-post sign systems are available in references 54 through 62.

BREAKAWAY DESIGNS

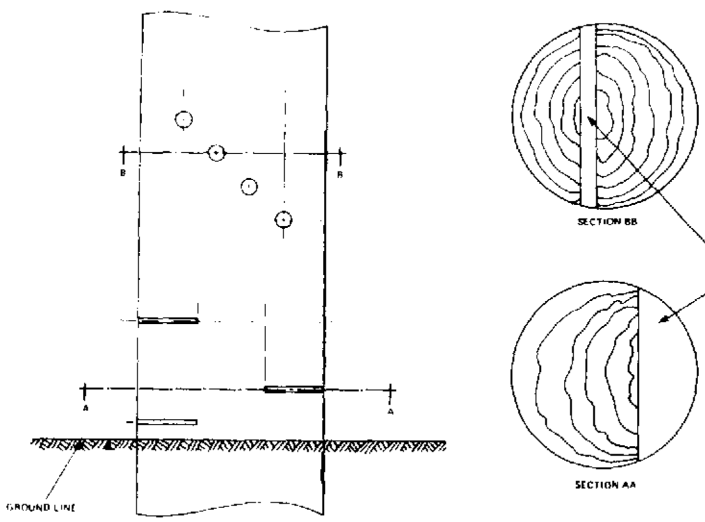
Initial research (63, 64) on the breakaway design concept was aimed at the large roadside sign with two or more support posts. The objective of reducing collision severity considered the characteristics of mass, structural rigidity, and connection at the base of support. Full-scale crash tests indicated that a slip joint at the base of the sign support would function satisfactorily but there was a need for the post to swing up while the vehicle passed under the support. Problems associated with the broken post section impacting with the windshield were eliminated by leaving the back flange intact to function as a hinge. The desired collision behavior is shown in Figure 6.

The initial crash studies investigated roadside signs in which the posts were so widely spaced that it was physically impossible for a vehicle to collide head-on with more than one support. There are a large number of signs, however, which are small enough that a vehicle can collide with both sign supports. Further research (65, 66) involved crash tests on 5 feet x 6 feet plywood signs supported by two W 5 x 16 beam posts 3-1/2 feet apart. Nineteen full scale crash tests were performed on the small signs employing the slip base and hinge joint features of the larger signs. One special feature, an inclined base plate, forced the sign up and over the vehicle. This feature provided better performance in slow speed collisions with both supports.

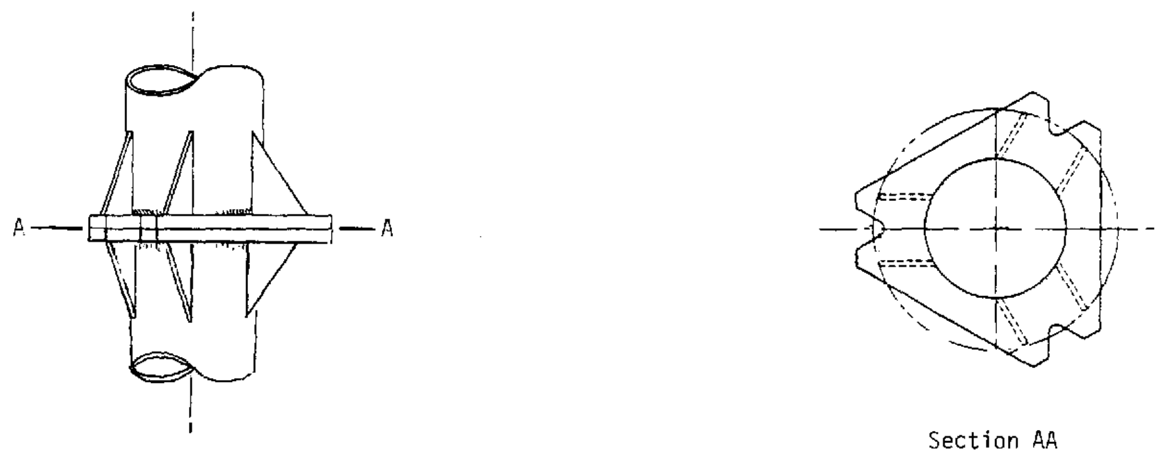
Studies were also made of standard galvanized steel pipe ranging in diameter from 2 inches to 5 inches. Initial studies utilized the slip base and hinge design but the crash tests proved that an inclined slip base would perform satisfactorily with a single-post, steel pipe sign installation.



Breakaway Stub



RETROFIX Design (Not Recommended)



SLIPBASE Design

Figure 6. Utility Breakaway Treatments That Have Been Crash Tested

SOURCE: References 75 and 78

The success of the research efforts on breakaway sign supports generated national interest and a pooled fund research project was undertaken. Further research (67, 68, 69), entitled "Highway Sign Support Research," involved an extensive research effort to address the nationwide use of the breakaway sign concept. The research studies consisted of laboratory tests, full-scale crash tests, and the use of a mathematical model. The multi-state research study developed the necesssary criteria that would permit highway designers to design adequate breakaway sign supports for application anywhere in the United States.

The success of the breakaway sign support is demonstrated by an analysis of 82 accidents in Texas that involved breakaway supports (67).

- In 43 of the accidents, the damage was so slight that the vehicle did not remain at the scene.
- In 38 of the accidents where the vehicle remained at the scene, there were eight cases of minor injury (bruises or complaint of pain).
- Only one accident had a serious injury, and in this accident the vehicle struck a culvert headwall after passing through the sign support.

Utility Poles

In 1980, the FARS listed 1,775 fatal accidents involving utility poles as the first harmful event. Table 1 indicates that impacts with utility poles account for approximately 10 percent of the run-off-the-road fixed-object traffic fatalities and approximately 4 percent of the national traffic fatalities. The severity of utility pole accidents is further confirmed in that about 50 percent of all utility pole accidents are injury accidents (70).

According to Mak and Mason (71) there is a 50-50 chance that an occupant in a utility pole accident will sustain some form of injury, AIS ≥ 1, even at a very low impact speed of 6 mph (velocity change of 4.7 mph or momentum change of 735 lb-sec). Injury rates for severe to fatal injuries of AIS ≥ 3 are minimal for low to medium accident severity; e. g., the probability is less than 10 percent for impact speeds below 32 mph (29 mph velocity change or 4,450 lb-sec momentum change), increasing to 50 percent at impact speeds of 50 mph (47 mph velocity change or 6,700 lb-sec momentum change). Smaller and lighter cars involved in collisions with timber utility poles and nonbreakaway luminaires are more likely to have higher resultant injury frequency and severity than their larger and heavier counterparts. The effect is much less evident in the case of breakaway luminaire impacts.

McCoy et al. (72) report on a methodology for evaluating safety improvement alternatives for utility poles. Using a total annual cost method of economic analysis, several types of alternative improvements were compared. The evaluation included multiple use of poles, relocation of poles, breakaway poles, impact attenuation systems, and placing utility lines underground. In all cases, the cited existing condition had the highest annual accident cost, and the underground alternative had the lowest. Although only one vehicle size, one utility pole spacing, and one other type of fixed object (nonbreakaway) were considered, the methodology demonstrates the applicability of relative economy of improvement alternatives.

According to Mak and Mason (71) collisions with timber utility poles have the highest frequency of severe to fatal injuries (7.4 percent), followed by nonbreakaway (4.9 percent) and breakaway (3.8 percent) luminaires. Accidents involving signs, both breakaway and nonbreakaway, result in very low injury frequency and severity. In terms of overall injury, nonbreakaway luminaires (72.4 percent) and timber utility poles (66.8 percent) have the highest frequencies while collisions with other pole types result in smaller frequencies (≤ 40 percent) of overall injury.

RELOCATION

Conflicts between vehicles and utility poles were examined by Jones (73) in a before and after study of a 2-mile section of a four-lane major arterial. In the before condition, many utility facilities were close to moving traffic. Some of the utilities were relocated under the sidewalk and all the utility poles relocated to the back of sidewalk on one side of the street. Accident records showed a disproportionately high number of traffic accidents with utility poles over a 6-year period (42). After relocation was completed, no collision with utility facilities had been reported in a 5-year period, even though the average daily traffic had almost doubled.

Nemec (41) describes how negotiation and coordination resulted in the elimination of 100 poles and 40 overhead crossings in a 3-mile section of major arterial highway. Since this section of roadway was being widened from four to six lanes, elimination and relocation of utility poles and facilities was considered for safety, economic, and beautification reasons. Accident analyses are not reported, but over $2 million in savings was realized due to proper planning, design, and installation of the utilities and roadway.

Jones and Baum (70) studied over 8,000 single vehicle accidents in 1975. They found the proportion of utility pole accidents decreased with pole offset distance, and 74 percent of all poles hit were within 10 feet of the road edge. Half of all utility pole accidents were within 4 feet of the road edge. They also found the proportion of utility pole accidents decreased with pole spacing.

BREAKAWAY DESIGNS

In many cases the utility pole is an example of an obstacle that cannot be relocated easily. The severity of impact can be reduced by making modifications (retrofitting) to the obstacle in place. However, these modifications can reduce the bending strength of the utility pole and its ability to resist wind and ice loads. Wolfe et al. (74), Post et al. (75), and others (76 and 77) investigated the feasibility of breakaway utility poles. Figure 6 shows examples of utility pole breakaway treatments that have been crash tested. Post et al. (75) tested the feasibility of using two breakaway joints in 40-foot class 4 Southern Pine utility poles located at ground level and at 7-to 8-feet above ground level. The breakaway joints are made by drilling a set of five 1-inch diameter holes and saw-cutting the spaces between the holes. These holes significantly reduce the bending strength of the pole, particularly in the service line direction. Upon impact, the short 7-to 8-foot section of pole will breakaway thereby helping to protect the vehicle occupants from severe injury. Post et al. (75) concluded the breakaway concept is very cost effective for utility poles within 10 feet of the roadway.

Labra and Kimball (78) investigated the breakaway potential of over 20 conceptual timber utility pole breakaway designs. Of all the concepts evaluated, they recommended two retrofit designs: (1) a slip base design known as SLIPBASE, and (2) a bore hole, saw-cut concept, called RETROFIX. According to their research both concepts reduce the inherent roadside hazards associated with in situ timber utility poles. In terms of meeting bending strength and safety criteria, SLIPBASE is presently implementable. RETROFIX did not meet safety criteria and is not ready for implementation, but it has potential for further development

According to Mak and Mason (71) rigidity of timber utility poles is determined by the pole size in terms of diameter and height. For metal poles, the rigidity is a function of the base design and anchoring mechanism. The majority of the nonbreakaway luminaires and large signs remained rigid after impact while most breakaway poles and small signs were knocked down. Accidents in which the struck pole is knocked down generally result in a lower injury severity than those in which the pole remains rigid after the impact given similar impact conditions.

Mak and Mason (71) stated the effectiveness of breakaway modification lies in its ability to limit and reduce the extent of velocity and momentum change regardless of impact speed. In contrast, velocity and momentum change is proportional to impact speed for nonbreakaway poles (with the exception of small signs). Since injury severity is closely related to the accident severity measures of velocity and momentum change, breakaway poles are effective in reducing the resultant injury frequency and severity.

Mak and Mason (71) also found incorporation of a breakaway design into luminaires and large sign supports is effective in reducing the accident severity and the resultant injury severity. However, for small signs, the breakaway modification is not effective in further reducing the accident or injury severity since the severity is already extremely minor, even for collisions with

However, for small signs, the breakaway modification is not effective in further reducing the accident or injury severity since the severity is already extremely minor, even for collisions with nonbreakaway small signs.

Luminaire Supports

The provision of illumination on high-speed, high-volume roadways presents somewhat of a paradox for the highway designer. It has been shown to improve traffic safety but the use of luminaire supports adjacent to the roadway introduces a considerable fixed-object hazard.

The problem of the fixed-object hazard can be reduced or eliminated by positioning the luminaire supports a safe distance from the travel way or by employing a breakaway design concept for the luminaire supports. Location of luminaire supports has been studied by Walton et al. (79). They compared "median-mounted" and "house-side" (right-hand side) lighting systems and the relative hazard created by the proximity and frequency of luminaire supports.

It was concluded that a 20° impact by a 2,900-lb. vehicle at 40 mph would not cause a pole to encroach on the opposing traffic lane if the median is 40 feet wide. A 4,000-lb. vehicle impacting at 25° and 60 mph would cause a pole to encroach approximately 11 feet into opposing lanes and may be more of a hazard than the upright poles themselves. A medium size vehicle impacting a downed pole within the traffic lane presents no more hazard than the original impact. From a relative hazard standpoint, median-mounted luminaire systems produce less hazard than house-sided systems for median widths of 30 feet or greater.

According to Mak and Mason (71) there is a 50-50 chance that an occupant in an accident involving a nonbreakaway luminaire will sustain some injury even at impact speeds as low as 9 mph (8 mph velocity change or 470 lb-sec momentum change). There are no severe injuries recorded below 10 mph impact speed (10 mph velocity change or 1,000 lb-sec momentum change). The severe injury rate is less than 10 percent up to a velocity change of 31 mph, increasing to 50 percent at 47 mph of velocity change.

BREAKAWAY DESIGNS

According to Mak and Mason (71) for breakaway luminaire supports, the probability of any injury is less than 50 percent up to impact speeds of 50 mph (25 mph velocity change or 4,000 lb-sec momentum change). No severe injuries are recorded below impact speeds of 25 mph (10 mph velocity change or 2,000 lb-sec momentum change). The severe injury rate is less than 10 percent up to a velocity change of 28 mph, increasing to 19 percent at 35 mph, which is the limit of the maximum velocity change.

There are four types of luminaire base breakaway designs: frangible, progressive shear, slip, and others. An analysis of breakaway light standards (71) has been performed by the Southwest Research Institute. Their summary of pertinent design concepts follows: The frangible cast aluminum transformer base is the oldest breakaway design for luminaires in use today. The frangible base is designed to fracture at or below certain recommended base fracture energy so that the momentum change criteria will be met. This design generally fails to meet the AASHTO breakaway requirements in that the resultant momentum change is higher than the recommended maximum in both laboratory pendulum and full-scale crash tests (53). The frangible cast aluminum shoe base, cast aluminum insert, and the frangible aluminum shoe base with an integral riser are all breakaway designs based on transformer base concept. These bases performed well in collisions above 25 mph; shoe bases and inserts have mainly been used for upgrading existing nonbreakaway bases.

As with the frangible bases, the progressive shear base utilizes some type of fracture of the metal to attain a breakaway characteristic. This design uses the shear strength of spot welds or steel rivets around the skirt to provide for the "progressive" failure of the base on impact. Testing has shown that it will perform at acceptable levels only when struck at speeds of greater than 30 mph by a large vehicle.

The triangular slip base consists of two plates with cut slots. One of the plates is welded to the base and the other to the luminaire support and the two plates are bolted together at the slots using a predetermined torque. Upon impact, the top plate, with the luminaire support attached, slides in the direction of vehicle travel. Thus the bolts are forced to slide out of the slots, freeing the luminaire support to move with the vehicle. Based on the results of several studies (80, 81, 82), the slip base appears to perform the best.

There are other breakaway concepts for luminaire supports that are less common in use, such as the notched-bolt insert, the notched aluminum coupler, and the fluted aluminum breakaway coupling. These designs use necked-down or fluted sections in the bolts or couplings to decrease the energy needed for fracture upon impact. The fluted aluminum couplings have shown acceptable performance according to the specified AASHTO requirements (53). However, the notched designs have a tendency of failing in an area other than the necked-down sections, thus failing to meet the specified safety performance criteria (83).

An alternate design was recently developed in Sweden, called the ESV lighting column, which reduces the probability of any subsequent collisions. The column consists of steel rods spot-welded to a thin sheet steel skin. Upon impact, the spot welds fail and the rods and skin act as independent weaker structures which deform as the vehicle is brought gradually to rest. Since the column is not broken, it has the advantage of trapping the errant vehicle and reducing the probability of any secondary collisions. The disadvantage is that the column is destroyed after each collision resulting in high repair/replacement costs. Test results have shown that this design significantly reduces the average and peak vehicle deceleration though it is not comparable to the AASHTO specifications because of the different failure mechanism (84, 85).

Another breakaway concept is the use of fiberglass instead of metal for the pole material. Structurally, fiberglass has a high strength-to-weight ratio so that fiberglass poles can be much lighter in weight and, consequently, lower in breakaway energies and resultant accident damage and injury severity. Fiberglass luminaire poles were developed in Italy and are currently in service in several countries and appear to be performing satisfactorily. In addition to safety, structural, and aesthetic advantages, fiberglass lighting poles have a lower initial cost than concrete and metal poles. The disadvantage is that the pole is usually destroyed after each collision, requiring replacement by a new pole. The Texas Transportation Institute performed both static and crash tests on a 30-foot, 85-pound fiberglass pole (86). While the crash test was successful, the static test showed less bending strength than expected. Research and development on fiberglass poles is expected to continue.

The use of breakaway street lighting columns fitted with a suspension cable was reported by Hignett and Walker (87). Low speed and high speed collision tests were carried out to investigate the feasibility and effectiveness of connecting the tops of breakaway columns by a steel suspension cable so that after a collision the shaft of the column involved is left suspended between the two adjacent columns, and does not fall onto the carriageway or footpath. The tests showed that a suspension cable can be easily attached to the tops of breakaway columns, and the restraint put on the columns does not significantly increase the damage to the car or the risk of serious injury to occupants. Due to the large final deflection of the suspension cable there is a risk of the lower end of the column obstructing the opposite carriageway after a collision if this type of installation without barriers is used on a 13.11-foot central reserve of a dual carriageway road. It also apears that after an impact adjacent columns would need to be examined and the flange bolts changed.

Drainage Facilities

Highway drainage facilities, if improperly designed or located, can be serious roadside hazards. Drainage structures including dikes, headways, ditches, channels, and culvert ends are examples of fixed roadside hazards. As shown in Table 1 about 3 to 5 percent of the fatalities result from culvert and ditch type collisions. Hosea (88) also reported a similar percentage of fatalities for ditches and culverts. According to an NCHRP study (89), a drainage structure or device should not have vertical faces projecting above the ground or steep-sided depressions and vertical drops below the surface. Such configurations may cause vehicles leaving the roadway to come to an abrupt stop or to veer out of control causing death or injury to the occupants and extensive damage to the vehicle.

Perchonok et al. (29) also investigated the extent of roadside accidents with culverts. For 39 percent of the 444 culvert impacts, the culvert ran under the road from which the vehicle had departed. For 57 percent of the impacts, the culvert ran under another road and for 5 percent the culvert either ran under something other than a road or it was unknown which road it ran under. They questioned if there was evidence that vehicles became "trapped" by ditches and were thereby directed toward the culverts, and found less than 20 percent of culvert impacts could be identified as involving vehicles which could have been traveling in ditches leading to culverts. Further analysis showed the likelihood of traveling parallel to the road, and presumably in the ditch, was significantly higher for vehicles striking culverts than for vehicles experiencing other events. Safe end treatments are currently being tested (90) to determine critical speeds in terms of various side slopes.

OBJECTS ON ROADWAY

The previous two sections synthesized safety research relating to the design of geometric and cross section features and to the design of needed off roadway devices. At times some critical design components such as bridges and their supporting abutments and columns, retaining walls, and guardrails are closer to the roadway and can be themselves potential hazards to highway users. In order to help keep vehicles on the roads and to improve safety, highway engineers have developed systems which will aid drivers of errant vehicles to regain vehicle control plus minimize the hazard to vehicle occupants. Typically, such systems include improved guardrail concepts; rigid barriers; bridge component designs such as railings, abutments, and columns; and crash cushions. Table 9 shows the severity index for objects that are typically found on or near the roadway (51). These severity indexes were not developed from a detailed accident analysis but are relative subjective measures of an obstacle's potential to produce a given outcome on the vehicle and/or occupant when a collision occurs.

TABLE 9 - Severity Index for On or Near Roadway Objects

Objects	Approximate Severity Index
1. W-Section Guardrail:	
with standard post spacing	3.6 to 5.7
with non-standard post spacing	3.9 to 5.9
2. Post and Cable Guardrail	3.9
3. Bridge Abutments:	
vertical face	9.3
sloped face	2.5
4. Bridge Columns	9.3
5. Bridge Rail:	
rigid but smooth	3.3
other with probable penetration snagging, or vaulting	9.3
6. Concrete Median Barrier	4.2
7. Retaining Walls:	
face	3.3
exposed end	9.3
8. Crash Cushions	1.0

SOURCE: Reference 51

Traffic Barrier

Traffic barrier systems are used to redirect and attentuate the impact of vehicles. Traffic barriers are typically located longitudinally along the roadside or in a median. They are often used on highways that are designed for vehicle speeds of 50 mph or greater. They protect against embankments and roadside obstacles and, in some cases, provide protection for pedestrians. Traffic barriers should only be installed when it is not feasible to remove hazardous conditions (91). Michie et al. (92) states barrier systems should not be overused since they can also constitute a major roadside hazard. This is because they may constitute larger targets and are located close to traffic.

Longitudinal roadside barriers have three sections: the main or standard section, the transition section, and the end section. The main section is designed to redirect and/or contain the vehicle. The transition section provides continuity of protection when two different longitudinal barriers join (such as roadside barrier to bridge rail) or when a roadside barrier is attached to a rigid object (such as a bridge pier). End sections must be provided for both the upstream and downstream terminals of roadside barriers, if the barrier terminates within the "clear zone." To be crashworthy for head-on impacts, the end treatment should not spear, vault, or roll the vehicle. Vehicle accelerations should not exceed the recommended limits because injuries and fatalities also increase as acceleration increases. Table 10 shows maximum vehicle accelerations for human tolerance. For impacts between the end and the standard section, the end treatment should have the same redirectional characteristics as the standard roadside barrier.

TABLE 10 - Maximum Vehicle Accelerations for Human Tolerance

Restraint	Maximum Acceleration (g's)*		
	Lateral	Longitudinal	Total
Unrestrained Occupant	3	5	6
Occupant restrained by lap belt	5	10	12
Occupant restrained by lap belt and shoulder harness	15	25	25

*Maximum onset rate of 500 g's per sec; acceleration duration not to exceed 200 msec.

SOURCE: References 92 and 93

Barrier Types

Over the years, numerous types of longitudinal roadside barriers have been designed, tested, evaluated, and implemented. Roadside barriers, Figure 7, are usually recognized as flexible, semi-rigid, or rigid systems. Flexible systems permit considerable dynamic deflection upon impact and dissipate more energy than semi-rigid systems since they impose lower impact forces on the vehicle (91). For flexible systems, the support posts break away from the barrier and thus offer little resistance. The support posts control lateral movement of the hitting vehicles. Semi-rigid barriers rely greatly on the combined flexure and tensile stiffness of the barrier, but the support posts in this area of the impact are designed to break or tear away to aid in dissipating the impact force. Rigid barriers are unyielding and are usually constructed of reinforced concrete. They are used where space for lateral deflection of the barrier is not available. Blocked-out systems have the barrier rails offset from the posts with blocks to minimize vehicle snagging and vehicle vaulting over the barriers. Table 11 gives an indication of the relative safety performance of various kinds of roadside barriers that are found on highways. Reference 91 describes the currently recommended barrier systems and contains an excellent bibliography on barrier research. Other barrier research has been undertaken (94, 95, 96) or is continuing.

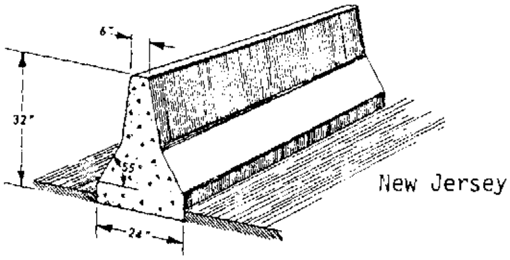
The most common rigid barrier system is constructed in a concrete shape, and these barriers are used for both medians and bridge parapets. Concrete barriers (as well as some other type barriers) while similar in appearance often perform quite differently because of many factors, e.g., vehicle weight, approach speed, impact angle, presence of superelevation, physical barrier shape, etc. Both simulation (9) and full-scale crash studies have been performed to evaluate and assess the safety performance of various concrete barrier shapes. For example, in 1977 Bronstad, et al. (97) reported the results of simulation tests on eight concrete barrier shapes including the two most commonly used designs, those developed by New Jersey and General Motors (98). According to a 36 agency survey, the Federal Highway Administration found 19 agencies used the New Jersey type and 8 agencies used the General Motors design (98). Both designs use an overall height of 32 inches and a lower impact slope of 55°. The New Jersey design has a somewhat longer and steeper wall which deters mounting, vaulting, and rolling. Crash tests by Bronstad et al. (97) have shown that the General Motors (GM) shape is more likely to cause small cars to roll over. Therefore, installation of the GM shape is no longer recommended. According to Table 12, developed by Tye (99), the repair cost for concrete barriers is considerably less than for other barrier systems.

TABLE 11 - Driver Injury by Type of Guardrail in Primary Impacts

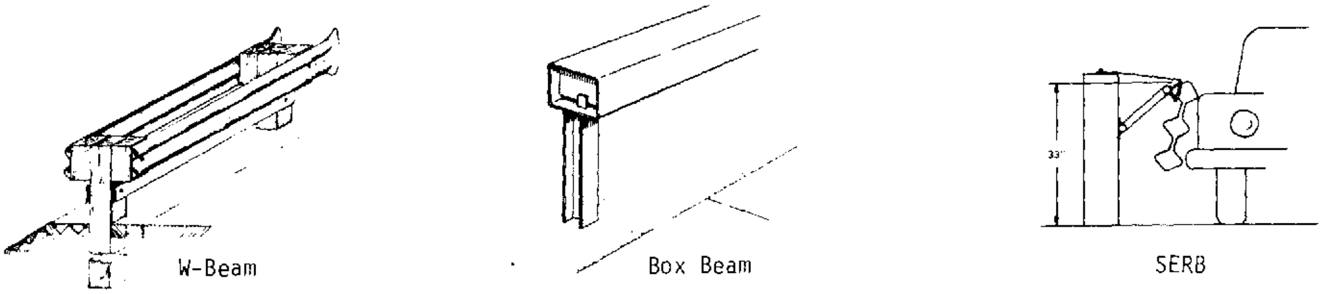
Guardrail Type	Type and Percent of Driver Injury			
	Number Observed	Percent Injury	Percent Killed	Percent Not Injured
Blocked W-Beam (Steel Post)	64	47	3	50
Blocked W-Beam (Light Steel Post)	7	29	0	71
Blocked W-Beam (Wood Post)	71	28	1	71
Parapet	11	27	0	73
Nonblocked W-Beam	30	27	7	66
Wood Post	4	25	0	75
Box Beam	14	21	0	79
Three-Strand Cable	17	18	0	82
Two-Strand Cable	13	8	0	92
TOTAL	231	31	2	67

SOURCE: Reference 29

a) Rigid



b) Semi-Rigid



c) Flexible

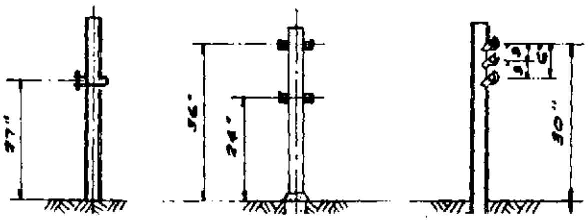


Figure 7. Examples of Roadside Barriers

TABLE 12 - Barrier Repair Costs

Barrier Type	Inventory Miles	Inventory %	Repair Cost Dollars	Repair Cost %	Percent Inventory Repaired	Repair Cost Per Inventory Mile
Cable	426	47	$719,950	73	14.8	$1,690
Beam	344	38	$258,903	26	3.8	753
Concrete	139	15	$ 8,255	1	0.03	59
Total	909	100	$987,108	100		

SOURCE: Reference 99

ACCIDENT FINDINGS

Over the years accident studies have been conducted to evaluate the performance of barrier systems. For example, Calspan reported the seriousness of barrier performance in a 6-year study (1953-58) of 935 accidents from 28 States (100). They reported in a subgroup of 595 accidents, 41 percent of the cars went over or through the barrier.

The State of California reported on operational experience of both cable-chain link fence and double blocked out median barriers (101, 102). California showed that both types of barriers were generally performing effectively, but that the cable-chain link fence median barrier was sometimes penetrated or vaulted in areas where it was installed on sawtooth-type medians. Another observed undesirable characteristic of the cable-chain link median barrier was that impacting vehicles frequently underwent rather violent spinouts that could cause the occupants to be ejected and thereby exposed to greater danger.

A study of 126 accidents was made by The Cornell Aeronautical Laboratory (103) on two-lane roads and four-lane divided highways. They found the barrier performed successfully for only 30 of the 126 collisions and 50 of the remaining 96 collisions involved end impact barrier failures. Table 13 summarizes their overall findings.

Balz (104) reported on 70 collisions with metal guardrails in Switzerland. Of the 70 accidents, 15 were end impacts, 52 were lateral collisions with the rail, and 3 were impacts in which the vehicles got behind and struck the rear side of the rail. Of the 52 lateral collisions, 33 vehicles were deflected normally and the others either spun out, rolled over, or stopped astride the rail.

Hutchinson and Kennedy (33) found that vehicles left the roadway at angles greater than 20 degrees about 15 to 20 percent of the time. Bitzl (105) found somewhat similar results when he reported about 28 percent of the guardrail accidents on the Autobahn occurred with impact angles greater than 20 degrees.

VanZweden and Bryden (106) evaluated the performance of both light-post and heavy-post barriers in a 2-year study of 4,213 accidents. They found the light-post designs resulted in less severe injuries than the heavy-post designs erected through 1965. They also reported on the effectiveness of box-beam barriers used on the Taconic State Parkway in New York. During the 29-month study, 286 median barrier accidents were recorded. Of 234 midsection accidents, 228 vehicles were contained by the box-beam barrier, while 1 vehicle penetrated the barrier and 5 overturned. They reported 22 of 31 end section accidents were also contained. They concluded, box-beam median barriers on light posts provided excellent performance even for the very narrow Parkway median.

As shown in Table 11 by Perchonok et al. (29), the steel post W-beam guardrails were the least effective in terms of mitigating injury. The nonblocked W-beam guardrail system, had the highest percent fatalities. This data showed the best performance for two-strand cable guardrail systems.

TABLE 13 - Results of Roadside Barrier Study

Barrier Performance	Number of Collisions With Type System			
	Cable	W-Section	Other	All
Successful	8	11	11	30
Principal Mode of Failure:				
End impact	4	33	13	50
Penetrated	2	2	11	15
Pocketed vehicle	2	5	0	7
Snagged vehicle	4	2	0	6
Vehicle rollover	1	2	1	4
High reflection	3	6	5	14

SOURCE: Reference 103

Single vehicle collisions with median barriers were investigated by the California Department of Transportation (99). Accident data was available for meaningful comparison of barrier experience. Barrier type and associated single vehicle collisions are shown in Table 14. A general downward trend in the accident rate is indicated for each barrier type. The 1973 total accident rate for metal beam barrier and concrete barrier was found to be significantly lower (0.10 level, chi-square test) than the similar rate for cable barriers. Although there was no difference in the fatal-plus-injury accident rates for the three barrier types, the fatal accident rate on concrete is significantly lower than on cable at the 10 percent level.

TABLE 14 - Single Vehicle Collisions With Barriers

Year/ Barrier Type	Barrier Miles	Travel (MVM)	Accident Rates Total Acc/MVM	F+I/ MVM	Fatal/ 100 MVM
1970					
Cable	379	12,956	0.38	0.14	0.43
Beam	245	8,217	0.24	0.11	0.24
Concrete	6	225	0.22	0.12	0.44
1971					
Cable	403	13,698	0.30	0.09	0.23
Beam	271	8,859	0.18	0.08	0.21
Concrete	7	249	0.20	0.10	0.00
1973					
Cable	426	14,773	0.28	0.07	0.24
Beam	344	10,554	0.18	0.07	0.17
Concrete	139	3,560	0.18	0.06	0.08

PDO = Property Damage Only Accidents
F+I = Fatal Plus Injury Accidents
F = Fatal Accidents
MVM = Million Vehicle Miles

SOURCE: Reference 99

In March 1971, the Federal Highway Administration issued a FHWA Notice (98) compiling the States' practices and experiences with concrete median barriers and parapets. The safety results were reported as having dramatically reduced head-on-cross-median accidents in New Jersey. The Transportation Department of New Jersey reported the following:

- "In Hillside, where over 70,000 cars a day traverse Route U.S. 22, 11 persons had died in the three-year period before the erection of center barrier in 1954. There have been no deaths from head-on collisions since January 1965.

- "More than 13 years have passed since the erection of center barrier on Route 4 in Teaneck. In this area, where about 70,000 cars a day now pass, there have been no deaths from head-on collisions reported during that period.

- "The Pulaski Skyway (Routes U.S. 1 and 9) had 367 accidents of all kinds resulting in 271 injuries and 8 fatalities during 1955 and 1956. There were only 172 accidents involving 106 injuries and no deaths in 1957 and 1958, after a center barrier was installed. This is less than half as many accidents with all deaths eliminated."

Bronstad et al. (97) described the accident experience of concrete barrier shapes used by 15 agencies. The data shown in Table 15 only reflects reported accidents and not "brush" impacts that also occur. No fatalities were reported for either type of barrier.

TABLE 15 - Concrete Barrier Accident Data

Barrier Type	Total Accidents	Barrier Performance(a) Vehicle Rollovers	Vehicle Mountings	Accident Severity(b) PDO	Hospital	Total Injury
New Jersey	180	6 (3)	1 (1)	133 (79)	35 (21)	168 (100)
General Motors	299	19 (6)	4 (1)	255 (79)	74 (25)	299 (100)

(a) Numbers in ( ) are percentage of total accidents
(b) Numbers in ( ) are percentage of total property damage only (PDO) and injury accidents

SOURCE: Reference 97

The State of Arizona reported that 25 cross-median accidents had occurred over a 5-year period prior to barrier installation and no cross-median incidents after construction of approximately 2 years. The District of Columbia reported the following annual accident experience on a particular project:

	1967 (Before)	1969 (After)
Reported Accidents	174	145
Injuries	95	74
Deaths	8	3

These reports demonstrate that concrete median barriers have kept vehicles from passing into the opposing lanes.

Ideally, guardrails should safely redirect vehicles along their intended path as opposed to creating sudden stops, vaulting, rollovers, or penetration. Sudden stops are undesirable because of the potential of creating high "g" forces. Perchonok et al. (29) examined 515 guardrail impacts for ensuing vehicle behaviors. They found 50 percent of the impacting vehicles were redirected or continued along the road. Almost one-third of the vehicles went through or over the guardrails they struck, while 3 percent vaulted as a result of hitting guardrails. Twelve percent of the vehicles came to a sudden stop after hitting guardrails, however, their average impact speed was only 13 mph with a standard deviation of 7 mph.

For selecting appropriate guardrail configurations, Calcote (107) developed a cost-effective model in which a user can consider 11 different configurations, specific criteria, and site impact conditions.

Crash Cushions and Impact Attenuators

Rigid objects or hazardous conditions that cannot be eliminated, relocated, or made breakaway should be shielded from errant vehicles. Crash cushions are defined as protective systems that prevent errant vehicles from impacting roadside hazards by either decelerating the vehicle to a stop when hit head-on or potentially redirecting the vehicle away from the hazard in the case of glancing impacts. Figure 8 shows examples of crash cushions.

Exit gores have typically experienced operational and safety problems (108). Taylor and McGee (108) provide a good summary of the problem of erratic driving maneuvers at gore areas, analyze causal factors, and recommend remedial devices. Bruner and Juba (109) evaluated the effects of improving gore area delineation through the installation of post-mounted reflectors and raised pavement markers. The average erratic maneuver rate was 43 percent of the observed rate before installation. The study found that 60 percent of the ramp vehicles sampled showed a significant decrease in speed without affecting the mainline traffic speed. When gore delineation or design cannot be improved, impact attenuators are candidate improvements, and various kinds of impact attenuators have been implemented.

o Steel Drums

This system ("Texas Barrel") dissipates the kinetic energy of the impacting vehicle primarily through the plastic deformation or crushing of the steel drums. The drums are restrained vertically and laterally by steel cables but are free to move to the rear during impact. A rigid backup structure is required. The steel drum system is designed to redirect a vehicle, if hit from the side. The results of a study (110) on the crash experience of the steel drum found the elimination of the redirection panels on crash cushions at sites with low probability of angular impacts would improve the safety and reduce the construction and maintenance costs of these devices by one-half or more. Problems with fatigue failure and system maneuverability were also found.

o Hi-Dro Cell Cushion

This system dissipates the kinetic energy of the impacting vehicle by discharging water through small openings in the plastic tubes and by transfer of momentum (movement of the water mass). Viner and Boyer (111) analyzed 188 impact attenuator sites from 48 installations in 17 States which had experienced a total of 593 accidents. They found that only one fatality occurred in 106 impacts. Further analysis estimated that 13 of the impacts would have resulted in death or serious injury if the system was not in place. They also found that the total accident experience increased due to a reduction of clear area in the gores and a higher accident reporting level in the "after" period. Kruger (112) also found a high reduction in fatalities and injuries when fixed objects were protected by Hi-Dro Cells. Kruger also reported the system to be highly cost effective.

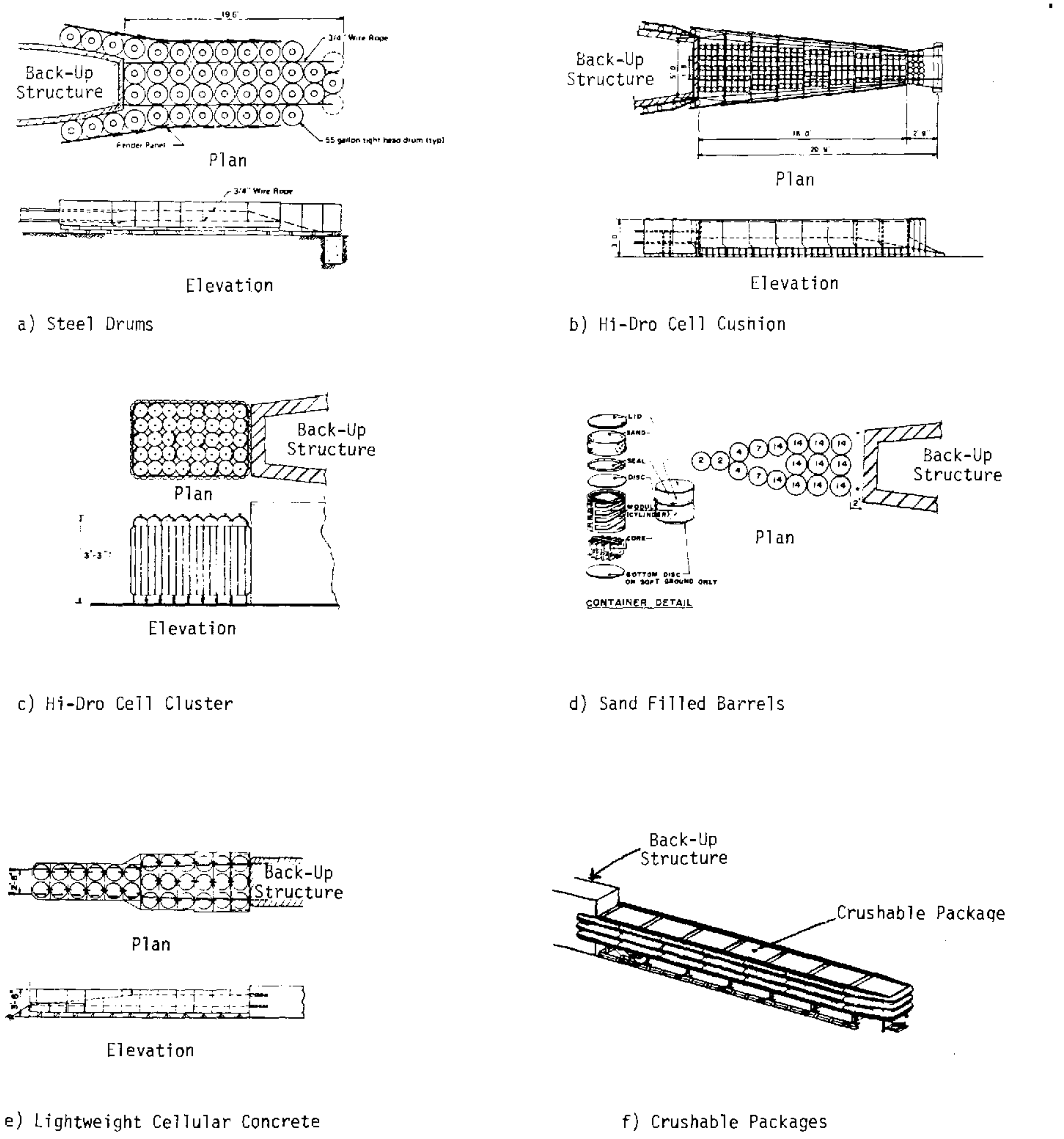


Figure 8. Examples of Crash Cushions

o Hi-Dri Cell Cushion

This system dissipates the kinetic energy of the impacting vehicle through the crush of the lightweight concrete components and through the transfer of momentum (movement of the cushion mass). The system is designed to redirect a vehicle, if hit from the side. Tests on the Hi-Dri Cell (113) were performed using an 1,800-lb. lightweight car, 3,700-lb. standard-size car, and a 3,700-lb. truck. Impacts were at both low and high speeds, head-on and at angles. The researchers found that Hi-Dri attenuators are significantly better than the Hi-Dro Cell cushion attenuator, that it has multiple-hit capacity, that it can be tailored for the traffic characteristics of a location, and that maintenance is fairly simple.

o Hi-Dro Cell Cluster

This system functions similarly to the Hi-Dro Cell Cushion. Its application is limited to roadways with design speeds of 45 mph or less. The system has no redirection capabilities. Accident information (111) on the Hi-Dro Cell Cluster, up to October 1972, indicated that there were 61 accidents resulting in 2 fatalities and 8 injuries. Of the fatalities, one involved a motorcycle and the other apparently impacted in excess of the design speed.

o Sand/Filled Plastic Barrels

This system dissipates the energy of the impacting vehicle by a transfer of the vehicle's momentum to the mass of the cushion. The system is not designed to redirect vehicles that impact it from the side. Driver crash tests (114) at 65 mph demonstrated satisfactory vehicle deceleration. Depending on vehicle speed, weight, angle of impact, and barrier configuration, the results ranged between 2 and 6 g's. While vandalism and scattered debris after impact are troublesome, the versatility as an effective protection device is considered a primary advantage. Accident reports (111) indicate that in many cases, serious injury or fatality would result if the barrels were not present. Two commercially available sand/filled plastic barrel systems are described in Reference 115.

o Lightweight Cellular Concrete

Ivey et al. (116) and White and Hirsch (117) have also reported the results of crash cushions made out of other materials. For example, lightweight low strength concrete has been constructed with vermiculite aggregate (118). While acceptable deceleration levels were obtained with 2,000-and 4,000-lb. vehicles in full-scale tests, implementation of lightweight concrete cushions has been a problem. States have had problems with construction especially in batching and forming the material. Capillarity and poor freeze-thaw properties have discouraged acceptance and implementation of this system. Cluster systems of lightweight concrete cylinders have also been developed for use with narrow objects such as piers or the end of concrete median barriers (115).

o Corrugated Steel Pipe

Full-scale vehicle crash tests on corrugated steel pipe crash cushions have also been performed. Initially, vehicle ramping of cushions was found but after additional hardware modifications and the use of a pipe-arch nosepiece, head-on deceleration levels were well below 12 g's (117).

o Crushable Packages

Crushable packages of energy absorbing materials have also been developed (115). The system consists of paper honeycomb cubes impregnated with rigid foam. The length of the cushion can be varied to accommodate the required design. The crushable packages are surrounded by a framework of overlapping sections of triple corrugated steel guardrail and restrained laterally at the bottom by a chainrail and longitudinally by a cable at the top. Successful full-scale vehicle crash tests for both head-on and small angle impacts have been performed using 1,900-to 4,300-lb. cars at about 60 mph (118).

Overall, the safety performance of impact attenuators has been good. For example, Kruger (112), reporting impact attenuator experience in Seattle, observed a 157-percent increase in property damage accidents at six locations. At the same time, injury accidents decreased 72 percent and fatal accidents were eliminated. Data on 129 attenuator accidents were studied by Viner (119). According to Viner, had the attenuators not been present, hospitalizing injuries would have increased from the 23 observed to an expected 30.

BRIDGE RAILING AND SYSTEMS

Bridge-related accidents typically involve vehicles hitting (1) bridge ends or approach guardrail systems and (2) bridge railings. Typical systems selected include flexible beam/posts, rigid beam/posts, or rigid concrete beam/posts. According to Bronstad and Michie (120), adverse accident experience of bridge railing systems results with poor treatment of transitioning from either no approach guardrail or a flexible approach guardrail to a rigid bridge rail or an abutment. In analyzing 1,195 Texas bridge end accidents, they found fatal accidents, as shown in Table 16, about seven times more likely when the vehicle penetrates through, under, or over the barrier as compared to being retained by it.

Bronstad and Michie (120) also described the findings from 8,562 single vehicle accidents involving bridge railings in Texas and Washington. They report about 90 percent of all bridge related accidents are of the single vehicle type. They also reported, as shown in Table 17, that about 1-percent of the accidents were fatalities when the bridge railings contained/redirected the vehicles on the bridge. The percent fatalities were 7 to 14 times greater when the vehicles went through, under, or over the bridge rail.

Bronstad and Michie (120) reported the concrete safety shape bridge parapet is currently the most commonly specified bridge railing. Kimball et al. (121) described other bridge railing research based on the use of collapsing rings.

TABLE 16 - Bridge Railing Accident Data

Performance	Injury Severity			Total
	None	Some	Fatal	
Vehicle Retained	711 (68)	290 (28)	38 (4)	1039
Through, Under, or Over	34 (22)	75 (48)	47 (30)	156
TOTAL				1195

Percent ( )

SOURCE: Reference 120

TABLE 17 - Bridge Railing Accident Data

Performance	Injury Severity			Total
	None	Some	Fatal	
Texas:				
Vehicle Retained	3607 (63)	2054 (36)	70 (1)	5731
Thru, Under, or Over	138 (31)	241 (55)	61 (14)	440
TOTAL				6171
Washington:				
Vehicle Retained	1362 (60)	909 (40)	14 (1)	2285
Thru, Under, or Over	43 (41)	56 (53)	7 (7)	106
TOTAL				2391

Percent in ( )

SOURCE: Reference 120

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CHAPTER 4 - ACCESS CONTROL AND DRIVEWAYS

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INTRODUCTION

As travel demand and adjacent land use increases in the developing urban fringe, highways deteriorate in their ability to accommodate traffic safely and efficiently. The roadways are serving the dual functions of providing land access and vehicular movement.

Freeways provide a high degree of safety primarily because of controlled access. Urban arterials and other non-freeway facilities operate in a developing roadside environment with a lower degree of safety. Solomon (1) advises that arterial highways constructed on new rights-of-way initially involve few commercial driveways. As traffic volume and roadside development increase, increasing numbers of driveways cause accident rates to gradually increase. Cirillo et al. (2) show that an increase in the frequency of intersections and business on two-lane rural highways results in increased accident rates as follows:

Intersections per Mile	Businesses per Mile	Accidents per Million Vehicle Miles
0.2	1	1.26
2	10	2.70
20	100	17.18

Access to highways from residential, commercial, and public property should be equitably managed to achieve both highway safety and reasonable access. A number of non-research references are listed that give current practice.

The two basic types of access control are <u>roadside</u> and <u>median</u>. <u>Roadside</u> control relates to the design and spacing of roadside access facilities including driveways, frontage roads, service roads, and intersections. <u>Median</u> control relates to the design and spacing of median crossovers, U-turns, and left turns for use by drivers desiring entrance to or exit from abutting property.

Access control on a given highway may range from none for a local street to full control of access for a freeway. Figure 1 illustrates the generalized relationship between the degree of access control, traffic flow, and highway functional classification.

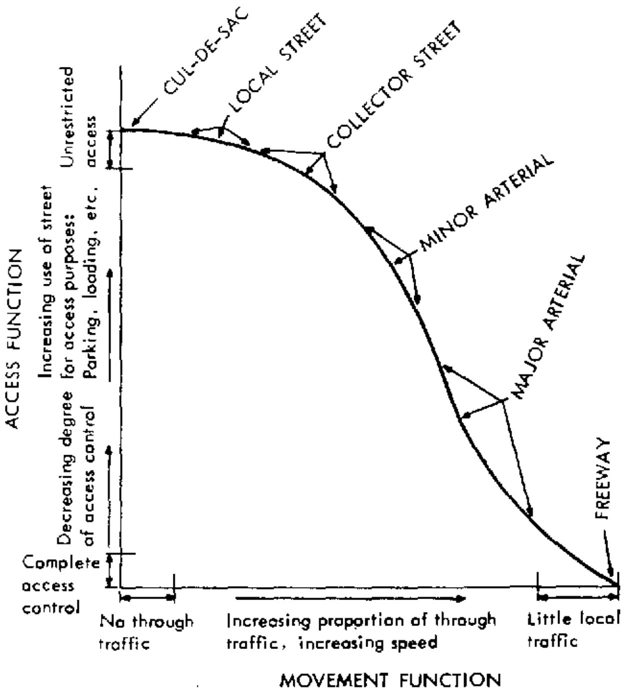


Figure 1. Relationship Between Control of Access and Traffic Movement

SOURCE: Reference 49

ACCESS CONTROL AND ACCIDENT EXPERIENCE

Traffic engineers recognize that the elimination of unexpected events and the separation of decision points simplifies the driving task. Access control reduces the variety and spacing of events to which the driver must respond. This has resulted in improved traffic operations and reduced accident experience.

Based on an analysis of data from 30 States, a report to the 86th U.S. Congress (3) concluded that full control of access has been the most important single design factor ever developed for accident reduction. Entrance and exit movements from and to the through traffic lanes are limited to designated points where these maneuvers can be performed safely. As shown in Table 1, accident and fatality rates on facilities with full control of access were about one-half on rural highways and one-third on urban locations when compared to facilities without access control. The average accident and fatality rates for rural highways were much lower for full control of access than for partial control of access. In urban areas, little difference was found between routes having partial access control and no access control.

TABLE 1 - Effect of Control of Access on Accidents and Fatalities in Urban and Rural Areas

Access Control	Urban Total	Urban Fatal	Rural Total	Rural Fatal
Full	1.86	0.02	1.51	0.03
Partial	4.96	0.05	2.11	0.06
None	5.26	0.04	3.32	0.09

Accident Rates - Accidents per Million Vehicle Miles

SOURCE: Reference 3

Gwynn (4) studied accidents on segments of two interstate highways with control of access and the parallel roadways without control of access. As illustrated in Tables 2 and 3, the accident and injury rates on the interstate highways during 1964 were less than one-fifth of those for the parallel highways for the same year. Also shown in these tables are 1958 data for the parallel routes prior to the opening of the interstate highway. It appears that the parallel routes were just as "unsafe" in 1964 as they were in 1958 prior to the opening of the interstate highway.

Solomon (5) reported data demonstrating that the chance of being involved in an accident follows a U-shaped distribution. The chances of being involved in an accident are at a minimum when the vehicle is traveling at about the average speed of traffic for both night and daytime conditions. Subsequent research by Cirillo et al. (6) produced similar results. As shown in Figure 2, the chance of being involved in an accident on conventional rural highways is minimum when a vehicle is traveling about or slightly above the average speed of traffic. The chance increases at speeds above and below the average speed. For freeways, the minimum chance of accident involvement is about 10 miles per hour above the average traffic speed. The freeway driver, being relatively free from marginal and median access events, can concentrate on the driving task involving only vehicles moving in the same direction.

TABLE 2 - Comparison Accident and Injury Rates
(Camden County, New Jersey)

	Interstate Route 295	U.S. Route 130	
	(1964)	(1964)	(1958)**
Total Mileage	10.06	10.30	10.30
Average Daily Volume	32,680	33,780	33,800
Accidents (Number)	89	539	549
Accident Rate*	0.74	4.24	4.32
Injuries (Number)	59	490	429
Injury Rate*	0.49	3.85	3.38

*Accidents per million vehicle miles
**Prior to opening Interstate

SOURCE: Reference 4

TABLE 3 - Comparison Accident and Injury Rates
(Morris County, New Jersey)

	Interstate Route 80	U S. Route 46	
	(1964)	(1964)	(1958)**
Total Mileage	13.27	14.03	14.03
Average Daily Volume	17,130	16,371	17,443
Accidents	73	489	482
Accident Rate*	0.88	5.83	5.40
Injuries	69	411	354
Injury Rate*	0.83	4.90	3.96

*Accidents per million vehicle miles
**Prior to opening Interstate

SOURCE: Reference 4

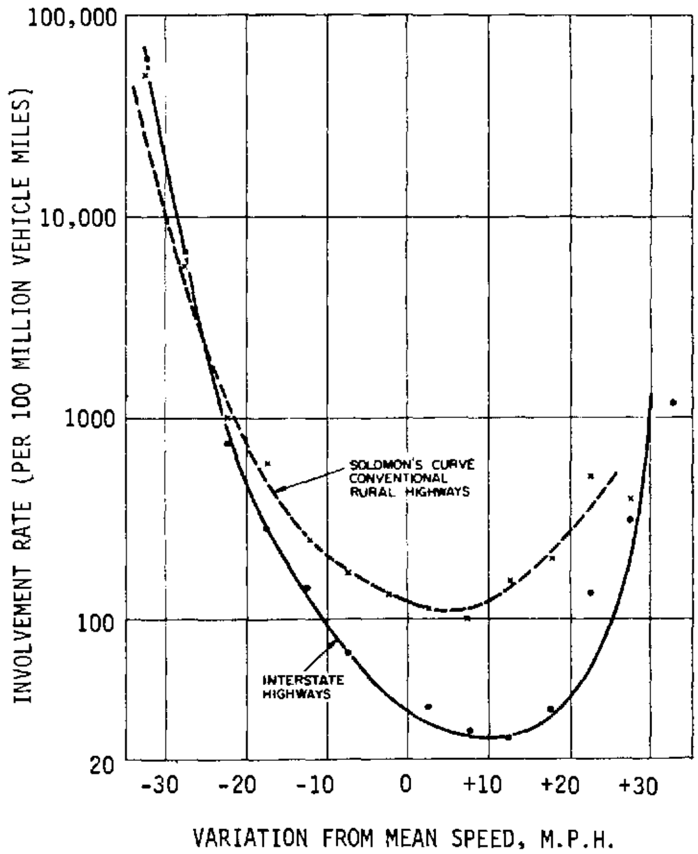


Figure 2. Accident Involvement Rate by Variation from Mean Speed on Study Units

SOURCE: Reference 6

ROADSIDE ACCESS CONTROL ACCIDENT EXPERIENCE

Urban and rural research has found that accident frequencies increase as traffic volumes increase. While partial access control can help offset this, the most effective method for reducing these accident frequencies is full control of access.

Research by Schoppert (7) reported findings in 1957 demonstrating that accident rates increase with increases in traffic volume and/or access frequency. This finding is still valid. The following conclusions resulted from Schoppert's study of 3 years of accident experience on two-lane rural highways in Oregon:

- o Accidents are directly related to vehicle volumes and highway physical features.
- o Highway access from driveways and intersections is directly related to accidents at all ADT levels. The number of access points is a reasonable predictor of the number of potential accidents within an ADT group.
- o Accidents are chance occurrences resulting from errors in judgment.
- o Accidents are chance occurrences particularly on low volume roads.
- o Accidents increase with the number of roadway and traffic changes which induce driver decisions. These changes include increases in volume and access points, inadequate sight distance, and reduced cross section.
- o The average number of access points per mile tends to increase as traffic volumes increase.

Research by Cribbins (8) in North Carolina found an inverse correlation between shoulder width and accident rates on two-lane rural highways. He found that paved shoulders effectively serve as right turn lanes and as by-pass lanes for through vehicles when left turning vehicles block the traffic lane. Fambro et al. (9), reporting a Texas study, found that 95 percent of the time, on two-lane roadways with paved shoulders, drivers use the travel lane. Five percent of the time drivers may pull onto the shoulder to let an overtaking vehicle through or to pass a left turning vehicle. The Colorado Highway Department uses pavement markings and signing to designate paved shoulders for use by "right turning vehicles only" in proximity to intersections.

McGuirk (10) found that driveway accidents increase significantly as both traffic volumes and frequency of access increase. He found a number of interacting variables affecting the number of accidents per mile. These variables include the numbers of traffic lanes, commercial driveways, intersections per mile, driveways per mile, commercial driveways per mile, and the urban area population.

Accidents increase as the number of traffic lanes and access points increase. This reflects the greater traffic stream friction that occurs as through vehicles change lanes to avoid the slower speed, turning vehicles. The interactions of traffic volume and number of driveways, and traffic volume and number of commercial driveways reflect the effect of driveway volume. Commercial driveways generally experience relatively high volumes. Access design deficiencies become more critical as the urban area becomes larger.

Glennon et al. (11) evaluated techniques and developed technical guidelines for the control of direct access to arterial highways. He made use of equations developed by Mulinazzi and Michael (12) to estimate the annual number of accidents per mile of highway. Table 4 gives the results of these calculations based on three ranges each for number of driveways per mile and average daily traffic. (Information regarding the estimated safety effectiveness of selected techniques from the Glennon study is presented later in this chapter.)

TABLE 4 - Annual Number of Driveway Accidents per Mile by Frequency of Access and Traffic Volumes

Level of Development (Driveways Per Mile)		Highway ADT (Vehicles Per Day) Low <5,000	Medium 5,000 15,000	High >15,000
Low	<30	12.6	25.1	37.9
Medium	30 - 60	20.2	39.7	59.8
High	>60	27.7	54.4	81.7

SOURCE: Reference 11

Table 4 demonstrates that accidents increase with increased frequency of access points and with arterial street volume. Sections with both high levels of driveway development and high ADT average 6.5 times the accidents per mile as those facilities with both low driveway development and low ADT. Table 4 also indicates that for high ADT volumes, the average number of accidents per mile is about three times the number for low ADT's.

During a study of traffic lane width on urban arterials, Heimbach et al. (13) found that the number of accidents increased as the number of access points increased. The number of side road intersections and the volume of trips to and from roadside commercial establishments were significant factors. Flow interruption accidents involved vehicles attempting to enter or leave driveways at an unsignalized street intersection. Lane maneuver accidents were related to driving skill. They involved lane encroachments, lane changes, centerline crossings, and leaving the traveled way.

DRIVEWAY ACCIDENT EXPERIENCE

Driveways are similar intersections. Their efficiency and safety depend on traffic volumes, geometric design, and traffic control systems. Observation indicates that more attention is given to the design, location, and control of intersections with public streets, though some driveways carry more traffic than many intersections. High volume, signalized driveways usually have the same geometric features as intersections having similar approach volumes and might therefore be similarly evaluated in terms of accidents. The relationship between safety and the location and design of lower volume driveways is a separate topic in the research literature.

Driveway accident data is helpful in diagnosing the problems of conflicting traffic maneuvers. Obtaining consistent and meaningful driveway accident data is complicated by:

- o Difficulty in identifying causal factors.
- o Difficulty in assigning collision locations.
- o Incompleteness of reporting.
- o Probable high proportion of unreported accidents.

Marks (14) reported that 6.5 percent of Los Angeles County, Calif. accidents involved uncontrolled driveway access. In a study using data for a 2-year period, Michael and Petty (15) found that 14.4 percent of the two vehicle accidents on Indiana county roads involved driveways. According to a study by Peterson and Michael (16), driveway accidents accounted for 6.8 percent of all accidents on county roads in Indiana.

The percent of rural accidents involving driveway maneuvers can be cited from three references:

Reference		%	Report Date
Cribbins et al.	(17)	13	1967
Box	(18)	11	1968
National Safety Council	(19)	9	1978

An early study, on the effect of access control on two-lane rural highways in Minnesota, by Staffeld (20) found that sections having one or more commercial driveways had an average accident rate about twice that of sections having driveways serving farms and rural residences. However, the average accident rate on sections without driveways was slightly less than that for sections having low volume driveways serving farms and residential uses. The average accident rate within 300 feet of a commercial establishment was about 30 percent higher than the average for commercial driveway sections totaling 25 miles. Table 5 shows that accident rates tend to increase with both ADT and frequency of access. Some of the accident rates were based on one or two road sections causing some irregularities, particularly with respect to the higher traffic volume groups.

Uckotter (21) studied data collected for a 3-year period from 14 road sections in five central Indiana cities. He developed a model to predict the number of driveway accidents per mile per year for roadway sections serving commercial land use. One-third of the traffic accidents studied were driveway accidents. This percentage is much higher than indicated by most recent driveway studies. When the data were broken down by movements, such as ingress, egress, and left and right turns, the findings are comparable. His other findings included:

- o 53.4 percent of the accidents involved vehicles entering driveways.
- o 43.1 percent of the accidents involved vehicles leaving driveways.
- o Left turn movements were involved in 63 percent of the total driveway accidents and 71.4 percent of the personal injury accidents.

Box (22) compared the right turn entering the driveway from the through traffic lane with the right turn from the driveway onto the street. Table 6 shows that these two maneuvers each produce 15 percent of the total driveway accidents.

TABLE 5 - Accident Rates Related to Average Daily Traffic and Access Points per Mile

Average Daily Traffic	Access Points Per Mile						
	0-3.9	4-7.9	8-11 9	12-15.9	16-19.9	20-23.9	24-27 9
	Accidents Rates (Accidents per Million Vehicle Miles)						
1000-1999	0.70	0.77	1.05	1.36	2.85	2.75	3.17
2000-1999	1.25	1.60	1.63	2.42	1.97	2.40	3.95
3000-3999	1.74	2.03	1.86	1.93	1.50	0.95	2.60
4000-4999	-	0.75	-	1.36	1.80	3.85	2.25

SOURCE: Reference 20

TABLE 6 - Driveway Accident Types Related to Turning Movements on Major Routes Without Barrier Median

	Number of Accidents	Percent of Group	Percent of TOTAL
Entering Driveway			
Left Turn			
Rear-end	148	45%	26%
Head-on angle	87	26	15
Other	11	3	2
Sub Total	246	74	43
Right Turn			
Rear-end	71	21	12
Backing	5	2	1
Other	9	3	2
Sub Total	85	26	15
Entering Total	331	100%	58%
Leaving Driveway			
Left Turn			
Rear-end	10	5%	2%
Right angle	136	58	24
Other	4	2	1
Sub Total	150	65	27
Right Turn			
Rear-end	11	5	2
Right angle	41	17	7
Backing	15	6	3
Other	17	7	3
Sub Total	84	35	15
Leaving Total	234	100%	42%

SOURCE: References 18 and 22

DRIVEWAY INFLUENCE ON THROUGH TRAFFIC

Stover (23) reported that, with commonly used driveway curb radii and throat widths, the right turning vehicle creates a high speed differential at a substantial distance upstream from the driveway. Figure 3 shows that when the speed of through traffic is about 30 mph, the potential right turning vehicle begins to have an effect on following vehicles 8 seconds prior to the turn. The average speed profile with respect to time is slightly different for various combinations of driveway curb radii and throat widths. When the turning vehicle clears the through traffic lane arterial speeds are not significantly different for the several driveway designs shown. Stover concluded that conventional designs for driveways and unsignalized intersections result in high speed differentials as well as long exposure time.

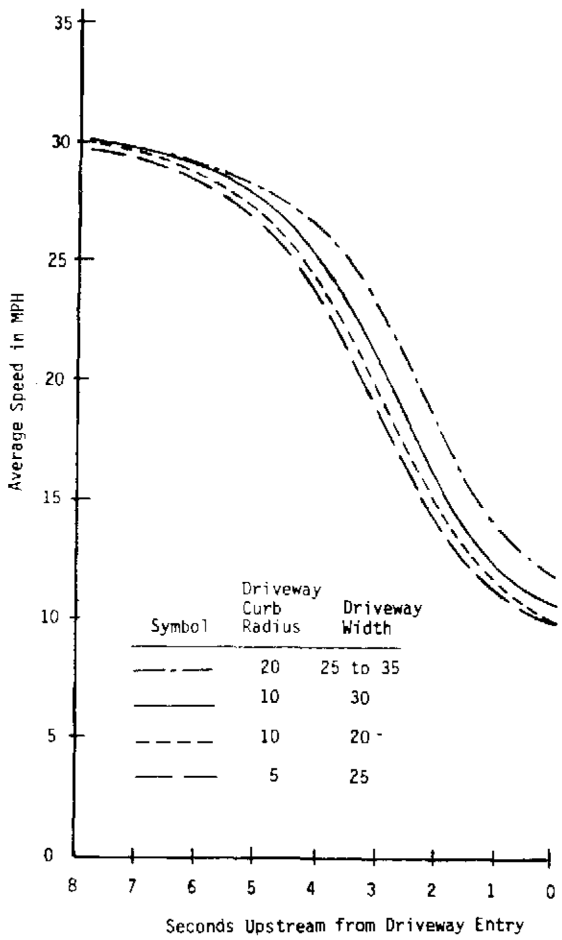


Figure 3. Potential Turning Vehicle Speeds Approaching Driveway

SOURCE: Reference 23

Notes: (1) No exiting vehicle is stopped in the driveway. The entire throat width is usable by the turning vehicle.

(2) Time 8 seconds is the point at which the turning vehicle begins to slow down for driveway.

(3) Time zero is the point at which the turning vehicle has cleared the through traffic lane.

Figure 3 also indicates the difficulty that may be involved in the identification of accidents associated with driveways. A vehicle decelerating and preparing to enter a driveway, under moderate to heavy traffic conditions, precipitates a shock wave in the traffic stream which may result in an accident a considerable distance upstream. Such an accident may be incorrectly identified as a rear end collison in the through lane rather than as a driveway related accident.

A study by Solomon (5) indicated that speed differential (or variance) on rural highways is a major factor in two-car accidents as shown by Figure 4. He found that rear end and angle collisions tend to increase as the number of intersections per mile increase. This was true for two-lane highways during both the day and night and for four-lane highways during the day. Head-on collisions tend to increase with the number of intersections per mile for two-lane highways at night and for four-lane highways during both day and night. Highways having few intersections per mile also had few driveways per mile. His analysis points up the safety benefits of controlling access to the highway.

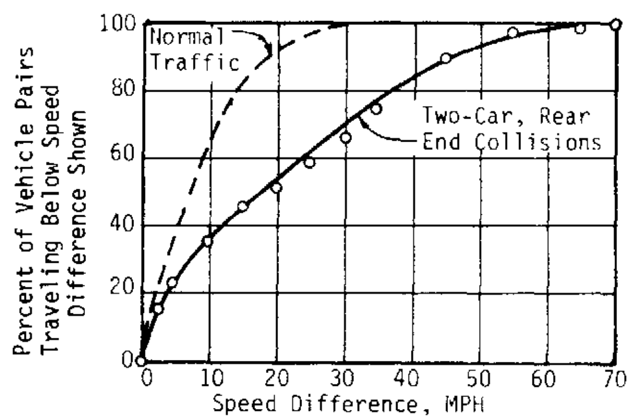
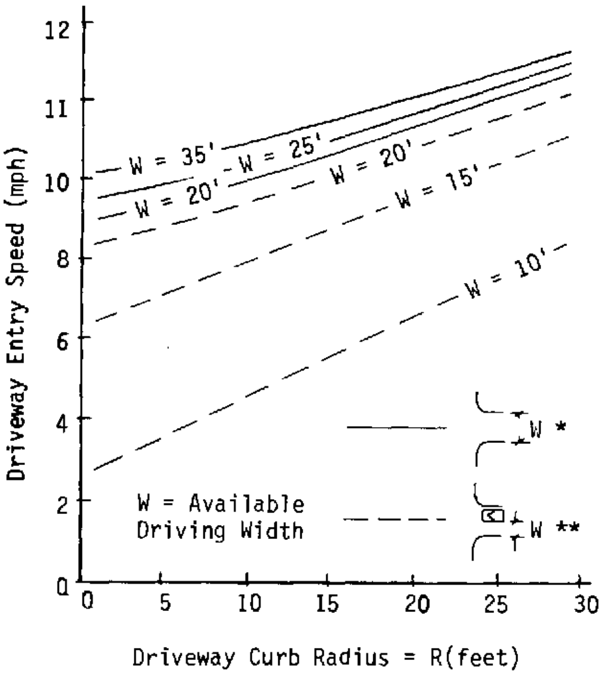


Figure 4. Speed Difference Between Passenger Cars Involved in Two-Car, Rear End Collisions Compared with Normal Traffic, Day and Night Combined

SOURCE: Reference 5

Richards (24) studied the speed of the vehicle turning into the driveway. When another vehicle is waiting to exit from the driveway, the lowest speed of the turning vehicle occurs in the through traffic lane. As shown in Figure 5 the speed of right-turning vehicles was found to be slow (less than 12 mph) even with wide available driveway widths (25 feet or more) and unusually large curb radii (20 feet or more). These conditions result in large speed differentials between the turning vehicle and through traffic. Another result is long exposure times, the time the turning vehicle occupies the through traffic lane. Speed differentials and exposure times increase as available driveway widths decrease. These data, together with accident and conflict data, indicate the importance of driveway design particularly on major arterial streets.



* No existing vehicle present; lowest speed occurs approximately midway in turn.

** Existing vehicle stopped in driveway; lowest speed occurs in through traffic lane.

Figure 5. Speed of Auto Entering Driveway as a Function of Curb Radius and Available Driveway Width

SOURCE: Reference 24

Richards (24) also found that the addition of a driveway "center stripe" caused the driver to better position the vehicle while waiting to reenter the street. In the absence of a driveway centerline marking, left turning vehicles tend to be positioned to the right of the center of the driveway when waiting to complete the left turn maneuver. Right turning vehicles tend to be positioned to the left in the driveway. This indicates that the 10-foot curb radii commonly used at driveways are inadequate.

Comparison of data regarding the path of the right front wheel (24) of vehicles entering a driveway indicates that the width of the driveway throat has little influence on the path of the vehicle for curb radii over 10 feet. For very short radii, the pattern of the paths within the driveway are widely dispersed. There is an increased tendency for drivers to use the entire available driveway width for the maneuver. Figure 6 illustrates how the path of the right front wheel of a turning vehicle is usually dispersed during a turn involving a 10-foot curb radius. The study found that curb radii and throat widths are interrelated. The desirable radii are between 10 and 20 feet. Desirable throat widths are between 25 and 30 feet. All combinations result in low turning speeds and high speed differentials with the through traffic.

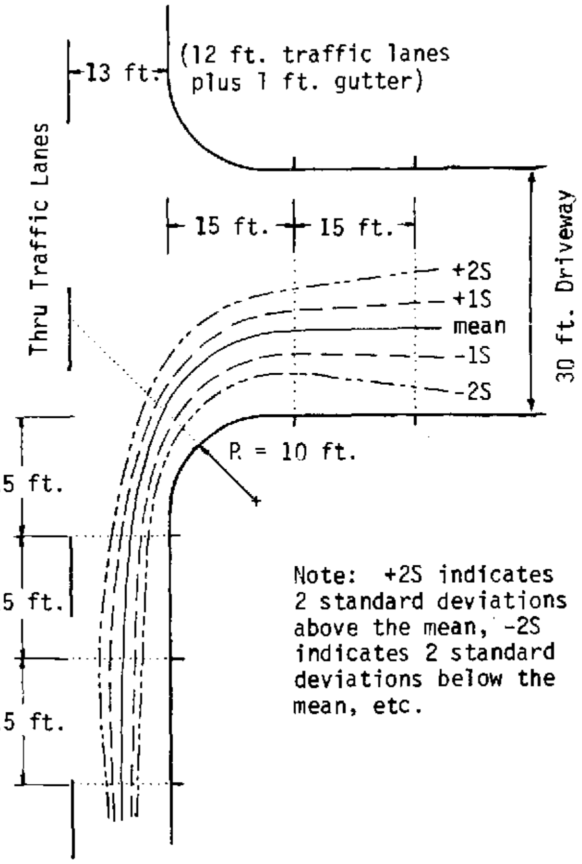


Figure 6. Path of Right Front Wheel for Selected Driveway Design

SOURCE: Reference 24

Offsetting the curb radius, with a spiral from the edge of the through traffic lane, enables right turning vehicles to partially clear the through traffic lane during the driveway entrance maneuver before a minimum speed is reached. This effect is shown in Figure 7 (24). While drivers will apparently take advantage of the sprial driveway geometry, high speed differentials still result in the through traffic lane.

DRIVEWAY SPACING EFFECTS

The relatively high percentage of rear end accidents involving turning vehicles entering driveways results, in part, from the overlapping conflict areas that exist with closely spaced driveways.

Major and Buckley (25) indicated that closely spaced driveways increased the conflict within the arterial street traffic and between driveways. The result was reduced street capacity and increased delays for traffic entering the arterial from abutting properties. These conclusions are confirmed by traffic simulation studies reported by Stover et al. (26). Access point spacing greater than 1.5 times the distance needed for entering vehicles to accelerate to the speed of the through traffic stream increases the absorption characteristics of the traffic stream and decreases delay to the entering vehicle. The resulting distances for locating access points on arterials based on speeds and average acceleration rates are:

Average Acceleration (fps^2)	Speed (mph) 30	35	40	45
2	735 ft.	990 ft.	1300 ft.	1630 ft.
3	490 ft.	660 ft.	860 ft.	1100 ft.
4	360 ft.	500 ft.	660 ft.	825 ft.

McGuirk and Satterly (27) found that driveway accident rates decrease as the number of driveways decrease. He concluded that each commercial driveway on an arterial street adds between 0.1 and 0.5 accidents per mile per year depending on the ADT and the number of traffic lanes.

Bochner (28) reported that the capacity of a four-lane arterial street is reduced 1 percent for each 2 percent of the traffic that utilizes the lane affected by the access points. For example, if a street carries 1,200 vehicles per hour in one direction and 120 vehicles turn into driveways and 120 turn out of driveways (20 percent turns), then the capacity of that direction will be reduced by 10 percent. He also indicated that as the level of design of the driveway is increased (allowing turns to be made at higher speeds), the capacity loss is reduced.

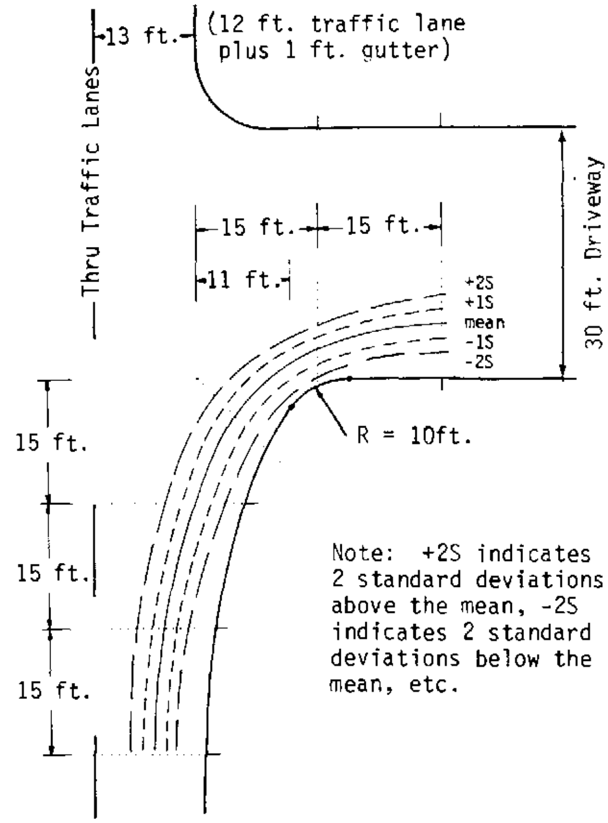


Figure 7. Path of Right Front Wheel with Spiralized Offset Design

SOURCE: Reference 24

Glennon et al. (11) and Stover (23) calculated minimum distances needed to eliminate overlapping conflict areas, as shown in Table 7. The vehicle entering the traffic stream needs time to accelerate to achieve a reasonable speed differential with through traffic. Meanwhile, a driver of a through vehicle should be presented with one conflict situation at a time. The spacing given in Table 8 provides for the acceleration of the vehicle entering the traffic stream and considers the distance traveled by the vehicle already in the through traffic during this acceleration. While these distances are less than that needed for the right turn maneuver from the street to an access point, they are considerably greater than the spacing required by most municipal ordinances and standards.

The Glennon et al. (11) distances are based on 8.5 fps^2 deceleration for a vehicle in the right-hand through traffic lane, 2.1 fps^2 acceleration for 30 mph arterial speed, and 1.7 fps^2 acceleration for all higher arterial speeds. Distances are measured between driveway centerlines.

TABLE 7 - Minimum Spacing of Driveways and Other Unsignalized Access Points to Alleviate Overlapping Right Turn Conflict Areas on Urban Arterials

Arterial Speed	Minimum Spacing	
	Glennon(11)	Stover(23)
30 mph	125 ft.	-
35 mph	150 ft.	160 ft.
40 mph	185 ft.	210 ft.
45 mph	230 ft.	300 ft.

SOURCE: References 11 and 23

Stover (23) provides spacing to allow the through vehicle to decelerate without changing lanes in order to avoid collision with a vehicle entering the traffic lane from a driveway. Limiting distances are measured between driveway centerlines. The vehicle in the right-hand through traffic lane cannot change lanes and decelerates at an average of 6.0 fps^2 after a 2.0 second decision reaction time. The driveway vehicle completes the 90 degree right turn while it accelerates from 0 mph to a speed equal to that of the decelerated through vehicle, at an average of 3.1 fps^2. No additional clearance is provided between the driveway vehicle and the through vehicle. The implied speed differentials which result between the driveway vehicle and the through vehicle(s) are:

Arterial Speed	Maximum Speed Differential
30 mph	14
35 mph	19
40 mph	24
45 mph	29

Distances needed to allow a turning vehicle to make a turn from the through traffic lane without creating an excessive speed differential with following vehicles are shown in Table 8 (23). The 1.5 second decision (perception and reaction) time used for the limiting condition is less than the AASHTO recommendation (3.0 seconds) for stopping sight distance. The distances given in Table 9 might be considered shorter than desirable but are considerably greater than those usually found on urban arterial streets.

Marks (29) advises that if there are at least 600 feet between driveways, vehicles can be absorbed into the through traffic stream under various flow conditions with little interference.

TABLE 8 - Minimum Distance to Allow Automobile to Turn from Right-Hand Lane on Urban Arterials

Arterial Speed	Minimum Distance (Feet)	
	Desirable	Limiting Conditions
35 mph	460	300
40 mph	510	378
45 mph	560	460

Desirable Distances based on:

3.0-second perception and reaction time

8 fps^2 maximum deceleration

3 fps^2 deceleration while moving laterally

3 fps lateral movement

10 mph speed differential

Limiting Conditions Distances based on:

1.5-second perception and reaction time

9 fps^2 maximum deceleration

5 fps^2 deceleration while moving laterally

4 fps lateral movement

15 mph speed differential

SOURCE: Reference 23

MEDIAN CONTROL OF LEFT TURNS

Left turn maneuvers have been involved in a disproportionately high percentage of median type crossing and turning accidents. For streets without medians or sufficient left turn storage provisions, they delay through traffic and reduce street capacity. Cribbins et al.(17), using 21 months of accident data from 388 miles of divided urban and rural highways in North Carolina, found that left turn, rear end accidents can be greatly reduced by construction of median area storage lanes. He indicates that median openings are not necessarily hazardous under conditions of low volume, wide median, and light roadside development. As volume and development increase, the frequency of median openings has a significant effect on increasing accident potential.

During a study of multilane highways in North Carolina, Cribbins et al. (30) found that injury accidents and total accidents are closely related and can be predicted from each other. Using a linear multiple regression equation, he found that 69 percent of the accident variance could be explained by five independent variables:

1. Posted speed limit

2. Traffic volume

3. Number of signalized openings per mile

4. Level of service

5. Access-point index

The "level of service" was defined as the minutes per mile obtained by dividing travel time by length of route segment. The "access-point index" was defined as an estimate of all movements per mile entering and leaving private driveways, intersecting roadways, and commercial and industrial developments. This research also found that median openings are involved in about 35 percent of accidents occurring between intersections on four-lane divided highways. As shown in Table 9, the largest percent of accidents at median openings involve vehicles attempting to cross four lanes through a median opening. This research concluded that whenever storage lanes are installed at median openings, the median opening accident rate is no longer significantly affected by the number of openings (excluding intersections), median width, speed limit, or traffic volume.

TABLE 9 - Frequency of Median Opening Accidents by Accident Type

Accident Type	Accidents Number	Accidents Percent
Hit while attempting to cross four lanes	899	38.5
Hit from front while turning through opening	589	25.5
Hit from rear while turning from outside lane	455	19.3
Hit from rear while turning through opening	297	12.9
Hit from rear after turning through opening	88	3.8
All Types Totals	2308	100.0

SOURCE: Reference 30

Results of a similar study, reported by the Los Angeles, Calif., area Chapter of the Institute of Traffic Engineers (31), indicated that properly designed left turn median channelization will generally reduce head-on, left turn, rear end, and opposing sideswipe accidents.

A before and after accident study on a 4-mile section of street in Denver, Colo., reported by Thomas (32), found that channelized left turns achieved a 6 percent reduction in left turn accidents compared to the before condition with no median. This study also showed a 52 percent decrease in rear end accidents, as well as decreases in pedestrian accidents, parked car accidents, and accident severity.

A study reported by the American Automobile Association (33) found that 67 percent of the pedestrians injured by vehicles turning at intersections are hit by vehicles turning left. The driver of the turning vehicle is concerned with leaving the through lane while avoiding oncoming vehicles and fails to observe pedestrians.

CURBED MEDIAN SAFETY

Wilson (34) investigated 12 types of improvements at 1,160 different locations and reported a significant accident reduction with barrier medians and intersection channelization.

Box (35) analyzed the accident experience for a 2-year period at 1238 access points to streets in Skokie, Ill. The data, summarized in Table 10, illustrates the value of barrier medians in reducing driveway accidents.

Hanna (36) found that a 6 inch median curb was superior to curb heights of 4 inches or less as well as 8 inches or higher. He reported that where medians in the urban area were not curbed, damage to grass, trees, and shrubs was frequent. The control of parking was impractical, especially near churches and shopping centers. The occurrence of both angle and parallel parking in the median area caused confusion, congestion, and high accident rates.

CURBED MEDIANS COMPARED TO PAINTED MEDIANS

Frick (37) compared the accident experience on two multilane streets in Springfield, Ill. (Table 11). The accident rate on the street having a painted median (zebra stripe) with left turn bays at selected locations was 2.63 times that on the street having a curbed median and intersection channelization. A comparison of accident frequencies further defined the desirability of the median curb. As indicated in Table 11, the total number of annual accidents per mile with curbed medians was about one-third that of streets with painted medians.

TABLE 10 - Two-Year Driveway Accident Experience As Related to Median Control

	TYPES OF DRIVEWAYS			
	Service Station	Commercial & Industrial	Residential	Alley
Routes with Barrier Median Curb (Study Length, 5.8 Miles)				
Number of Driveways	25	30	244	13
Number of Accidents	0	5	6	0
Accidents/Driveway/Year	-	0.08	0.01	-
Routes with Non-Barrier Median Curb (Study Length, 33.9 Miles):				
Number of Driveways	150	422	325	29
Number of Accidents	51	234	17	6
Accidents/Driveway/Year	0.17	0.28	0.03	0.07
Ratio of Accidents Rates Barrier/Non-Barrier Median Curb	-	0.30	0.47	-

SOURCE: Reference 35

TABLE 11 - Comparison of Accident Experience Between Streets with Curbed Median and Painted Median

			ANNUAL		
Accident Location	Number of Accidents	Number of Openings	Accidents Per Opening	Accidents Per Mile	Accident Rate (Acc. Per MVM)
Intersections					
Curbed Median(1)	64	21	1.5	17	3.23
Painted Median(2)	105	14	3.8	35	5.74
Mid-Block (Other Than Driveways)					
Curbed Median(1)	19	8	1.2	5	0.96
Painted Median(2)	54	12	2.3	18	2.95
Private Drives					
Curbed Median(1)	3	56	0.03	0.8	0.15
Painted Median(2)	50	188	0.13	17	2.73
TOTALS					
Curbed Median(1)	86	85	0.5	23	4.34
Painted Median(2)	209	214	0.5	70	11.43

(1) Stevenson Drive 14,300 ADT, 1.9 mile length, two-year period
(2) MacArthur Boulevard 16,700 ADT, 1.5 mile length, two-year period
Accident Rate - Accidents per Million Vehicle Miles

SOURCE: Adapted from Reference 37

Frick advised that where a choice as to cross section exists, the primary benefits of using curbed medians and intersection channelization are operational safety and increased capacity. There are also the following advantages:

1. Smoothes and enhances the highway free flow traffic carrying ability.
2. Decreases conflicts by providing a positive separation of opposing lanes of traffic.
3. Permits the regulation of traffic, through the prohibition of certain movements.
4. Controls the angles of conflict more adequately.
5. Provides a protection and storage area for heavy vehicle directional movements.
6. Gives better indication to motorists of the proper use of travel lanes and intersections.
7. Provides an opportunity to favor a predominant movement.
8. Provides a protected area for the location of traffic control devices.
9. Controls the speed of turning vehicles through the intersection area.
10. Serves as a protected refuge area for pedestrians.

Frick concludes that the installation of curbed medians and intersection channelization will pay dividends far exceeding the original cost, mainly by substantially reducing certain types of accidents and increasing capacity.

A study by the California Division of Highways, reported by Moskowitz (38), compared medians of different design. Accident data were analyzed for 12 roadway sections having curbed medians and 9 sections having painted medians. The making of left turns was legal only at median openings for both median types. All sections were within developed areas. Accidents between intersections involving turning vehicles accounted for 2 percent of all accidents on sections with curbed medians and 5 percent of all accidents on sections with painted medians. It was concluded that the curbed medians had better accident experience in the cases studied. Results were not conclusive as to the relative merits of painted versus curbed medians.

DEPRESSED COMPARED TO RAISED MEDIAN

Depressed medians are usually preferred over raised medians because they provide better drainage. This includes areas for snow storage and a reduction in hazardous ice spots.

Garner and Deen (39) compared the accident histories of different median types and provided verification of generally recommended median widths and slopes. A major limitation of the analyses was the small number of possible combinations of median width and cross slope available for study. The analyses provided evidence from accident histories to support the general assumption that wider medians are safer medians. It was indicated that medians should be a minimum of 30 to 40 feet wide for high speed facilities. Flat slopes should be provided as 4:1 slopes are inadequate for medians less than 60 feet wide. There was an indication that 6:1 or flatter slopes should be used. Raised medians provided unsuitable vehicle recovery areas on rural highways and were undesirable from the standpoint of roadway surface drainage. The irregular Interstate highway medians that result from independent roadway alinement should be used only with adequate clear zones in the median. Shoulders 12 feet wide should be provided where guardrail is to be used.

Based on controlled tests, Stonex (40) recommended 6:1 slopes as a minimum to permit encroaching vehicles to recover safely.

CONTINUOUS TWO-WAY LEFT TURN LANES

A number of studies have used before and after accident data in the evaluation of continuous two-way left turn lanes. Horne and Walton (41) found that, where no median was previously provided, the installation of continuous two-way left turn lanes reduced total accidents by about 33 percent with reductions of 45 and 62 percent for head-on and rear end type accidents, respectively. Sawhill and Neuzil (42) reported that the head-on collision, which has been a primary concern among those considering the installation of the continuous two-way left turn lane, has proved to be an uncommon occurrence and of negligible concern.

Studies conducted in Seattle, Wash., and reported by Hall (43), indicated that installation of continuous two-way left turn lanes facilitated the movement of through traffic on streets which did not have a median. They provided a high degree of access service without an increase in traffic accidents. Consideration was given to the effect of the left turn lane on the accident experience along streets serving commercial and industrial areas. Two-way left turn lane usage varied from 3 percent of the total traffic in an industrial area to 23 percent on an arterial adjacent to a shopping center and commercial development.

Nemeth (44) presents a summary of literature concerning operational effects as well as safety aspects of continuous two-way left turn lanes. He also presents the results of before and after studies for three installations on State highways in three Ohio cities. He concluded that on U.S. 20, a previously undivided four-lane arterial in Painesville, Ohio, the conversion of two of the four lanes of this arterial into one continuous two-way left turn lane resulted in increased travel times, increased weaving, and some reduction in conflicts. It appears that the roadway access function was improved while a measurable deterioration of the movement function resulted. The two-lane roadway U.S. 42 in Mansfield, Ohio, was converted to two traffic lanes plus a continuous two-way left turn lane. Nemeth concluded that the introduction of the left turn lane, by narrowing each of the two through lanes by about 4 feet, considerably improved the safety of the roadway and resulted in a moderate increase in running speeds. Safety was evaluated in terms of vehicular conflicts. The number of conflicts on U.S. 42 immediately after the conversion were 35 percent lower than in the before period. Total conflicts 6 months after the conversion were 42 percent of those in the before period.

Analysis by Glennon et al. (11) also found that the continuous two-way left turn lane is inferior to the raised median where frequent driveways are in combination with high arterial street volumes. His estimates found it to be a more effective accident reduction technique at lower levels of roadside development and traffic volumes as reflected in the tabulation below:

Conditions		Estimated Annual Accident Reduction Per Mile	
Level of Roadside Development	Highway ADT	Raised Median Divider	Continuous Two-Way Left-Turn Lane
Low <30 driveways Per Mile	Low <5,000	2.2	4.4
High >60 driveways Per Mile	High >15,000	31.2	28.6

MEDIAN WIDTH CONSIDERATIONS

Priest (45) has shown the value of having a median of sufficient width to "shadow" a left turning or crossing vehicle on a major roadway. Accident frequency showed an inverse relationship to the median width and magnitude of an exposure index, a measure based on arterial ADT, cross street ADT, and the exposure time of a crossing vehicle.

Telford (46) investigated the advantages of narrow medians . A 4-foot median separating two 33-foot roadways was installed on a major street through a central business district. Head-on collisions were reduced 65 percent after median installation. The median reduced both the total number and the severity of accidents. The median also provided a pedestrian refuge area, reducing the pedestrian accident rate by 70 percent.

CONFLICT CONTROL FOR COMMERCIAL DRIVEWAYS

Glennon et al. (11) evaluated 70 techniques and developed guidelines for the control of direct access to arterial highways. He made use of prediction equations developed by Mulinazzi and Michael (12), McDonald (47), and Webb (48) to estimate the accident reductions for each of the techniques. This section cites the estimated accident reductions that might be expected if use were made of the following techniques to control conflicts at commercial driveways.

1. Install a raised median divider with left turn deceleration lanes. (Table 12)
2. Install traffic signals at high volume driveways. (Tables 13 and 14)
3. Install two-way, continuous, or alternating left turn lanes. (Tables 15 and 16)
4. Install left turn deceleration lane in median in lieu of a right-angle crossover. (Table 17)
5. Offset opposing driveways. (Table 18)
6. Install right turn acceleration or right turn deceleration lanes. (Table 19).

Reductions in vehicle delay and benefit/cost ratios were also estimated for the developed techniques.

TABLE 12 - Estimated Annual Accident Reduction (Per Mile) by Installing Raised Median Divider with Left Turn Deceleration Lanes

LEVEL OF DEVELOPMENT (Driveways per Mile)		HIGHWAY ADT (Vehicles per Day) LOW <5,000	MEDIUM 5-15,000	HIGH >15,000
LOW	<30	2.2	4.1	6.3
MEDIUM	30-60	5.8	11.2	17.2
HIGH	>60	10.7	20.7	31.2

SOURCE: Reference 11

TABLE 13 - Estimated Annual Accident Reduction (Per Driveway) by Signalizing (Two-Phase) Commercial Driveways (Three-Way)

DRIVEWAY ADT (Vehicles per Day)		HIGHWAY ADT (Vehicles per Day) LOW <5,000	MEDIUM 5-15,000	HIGH >15,000
LOW	<500	0.12	0.20	0.28
MEDIUM	500 - 1500	0.28	0.49	0.67
HIGH	>1500	0.43	0.76	1.02

SOURCE: Reference 11

TABLE 14 - Estimated Annual Reduction (per Driveway) by Signalizing (Three-Phase) Commercial Driveways (Three-Way)

ORIVEWAY ADT (Vehicles per Day)		HIGHWAY ADT (Vehicles per Day) LOW <5,000	MEDIUM 5-15,000	HIGH >15,000
LOW	<500	0.17	0.30	0.42
MEDIUM	500 - 1500	0.42	0.74	1.00
HIGH	>1500	0.65	0.14	1.53

SOURCE: Reference 11

TABLE 15 - Estimated Annual Accident Reduction (per Mile) by Installing:

Two-Way Left Turn Lane

<u>or</u>

Continuous Left Turn Lanes (for Each Direction of Traffic)

LEVEL OF DEVELOPMENT (Driveways per Mile)		HIGHWAY ADT (Vehicles per Day) LOW <5,000	MEDIUM 5-15,000	HIGH >15,000
LOW	<30	4.4	8.8	13.3
MEDIUM	30-60	7.1	13.9	20.9
HIGH	>60	9.7	19.0	28.6

SOURCE: Reference 11

TABLE 16 - Estimated Annual Accident Reduction (per Mile) by Installing Alternating Left Turn Lane

LEVEL OF DEVELOPMENT (Driveways per Mile)		HIGHWAY ADT (Vehicles per Day) LOW <5,000	MEDIUM 5-15,000	HIGH >15,000
LOW	<30	1.7	3.2	5.1
MEDIUM	30-60	3.5	7.1	11.6
HIGH	>60	6.4	13.3	21.0

SOURCE: Reference 11

TABLE 17 - Estimated Annual Accident Reduction (per Driveway) by Installing a Left Turn Deceleration Lane (in Median) in Lieu of a Right-Angle Crossover

DRIVEWAY ADT (Vehicles per Day)		HIGHWAY ADT (Vehicles per Day) LOW <5,000	MEDIUM 5-15,000	HIGH >15,000
LOW	<500	-	-	1.2
MEDIUM	500-150	1.3	2.2	3.0
HIGH	>1500	1.9	3.4	4.5

SOURCE: Reference 11

TABLE 18 - Estimated Annual Accident Reduction (per Driveway) by Offsetting Opposing Driveways

DRIVEWAY ADT (Vehicles per Day)		HIGHWAY ADT (Vehicles per Day) LOW <5,000	MEDIUM 5-15,000	HIGH >15,000
LOW	<500	0.4	0.7	1.0
MEDIUM	500-1500	0.9	1.7	2.3
HIGH	>1500	1.6	2.6	3.6

SOURCE: Reference 11

TABLE 19 - Estimated Annual Accident Reduction (per Driveway) by Installing:

Right Turn Acceleration Lane (Exiting Driveway)

<u>or</u>

Right Turn Deceleration Lane (Entering Driveway)

DRIVEWAY ADT (Vehicles per Day)		HIGHWAY ADT (Vehicles per Day) LOW <5,000	MEDIUM 5-15,000	HIGH >15,000
LOW	<500	0.02	0.03	0.05
MEDIUM	500-1500	0.05	0.08	0.11
HIGH	>1500	0.07	0.13	0.17

SOURCE: Reference 11

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CHAPTER 5 - INTERSECTIONS

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INTRODUCTION

Even though an "intersection" is defined by Webster as the place where two roadways cross, the term as used in this synthesis refers to an "intersectional area." This includes not only the intersection proper but also the approaches in which intersectional maneuvers such as lane changing and deceleration take place. Intersections can be described as including approaches such as channelization and other intersectional geometry.

Although intersections comprise very small parts of rural highway networks or urban street systems about half the urban accidents and 24 percent of rural accidents occur at intersections. Data from a number of countries show that over the years the number of accidents at intersections has increased at a faster rate than other accidents. Ninety-four percent of urban and 88 percent of rural intersectional accidents involve two or more vehicles. Over one-third of the fatal accidents at urban intersections resulted in the deaths of pedestrians (1, 2).

High accident rates at these locations are to be expected, however. Intersections are the places where continuity of travel is interrupted, where traffic streams cross, where many types of turning movements occur. They are the places where traffic conflicts are concentrated and, therefore, where traffic safety countermeasures should have high priority.

Causes of intersection accidents were identified in 1979 as part of an extensive research project performed by Stanford Research Institute (3). Over one-half the accidents studied resulted from human failure. The remainder were caused by the environment or by a combination of human and environmental factors.

The Stanford study found that 15 percent of all accidents could be attributed to the physical environment and/or the method of traffic control and enforcement. The following pages are directed toward how these factors are related to traffic safety at intersections.

Many of the studies referenced herein compared accident frequencies, severities, and other characteristics before and after traffic safety countermeasures had been implemented. In most of the remaining studies, traffic and accident patterns were compared between groups of intersections having different geometrics or types of traffic control. These research studies show that various countermeasures can reduce accident experience.

It is suggested, however, that findings of before and after studies be carefully appraised. A statistical phenomenon termed "regression of the mean" can bias results and can indicate benefits following treatment even if the treatment or countermeasure was totally ineffective.

Additional cautions which should be considered in evaluating intersection safety improvement countermeasures were presented in the National Highway Safety Needs Study - (1976) (4).

- o The expected effects of each countermeasure are reduced to one single-valued estimate which may be, in fact, very subjective. This single value represents a guide or a benchmark for comparison purposes, but cannot portray the expected range of variations resulting from different applications.

- o Research and experimentation which led to the effectiveness estimate were usually conducted on a limited scale. Generalization of these results as national experience requires conclusions which extrapolate beyond the range of the data; such extrapolations are risky.

- o The effectiveness of any countermeasure is highly dependent on the specific characteristics of the site where it is employed or to what segment of the driving population it is aimed. It is also dependent upon what other complementary countermeasures may be initiated or already in effect.

The "Additional References" is a selected list of manuals, handbooks and other publications used by those responsible for the design and operation of roadways, including intersections. These publications are typically developed by agencies, organizations, or societies for the purpose of promoting uniformity in design and operation.

The effect of the growing disparity between the physical characteristics of automobiles and transit and commercial vehicles on the safe operation of intersections has generated many concerns:

- o The confined space allotted drivers in small vehicles and the need for clutching and gear-shifting could distract drivers' attention when intersection maneuvers are necessary.

- o The bulkiness of trucks and buses seriously limits the vision of drivers of small automobiles.

- o Collisions between small and large vehicles will have more serious consequences than between larger vehicles of similar size.

A second trend has been a shift in the types of accidents in which deaths occur. In the 1950's the majority of fatalities were in single car crashes; by the 1970's the majority were in multicar accidents. This shift plus the increasing number of collisions between small and large vehicles are matters of serious concern (5). Both of these trends have generated considerable research directed toward reducing the severity rather than number of accidents through new vehicle designs and various passenger restraints.

To complement research efforts in vehicle design and passenger restraints, research on intersection geometry and control countermeasures should also be accelerated in an effort to reduce the frequency of accidents. The changing characteristics of accidents, however, will probably require major changes in research methodology. Safer operations of intersections in the 1980's will present a formidable challenge.

THE PHYSICAL ENVIRONMENT

Stanford Research Institute (3) studied relationships between accident data and various parameters of intersection geometry and control for 558 intersections in the San Francisco, California, area covering a 3-year period. The study concluded that State highways had the highest accident experience, averaging 6.8 accidents per intersection per year. Arterial streets averaged 3.9 accidents per year and collector and local streets averaged 1.5 and 1.8, respectively. (Note: These figures refer to the average numbers of accidents and would require integrating with traffic volumes in order to compare accident rates.) The study indicated that there was practically no difference in the proportion of severe accidents on the various classes of roadways -- in each case 76 to 79 percent of accidents involved property damage, 9 to 11 percent resulted in minor injuries, 8 to 12 percent in moderate injuries and 2 to 3 percent in severe injuries. In all four roadway classes less than 1 percent of accidents resulted in fatalities.

INTERSECTIONAL GEOMETRY

Accidents per intersection per year differed considerably at intersections of varying size as shown in Table 1. A consistent pattern of higher accident occurrence for narrow streets was not evident in the data. The larger annual number of accidents at intersections of wider roadways is probably due, in some part, to the higher volume of vehicles entering the larger intersections.

TABLE 1 - Effect of Street Widths and Daily Traffic Volumes on Annual Accidents at Intersections

Street Width: Minor Street	Street Width: Major Street	Daily Entering Vehicles Per Intersection	Average Accidents Per Intersection Per Year
less than 20 feet	less than 20 feet	Predominately 0 - 10,000 ADT	1.3
less than 20 feet	20-40 feet	Predominately 0 - 10,000 ADT	3.2
less than 20 feet	more than 40 feet	Predominately 0 - 20,000 ADT	4.5
20-40 feet	20-40 feet	Predominately 0 - 20,000 ADT	2.3
20-40 feet	more than 40 feet	Predominately 5,000 to over 20,000 ADT	5.0

SOURCE: Reference 3

Table 2 shows the variations in annual accidents at stop sign controlled intersections as a function of intersection geometry. Total accidents were found to vary only slightly between intersections with three approaches (T or Y) and those with four approaches (cross), except at those with very large traffic volumes.

TABLE 2 - Effect of Intersection Geometry and Daily Traffic on Number of Annual Accidents (Stop Sign Control)

Intersection Type	ADT: Less than 5,000	5,000-10,000	10,000 20,000	Over 20,000
Cross	1.3	1.9	3.0	8.0
T or Y	1.3	1.6	2.7	4.2

SOURCE: Reference 3

Although T intersections have generally been thought to have considerably lower accident rates than cross intersections, a 1976 study of 232 intersections in rural municipalities of Virginia (6) indicated the rates at both T and non-T intersections were small with T-intersection rates markedly less than others, as shown in Table 3. The variation in accident type and rate with intersection geometry and traffic control is shown in Table 3.

TABLE 3 - Variation of Accident Type and Rate with Intersection Geometry and Traffic Control - Rural Municipalities

Intersection Geometrics and Control	Accident Type and Percent of Total: Rear End	Angle	Side-swipe	Other	Accident Rate*
Cross					
Signals	40	40	11	9	1.47
Stop Sign	22	59	10	9	1.27
T					
Signals	58	25	11	6	0.82
Stop Sign	28	43	12	17	0.79
Y					
Signals	42	29	25	4	1.40
Stop Sign	66	23	4	7	1.04
Offset					
Stop Sign	34	30	13	23	0.76

*Accidents per million entering vehicles

SOURCE: Reference 6

CHANNELIZATION AND LEFT TURN LANES

Where traffic is heavy, where a large number of turns must be accommodated, or where the intersectional area is large and vehicles should therefore be directed through clearly defined paths, channelization is usually considered.

The effectiveness of various safety improvement projects was evaluated in the early 1970's by Dale of the Federal Highway Administration (7, 8). He found that channelization of intersections produced an average 32.4 percent reduction in all types of accidents. Accidents involving personal injuries decreased by over 50 percent. An analyses in 1978 by Strate (9) of the impact of 34 types of safety improvement projects indicated that intersection channelization projects had produced an average benefit/cost ratio of 2.31.

Establishment of left turn lanes is fundamental to most channelization projects. A California study (10) of 53 safety improvement projects showed that reduction in accident rates at unsignalized intersections was significantly greater with use of raised barrier left turn lanes than with painted left turn lanes. The findings are summarized in more detail in Tables 4 and 5.

Table 4 indicates that in urban areas the raised barrier protected left turn lane was much more effective than a painted left turn lane.

As shown in Table 5, there was relatively little difference in the effectiveness of the raised barrier protected left turn versus the painted left turn in rural areas. Both treatments, however, provided a significant reduction in accident rates.

A comparison of accident reduction resulting from left turn channelization at signalized versus unsignalized intersections is shown in Table 6.

As Table 6 indicates, the benefits of adding left turn lanes are dependent, to some extent, upon whether intersections with left turn lanes are signalized. Research by the Ohio Department of Transportation (11) (1973) pointed out the safety advantages of left turn lanes and the effect of traffic signal control as summarized in Table 7. (It was not stated whether there were separate left turn phases in the signal cycles.)

The effect of the inclusion of a left turn phase in the traffic signal cycle, which also is a consideration in evaluating the effectiveness of left turn lanes, will be discussed later in the section on Traffic Signal Control.

Further evidence of the safety effectiveness of traffic signals was demonstrated by the Federal Highway Administration (8) in 1973. The installation or modernization of traffic signals was found to reduce the number of accidents and their severity as shown in Table 8. However, significantly better results were obtained when such projects included geometric improvements such as intersection channelization.

TABLE 4 - Accident Rates Before and After Adding Left Turn Channelization at Unsignalized Intersections in Urban Areas

	Raised Barrier Protected Left Turn Lane			Painted Left Turn Lane		
	Rate Before	Rate After	Percent Change	Rate Before	Rate After	Percent Change
Accident Type						
Single Vehicle	0.10	0.07	-30	0.12	0.12	
Left Turn	0.08	0.07	-12	0.27	0.21	-22
Rear End	0.80	0.06	-92 S	0.50	0.16	-68 S
Crossing	0.01	0.11	+1000 S	0.21	0.39	+86 S
Other	0.15	0.04	-73 S	0.07	0.12	+71
Severity						
Property Damage	0.73	0.24	-67 S	0.64	0.64	
Injury	0.42	0.08	-81 S	0.52	0.34	-35
Fatal	0.00	0.01		0.01	0.01	
Light Conditions						
Day	0.82	0.24	-71 S	1.00	0.74	-26
Night	0.32	0.09	-72 S	1.53	1.54	
TOTAL	1.14	0.34	-70 S	1.17	1.00	-15

Accident rates are the number of accidents per million entering vehicles

(Changes indicated with "S" are significant at the 0.10 level using the Chi-Square test)

SOURCE: Reference 10

TABLE 5 - Accident Rates Before and After Adding Left Turn Channelization at Unsignalized Intersections in Rural Areas

	Raised Barrier Protected Left Turn Lane			Painted Left Turn Lane		
	Rate Before	Rate After	Percent Change	Rate Before	Rate After	Percent Change
Accident Type						
Single Vehicle	0.10	0.07	-30	0.10	0.15	+50
Left Turn	0.18	0.05	-72	0.28	0.15	-46
Rear End	0.49	0.02	-96 S	0.51	0.09	-82 S
Crossing	0.28	0.27	- 4	0.19	0.16	-16
Other	0.13	0.07	-46	0.07	0.03	-57
Severity						
Property Damage	0.72	0.34	-53 S	0.61	0.31	-49 S
Injury	0.39	0.15	-62 S	0.54	0.25	-54 S
Fatal	0.08	0.00	-100	0.01	0.01	-
Light Conditions						
Day	0.67	0.24	-64 S	1.18	0.55	-53 S
Night	0.51	0.24	-53 S	1.13	0.63	-44
TOTAL	1.18	0.49	-58 S	1.16	0.58	-50 S

(Changes indicated with "S" are significant at the 0.10 level using the Chi-Square test)

Accident rates are the number of accidents per million entering vehicles

SOURCE: Reference 10

TABLE 6 - Accident Rates Before and After Adding Left Turn Lanes at Signalized and Unsignalized Intersections

	Signalized			Unsignalized			All		
	Rate Before	Rate After	Percent Change	Rate Before	Rate After	Percent Change	Rate Before	Rate After	Percent Change
Accident Type									
Single Vehicle	0.07	0.09	+29	0.11	0.10	- 9	0.09	0.09	-
Left Turn	0.36	0.16	-56	0.19	0.12	-37	0.28	0.14	-50
Rear End	0.32	0.37	+16	0.61	0.08	-87	0.46	0.23	-50
Crossing	0.11	0.10	- 9	0.14	0.21	+50	0.13	0.16	+23
Other	0.14	0.10	-29	0.11	0.06	-45	0.12	0.08	-33
Severity									
Property Damage	0.62	0.48	-23	0.67	0.37	-45	0.65	0.42	-35
Injury	0.37	0.34	- 8	0.47	0.20	-57	0 42	0.27	-36
Fatal	0.00	0.01	-	0.02	0.01	-50	0.01	0.01	-
Light Conditions									
Day	0.94	0.73	-22	1.12	0.50	-55	1.03	0.62	-40
Night	1.12	1.00	-11	1.24	0.73	-41	1.17	0.86	-26
TOTAL	1.00	0.82	-18	1.16	0.58	-50	1.08	0	-35

Accident rates are the number of accidents per million entering vehicles

SOURCE: Reference 10

TABLE 7 - Effect of Traffic Signal Control and Left Turn Lanes on Accident Rates

	ACCIDENT RATES*			
	Unsignalized		Signalized	
Type of Accident	No Left Turn Lane	With Left Turn Lane	No Left Turn Lane	With Left Turn Lane
Left Turns	1.20	0.12	0.65	0.37
All Other	3.15 S	0.92 S	1.82 S	1.17
TOTAL	4.35 S	1.04 S	2.47 S	1.54 S

*Accidents per million entering vehicles

S denotes a difference that is statistically significant

SOURCE: Reference 11

TABLE 8 - Effect of Installation of Traffic Signals and Channelization on Severe Accidents

	Percent Reduction		
Type of Improvements	Injuries	Fatalities	Total Accidents
Traffic Signals	30.2	17.3	6.0
Traffic Signals plus Channelization	33.3	83.3	19.7

SOURCE: Reference 8

Visibility and Sight Distance

An evaluation of Federal Highway Safety Program projects indicated that, out of a total of 34 different improvement types, the improvement of sight distances at intersections was the most cost effective. Improvement benefits exceeded costs by a factor of five (9).

The accident rate at most intersections will generally decrease if and when problem sight obstructions are removed. A before and after study in Concord, California, illustrates this (12). Sight distances at five intersections improved. Total accidents at these intersections dropped from 39 in the year before to 13 in the year after obstruction removal, a 67 percent reduction. In the same study, many other intersections at other locations in Concord were improved by use of signal installation or modification, delineation striping, improved pavement markings, and increased police enforcement. Although all improvements resulted in a reduction in accidents, the greatest percentage of reduction was experienced at the intersections where the sight distances were improved.

Although the sample size in Concord was small, the study reveals the potential reduction in accidents that could be obtained if this type of obstruction removal program were widely implemented.

The Stanford Study (3) estimated the reduction in accidents which could be expected if right-angle sight distance could be improved at intersections with limited visibility. It was found that intersectional accident potential could be reduced 10 to 25 percent if visibility of an object on a cross road, as seen by a driver from a main road vehicle 50 feet from the intersection, could be increased from a point 20 feet from the intersection to one 50 feet away. (Figure 1 illustrates the 50-foot sight distance triangle.) Major additional benefit, however, would not be obtained unless visibility of objects along the side road was increased to more than 100 feet.

A 50-foot sight distance triangle was also considered optimum by 154 government officials in a study of sight distance obstructions on private property (13). The surveyed officials felt that a 30-foot sight distance triangle was adequate when the intersection was regulated by traffic control devices.

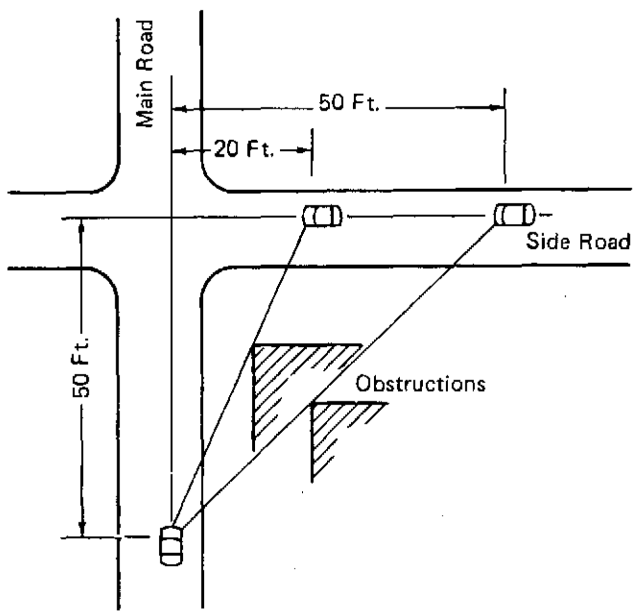

Figure 1. Typical Sight Distance Triangles

SOURCE: Reference 3

This study's recommendations for sight distance triangles require obstructions on the ground to be less than 2-1/2 feet high and overhead obstructions to be no closer than 8 feet from the ground. Allowable exceptions to these general rules include: small trees not exceeding 12 inches in trunk diameter, existing permanent buildings, existing grades which because of natural topography rise more than 30 inches above the center of the intersection, fire hydrants, utility poles, street markers and traffic control devices.

The study also presented ways in which private property owners might be persuaded to clear obstructions. Figure 2 shows an intersection sketch to be sent to property owners indicating "dangerous" and "safe" conditions. Most of the agencies and persons surveyed indicated that an ordinance or law would simplify sight distance control.

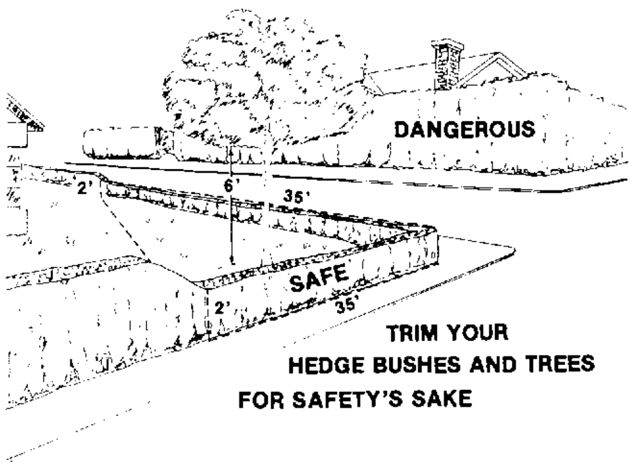

Figure 2. Intersection Sketch Sent to Property Owner

A 1973 study by the Michigan Department of Highways and Transport (14) also found that intersections with no sight distance limitations had significantly lower rates of accidents and severity than those with limited sight distance. It recommended that providing clear vision on State highway routes should be carefully considered in initial purchases of property for highways so that later negotiations with owners of property adjacent to intersections could be avoided. In growing areas, property along existing streets and highways should be acquired before development makes it difficult or impossible to do so.

The effect of poor sight distance and severe grade on accident occurrence was analyzed in the study of intersection accidents in rural Virginia municipalities (6). Results are shown in Table 9. The accident rate at intersections with severe grades was unexpectedly low as compared to an average rate of 1.13 for all intersections. Apparently drivers were aware of poor physical conditions and exercised more than average caution at those locations. On the other hand the high rate of accidents at places with restricted sight distances was due to the large number of angle collisions, a result of the inability of drivers to properly view vehicles approaching on cross streets. The researchers concluded that intersections with severe grades can operate safely under traffic signal control even though they have potential hazards.

TABLE 9 - Variation in Accident Type and Rate at Physically Deficient Intersections

Intersection Condition	Accident Type - Percent of Total: Rear End	Angle	Side-swipe	Other	Accident Rate*
Severe Grade	39	38	9	14	0.97
Poor Sight Distance	20	56	9	15	1.33

*Accidents per Million entering vehicles

SOURCE: Reference 6

LIGHTING

The same study of accidents in rural Virginia municipalities (6) found that about one-third of intersection accidents occur at night. There was less than 3 percent variance from this relationship for any traffic volume or pattern or roadway geometric category.

Data from the California Department of Public Works (10) at intersections for which illumination was being considered indicated that 56 percent of all accidents were occurring after dark. The night accident rate was 2.88 per million entering vehicles. Lighting of these intersections reduced the average night accident rate to a much more satisfactory 1.08. (Adding illumination to intersections of various geometrics (8) produced rather similar results as shown in Table 10.)

TABLE 10 - Effect of Intersection Illumination on Night Accident Rates

Intersection Type	Night Accident Rates*: Rate Before	Rate After	Percent Change
T or Y			
3-Leg	1.47	0.45	-69
Cross			
2-Lane Approaches	1.00	0.47	-53
4-Lane Approaches	0.53	0.20	-62

*Accidents per million entering vehicles

SOURCE: Reference 8

Table 11 indicates the effect of illumination on different accident types.

The California Department of Public Works established the following warrant for installing lighting: more than 5 accidents per year with more than 50% occurring at night, or less than 5 accidents per year of which 3 occurred at night. At intersections where illumination was warranted, the average night accident rate was 4.59. This was reduced 72 percent, to 1.28 when lighted. When intersections were illuminated where lighting was not warranted, accident rates dropped from 1.49 to 0.92, a reduction of only 38 percent.

In 1976 the Iowa Department of Transportation (15) analyzed the impact on night accident rates of adding illumination at rural intersections as shown in Table 12. The average reduction in night accident rates where illumination was installed was 51.9 percent. Daytime accident rates at those same intersections fell 12.7 percent during the same period.

A study of the effect of illumination on accident rates at rural Illinois intersections (16) compared accident data over a span of time equivalent to 445 intersection data years, 263 years of which were related to lighted intersections and 182 years related to unlighted intersections. The study found that the night accident rate was 45 percent less at lighted than at unlighted intersections. The ratio of night accidents to total accidents was 22 percent less at lighted than at unlighted intersections. The study also determined that night accident rates could be further reduced by including channelization with the illumination of intersections.

Researchers at Ohio State University studied in detail the characteristics of drivers approaching four intersections, each having a different treatment of illumination or special reflectorized delineators and signs (17). Data taken during 168 test approaches indicated that:

- The use of lighting significantly improved driving performance and earlier detection of the intersection by the driver.
- Signing and delineation had only marginal effects.
- New pavement markings had no effect.

Since the ratio of night to total accidents is lower at urban intersections, there is less research interest in impacts of lighting in urban areas. The Stanford study (3) included a brief analysis of the impact of illumination. The percent of total accidents that occurred at night was used as a basis of comparison. No difference in the ratio of night accidents was found with various degrees of illumination.

Illuminated intersections in urban areas did not have significantly lower percentages of nighttime accidents than those that were unlighted.

TABLE 11 - Effect of Intersection Illumination on Nighttime Accidents by Type

Accident Type	Percent Decrease In Nighttime Accidents
Single Vehicle	71
Multi-Vehicle	60
Left Turn	25
Rear End	57
Angle	57
Other	86

SOURCE: Reference 10

TABLE 12 - Reduction of Night Accident Rates With Variation in Intersection Types and Extent of Illumination

Intersection Type	Percent Reduction In Night Accident Rates
Channelized	56*
Non Channelized	44
With Route Turn	54*
Without Route Turn	49
3-Legs	28
4-Legs	62*
Extent of Illumination	
3 - 5 Lights	33
5 - 9 Lights	55*
10 - 15 Lights	71

* Significant at 99 percent level, before to after

SOURCE: Reference 15

Synthesis Chapter 12 - Roadway Lighting - includes citations of safety research that may be helpful when considering the overall aspects of lighting as related to the roadway and the intersection.

Pavement Surface Condition

Accidents due to skidding are significant in many parts of the country. Special surface treatments can be placed within and on the approaches of intersections to reduce such hazards. Where an experimental surface treatment was installed at an intersection in Lansing, Michigan, accidents were reduced 70%. During wet and icy periods rear end and right-angle collisions dropped almost 90% (18).

Research citations related to pavement surface conditions are included in Chapter 2 - Pavement Surface and in Chapter 11 - Adverse Environmental Operations. Material in these two chapters will be of interest with regard to improving safety on the approaches to intersections as well as within the immediate area of the intersection.

Fixed Objects

Fixed objects were involved in 5 percent of the 214 single-vehicle accidents included in the Stanford (3) study data. While the number of single-vehicle accidents at intersections was constant irrespective of the number of fixed objects in the intersection area, Table 13 indicates that the proportion involving fixed objects is related to the number of objects in the intersection.

Street Signs

A total of 525 intersections in the Stanford (3) study had street signs, most with 4-inch letters. Statistically significant differences were found at the 99 percent level in the accident rates between signs with white letters on dark background and those with dark lettering on white backgrounds. As shown in Table 14, signs with dark lettering on white background had lower accident rates. The authors postulated that clearer identification of street names offered by the light background signs permitted drivers to focus attention on negotiating intersection geometrics.

Bus Routes

Differences were found, in the Stanford (3) study, at the 95 percent confidence level, between the accident experience at intersections on and off bus routes. Where daily traffic volumes were 5,000 - 10,000 vehicles, accident rates were 5 percent higher at intersections on bus routes and 26 percent higher when there was a bus stop at the intersection. Increases were larger where traffic volumes were greater. In the 15,000 - 25,000 daily vehicle range, accidents were 13 percent higher on bus routes, and almost 120 percent higher where a bus stop was in the intersection.

TABLE 13 - Relationship of Fixed Objects at Intersections and Accidents

Average Number of Fixed Objects/Leg	Total Single-Vehicle Accidents/ Intersection/ Year	Fixed Object Accidents/ Intersection/ Year
Less than 5	0.68	0.04
5 to 7	0.70	0.08
More than 7	0.66	0.10

SOURCE: Reference 3

TABLE 14 - Comparison of Accident Experience With Street Name Signs (White Letters on Dark Background vs. Dark Letters on White Background)

Traffic Volume (Vehicles per Day)	Increased Accident Rate With Dark Background As Opposed to White Background	
	Day	Night
Less than 5,000	Little difference found	
5,000 - 10,000	25% more	40% more
10,000 - 20,000	17% more	50% more
Greater than 20,000	90% more	110% more

SOURCE: Reference 3

TRAFFIC CONTROL

Traffic control measures are used at intersections where traffic volumes or conflicts are sufficiently large to require the management of the flows of individual movements and/or where accident rates are undesirably high. Controls for the most part consist of the use of signs and/or signals.

Sign Control

A 1978 report by Roy Jorgensen and Associates (19) noted the following general safety aspects of sign controls at intersections:

1. Yield signs effectively reduce accidents at low volume isolated urban intersections.

2. Four-way stop controls significantly reduce accidents at intersections where entering traffic volumes on all approaches are relatively equal.

3. Four-way stop controls result in increased accidents where traffic volumes on approaches are not relatively equal.

Yield signs are used to regulate traffic at low volume intersections by assigning right-of-way to certain approaches. A comprehensive review of several studies (20) in 1977 found that accidents can be reduced from 20 to 60 percent by proper use of yield signs at low volume crossings. Little additional reduction is obtained if yield signs are replaced by stop signs on roads of very low volume. Based on total costs and benefits it was determined that yield signs would be justified at intersections with ADT's between 200 and 800, and stop signs warranted where ADT's are over 800. The authors found that the Hall study (21) indicated that, if a policy emphasizing use of yield or no sign control rather than stop signs was adopted for 120,000 unsignalized intersections in Indiana, an annual potential savings of several million liters of gasoline per year could be achieved.

A 1975 Kentucky study (22) which analyzed rural road accident records covering a 3-year period showed accident types at yield signs to be quite different from those at stop sign controlled intersections. At yield signs over half the accidents were rear end collisions, while angle collisions made up over half the accidents at stop signs as shown in Table 15.

Table 15 also summarizes results of a similar 1976 study which used data from rural towns in Virginia (6) but did not differentiate between accidents at yield signs and those at stop signs.

TABLE 15 - Accident Types at Sign Controlled Rural Intersections

STATE Control	Percent of All Accidents: Rear End or Side-swipe	Right Angle	Other	Accident Rate*
KENTUCKY				
At Yield Signs	56.2	22.5	21.3	N/A
At Stop Signs	29.6	51.9	18.5	N/A
VIRGINIA				
At Yield and Stop Signs	39	49	12	1.08

*Accidents Per Million Entering Vehicles

SOURCE: References: 22 and 6

The Virginia study also noted that accident rates at stop sign controlled intersections were lower at those intersections having high traffic volumes as shown in Table 16.

TABLE 16 - Relationship of Accident Rates to Traffic Volume Entering Stop Sign Controlled Intersections

ADT	Accident Rate*
Less than 10,000	1.12
10,000 - 15,000	1.05
15,000 - 20,000	0.97
Over 20,000	0.52

*Accidents per million entering vehicles

SOURCE: Reference 6

Four-way stop control is typically used where principal streets or highways intersect but where traffic signal installations are not warranted.

The City of Philadelphia (23) devised an accident warrant which defined an intersection to be dangerous when the accidents per year exceeded the average daily traffic in thousands. A program of four-way stop installations was initiated which not only reduced accidents but also reduced the need for new traffic signal installations. The study to determine the impact of this program included collection of data at 509 intersections controlled by stop signs, 154 of which were two-way stop installations. The average daily traffic at 154 two-way stops was 4,400, and 5,050 at 355 four-way stops. Comparisons of accident type and severity by type of control, based on a summary of 9 years of accident records, are shown in Tables 17 and 18.

TABLE 17 - Comparison of Accident Types at Two- and Four-Way Stop Controlled Urban Intersections

Type of Control	Accident Type - Percent of Total: Rear End	Side-swipe	Angle	Pedes-trian	Fixed Object
Two-Way Stop	11	5	51	12	21
Four-Way Stop	17	9	20	12	42

SOURCE: Reference 23

TABLE 18 - Comparison of Accident Severity at Two- and Four-Way Stop Controlled Urban Intersections

	Percent of All Accidents		
Type of Control	Property Damage	Occupant Injury	Pedestrian
Two-Way Stop	68	20	12
Four-Way Stop	78	10	12

SOURCE: Reference 23

The Philadelphia study (23) converted 222 intersections (having an accident rate of 9 or more) from two-way stop control to four-way stop control during the early 1970's. The general results of this study were:

1. Three out of every four conversions from two-way stop control improved conditions, regardless of the before accident rate.

2. Where two-way stop intersections with relatively low accident rates (less than 9.0 accidents per 10 million entering vehicles) were converted to four-way stops, intersection accidents increased in half of the cases.

3. Where two-way stop locations with relatively high accident rates (greater than 9.0 accidents per 10 million entering vehicles) were converted to four-way stops, intersection accidents were reduced in six of seven cases.

4. Total accidents decreased by 55 percent after conversion to four-way stop control.

5. Occupant personal injury accidents decreased by 81 percent after conversion.

6. Right-angle accidents decreased by 83 percent after conversion.

7. Rear end, fixed object, and sideswipe accidents were unchanged.

8. Pedestrian injury accidents decreased by 83 percent.

Stop signs are frequently suggested by the public to reduce vehicular speeds on local streets in residential areas. A 1976 study of vehicle speeds and stop sign observance in Troy, Mich., to determine if stop signs could reduce average travel speeds (24) revealed that speeds were not significantly changed from those before signs were installed. In some instances, speeds increased slightly.

In addition, stop sign compliance was poor. The number of vehicles making full stops at stop signs ranged from 2 percent to 51 percent. The number making no stop, not even a rolling stop, ranged from 15 to 47 percent.

Traffic Signal Control

Properly located and operated signals typically reduce the frequency of certain types of accidents, especially the right-angle type. However, some accidents, especially the rear end type, can significantly increase. These general concepts, known and accepted for a long period of time, seem to be consistently confirmed by research.

In 1975, a comprehensive review of research and statistical analyses of a large nationwide accident data base (25) led to the following tentative conclusions:

1. Signalization leads to a reduction in right-angle accidents and an increase in rear end accidents.

2. Signalized intersections have higher accident rates, but this is usually offset by less severity per accident, which leads to no significant change in total accident-related economic loss.

3. There appears to be no clear-cut evidence that the installation of signals will reduce the adverse effects of accidents. This appears to hold especially for those cases where signals would not be warranted.

4. As far as accident patterns are concerned, there is no clear-cut justification for lowering numerical warrant minimums for rural conditions. In fact, the effect of unwarranted signals is more adverse for rural conditions.

5. The number of right-angle accidents appears to be an insensitive indication of any expected improvement in accident patterns as the result of signalization. The right-angle ratio seems to be better suited to that purpose.

The study of intersection accidents in rural communities of Virginia confirmed previous findings on rear end and right-angle relationships and total accidents (6). An analysis of 2,301 accidents produced the results shown in Table 19. The findings in Table 19 are subdivided to show accident experience by type of intersection geometry in Table 20.

TABLE 19 - Variation in Accident Type and Rate With Type of Control -- Rural Municipalities

Type of Control	Accident Type - Percent of Total				Accident Rate*
	Rear End	Angle	Side-swipe	Other	
Traffic Signal	43	37	12	8	1.26
Yield or Stop Sign	29	49	10	12	1.08

*Accidents per million entering vehicles

SOURCE: Reference 6

TABLE 20 - Variation of Accident Type and Rate with Intersection Geometry and Traffic Control - Rural Municipalities

Intersection Geometrics and Control	Accident Type — Percent of Total				Accident Rate*
	Rear-End	Angle	Side-swipe	Other	
Cross					
Signals	40	40	11	9	1.47
Stop Sign	22	59	10	9	1.27
T					
Signals	58	25	11	6	0.82
Stop Sign	28	43	12	17	0.79
Y					
Signals	42	29	25	4	1.40
Stop Sign	66	23	4	7	1.04
Offset					
Stop Sign	34	30	13	23	0.76

*Accidents per million entering vehicles

SOURCE: Reference 6

The general findings were similar to those from other studies of the same nature -- that installation of traffic signal controls could result in slight increases in accident rates, significant increases in rear end accidents, and comparable decreases in angle collisions. Table 20 adds further confirmation of previous studies showing that accident rates at T-intersections are markedly less than those at other types of crossings.

Table 21 indicates that the Virginia study (6) of rural intersections did not confirm the common contention that accident rates are higher at signal controlled intersections where the traffic signals do not meet MUTCD warrants and/or specifications.

TABLE 21 - Variation in Accident Type and Rate At Intersections Where Traffic Signals Are Not Warranted

Intersection Characteristics	Accident Type - Percent of Total				Accident Rate*
	Rear End	Angle	Side-swipe	Other	
Meets Warrants					
Standard Display	45	35	12	8	1.26
Substandard Display	36	46	9	9	1.28
Below Warrant					
Standard Display	38	40	14	9	1.26
Substandard Display	33	48	3	16	1.23

*Accidents per million entering vehicles

SOURCE: Reference 6

The above findings might be suspected of bias because traffic volumes could be assumed lower at those intersections where signals are not warranted. However, analysis of the 232 intersections showed little variation in accident rate with changes in volume for signalized control as shown in Table 22. Some variation may be noted for sign control.

TABLE 22 - Average Accident Rate by Intersection ADT

	Traffic Control	Number of Rural Intersections	Average Accident Rate
<10,000	Sign	93	1.12
	Signal	15	1.33
10,000 to 15,000	Sign	47	1.05
	Signal	35	1.26
15,000 to 20,000	Sign	11	0.97
	Signal	12	1.09
>20,000	Sign	5	0.52
	Signal	14	1.26

*Accidents per million entering vehicles

SOURCE: Reference 6

Urban Intersection Traffic Control

The previously described study of urban accident characteristics in Philadelphia (23) also compared accident experience at signalized and stop sign controlled intersections. Results are shown in Table 23. Although these data reflect urban conditions, the results were similar to those reported from rural areas.

The Stanford study of intersections in the San Francisco Bay area (3) indicated that multiphase traffic signals appear to have lower percentages of fatal and injury accidents than do two-phase as shown in Table 24.

Research by the Kentucky Department of Highways in 1979 (26) also reported a decrease in accident severity of from 11 to 13 percent for accidents where multiphase signalization, rather than two phase, was used. There was an 85 percent reduction in total left turn accidents, offset in part by a 33 percent increase in rear end collisions. As a result of this study, the following accident warrants for left turn phasing were recommended:

For one approach -

4 left turn accidents in 1 year or
6 left turn accidents in 2 years.

For two approaches -

6 left turn accidents in 1 year or
10 left turn accidents in 2 years.

Left turn phasing also should be considered if a consistent average of 14 or more total left turn conflicts or 10 or more basic left turn conflicts occur in a peak hour.

In contrast to the above recommendations, the Los Angeles Area ITE Technical Committee (27) suggests that left turn phasing should not be added to signal cycles until less strenuous measures have been considered and rejected on the basis of factual engineering studies. The two most common problems -- excessive left turn accidents or delays -- should be determined by measurements (as opposed to projections) whenever possible. It should be noted that the Los Angeles recommendations appear to be directed toward the reduction of delays and the optimization of capacity rather than accident reduction.

The Yellow Indication

It has been postulated by some researchers that the critical time in the traffic signal cycle, when rear end and right angle accidents are generated, is when the yellow indication warns of the ending of the green phase. Motorists approaching an intersection when the yellow indication appears must decide whether to quickly stop and take a chance of being involved in a rear end collision, or to continue driving and thus risk a right angle collision with cross street vehicles.

The section of the approach to an intersection wherein drivers are required to make the critical decision as to whether or not to prepare to stop as the green phase comes to an end has been called the "decision zone." Understandably its location and extent varies with different approach speeds.

Blackman (28) using test subjects approaching an intersection mock-up, found an average reaction time of 0.8 seconds at the point half of the drivers decided to stop. Gazis (29) found an average reaction time of 1.14 seconds based on observations at actual intersections in Detroit. Jenkins (30) collected driver reaction time data at one intersection using time-lapse photography and found a mean of 1.16 seconds. The 85th percentile reaction time was 1.5 seconds.

TABLE 23 - Variation in Accident Type and Rate with Traffic Control Type -- Urban Intersections

	Accident Type - Percent of Total					
Intersection Type	Rear End	Angle	Side-swipe	Fixed Object	Pedes-trian	Accident Rate*
Two-Way Stop	11	51	5	21	12	1.5
Four-Way Stop	17	20	9	42	12	0.8
Traffic Signals	23	30	8	27	12	1.2

*Accidents per million entering vehicles

SOURCE: Reference 23

TABLE 24 - Effect of Multiphase Traffic Signal Phasing on Accident Severity

	Injury and Fatal Accidents Percent of Total	
Intersection Type	With Two Phase Signals	With Multi-Phase Signals
T Inter-sections	35	14
Cross Inter-sections	23	19

SOURCE: Reference 3

Olson and Rothery (31) observed driver stop-go decisions after yellow onset at two intersections. One had a 30 mph speed limit and the other had a 50 mph speed limit. They found that virtually all vehicles stopped when the required deceleration was 8 ft/sec^2 or less and continued through for required decelerations higher than 12 ft/sec^2.

A number of studies have investigated the effect of the length of the yellow on driving behavior but there is little evidence of its effect on safety. Data collected by Olson and Rothery (32) show that drivers tend to treat long yellows as extensions of the green. Comparing results between similar intersections with different yellow durations, they found that the probability of a driver stopping at a given point decreased as the length of the yellow increased. McGill (33) found that the number of drivers entering after the green increased with the length of the inter-green period. The amount of the period that was yellow did not matter. Fortuijn (34) found that lengthening the yellow did reduce the number of drivers running the red but did not increase safety.

Stimpson et al. (35) observed driver response at two suburban intersections where the yellow was increased about 1.4 seconds. The percentage of vehicles crossing after the signal changed to red was reduced from 15 percent to 1 percent with the longer yellow at one site and from 63 percent to 19 percent at the other. The data were taken immediately after the change. The long term effect on driving behavior, rear end conflicts, and accidents is not known.

Knoflacher (36) found a definite relation between yellow interval length and safety for intersections in Austria. For yellows greater than 4 seconds, accidents increased significantly with increases in yellow length. Volume was controlled in the study, but other factors such as approach speed were not.

Benioff (37) investigated the use of yellow intervals of the approximate same length at all signals in Fresno, California. A 3-year before period and 1-year after period were used. The overall accident rate barely changed with the uniform yellow. Injury accidents decreased but approach turn accidents increased. Right-angle accidents also increased in the central business district where most yellow intervals were increased.

Recent research has focused on advanced traffic controller and detector systems on high speed roads (38, 39, 40). In these systems a vehicle traveling through its "decision zone" on the green phase is detected and the controller, if other traffic characteristics at the intersection permit, extends the green phase of the signal cycle to allow it to pass through the intersection without being confronted with the untimely appearance of the yellow indication. At a trial installation in Georgia (38), vehicles running the red light decreased by 42 percent and vehicles accelerating through the yellow were reduced 33 percent. Abrupt and skidding stops were virtually eliminated as was braking followed by accelerating to get through the crossing before the green interval terminated. Zegeer (41) found similar results at five locations in Kentucky where the green-extension system was used. Before and after accident data were evaluated at three of the sites. Accidents were reduced 54 percent from 8.2 to 3.8 accidents per year. Rear end collisions were down 75 percent; right-angle collisions were reduced 31 percent.

ALL-RED INTERVALS

At some intersections, the yellow interval is followed by an all-red period, in which all traffic is simultaneously presented with a red signal.

Benioff (37) made a comprehensive study of adding an all-red clearance interval at 45 locations in 6 cities. There was a significant reduction in right-angle accidents from 219 to 130 the first year after the all-red intervals were installed. But not all cities had a decrease and the reduction was most marked at locations with a high rate of right-angle accidents. Rear end accidents increased slightly and turning accidents remained unchanged. All-red intervals did not have to be long to be effective. Right-angle accidents were significantly decreased for interval lengths less than the time required for a vehicle to pass through the intersection but not for longer intervals.

In 1973, Los Angeles (42) made a study of 36 high volume, high accident locations where all-red intervals were added. The before and after period ranged from 12 to 30 months. Total accidents decreased 19 percent from 983 to 800. The main reduction was in right-angle accidents which dropped from 271 to 161. Left turn accidents decreased and rear end accidents increased, but not significantly. Ten of the intersections had an increase or no change in accidents after the all-red was installed. Only at one intersection did the accident savings exceed the increase in delay costs.

Four years later, Hoppe (43) conducted a before and after study of all-red intervals at 148 intersections in Los Angeles. The average number of right-angle accidents per year were reduced from 2.94 before to 1.77 after. There was no significant reduction in left turn accidents.

The State of Michigan (44) made a study of the effects of adding all-red intervals at 17 locations that had a right-angle accident problem. The locations generally had high approach speeds or poor visibility. Total accidents were reduced 10 percent from 429 to

385. Right-angle accidents decreased from 141 to 75, but rear end accidents increased and left turn accidents remained unchanged.

The City of Portland (45) removed the all-red interval from 20 of its 525 signalized intersections. In the central business district where volumes were high and speeds were low, the accident rate decreased. At locations with high volumes and high speeds, the accident rate remained the same. At isolated intersections where speeds were high but capacity was seldom reached, the accident rate increased.

Flashing Traffic Signal Operation

Another aspect in the operation of traffic control signals was investigated in a 1972 project in Los Angeles County (California) (46) where many of the traffic signal installations are placed in flashing operation in the low volume early morning hours. It is presumed that motorists' irritation is high when they are forced to wait at a red light when there is no cross traffic, and that they might be inclined to drive through red indications.

A portion of the intersections previously under flashing operation during early hours were returned to normal continuous 24-hour red - yellow - green operation. Flashing operation was retained on the remainder for comparison. One year of "before" accident data and another year of "after" revealed that, where there was a relatively high accident rate in early morning hours, the change from flashing to pretimed cycle operation reduced both the incidence and severity of accidents. But where the previous accident experience was low the change from flashing to pretimed operation did not significantly change accident experience.

Right Turn On Red

While vehicles were allowed to turn right, after stopping, during red intervals at some traffic signals more than 40 years ago, the practice was generally ignored until 1937 when the State of California permitted this practice at intersections with authorizing signs. The California rule was changed in 1947 to permit right turns at all intersections except where specifically prohibited by signs. Although this practice spread slowly, right turns on red (RTOR) are now legal in most parts of the United States (47).

Advantages attributed to this practice include accident reduction and reduced energy consumption. Based on exposure, RTOR accidents occur equally or less frequently than right-turn-on-green accidents according to the experience of several cities as summarized in a comprehensive FHWA sponsored project (48). At intersections where the turn is permitted, RTOR accidents account for 0.61 percent of all accidents. The proportion ranged from 0.4 to 3.0 percent in 10 cities supplying data. An average rate for each city is shown in Table 25.

Table 25. Right-Turn-On-Red Accidents Versus Total Intersection Accidents

City	Number of Intersections In Study	Intersection Accidents Total	Intersection Accidents Percent RTOR
Los Angeles	3,235	41,316	0.7
Denver	1,059	7,431	0.7
Chicago	78	694	3.0
San Francisco	75	3,328	0.4
Portland	*	51,677	0.5
Jacksonville	405	1,756	0.7
Dade County	29	700	1.3
Omaha	26	497	2.2
Salt Lake City	24	600	1.3

*Not available

SOURCE: Reference 48

Where the practice is permitted only when authorized by signs, RTOR accidents were 2.91 percent of all accidents. Variations by city ranged from 2.7 to 3.3 percent. It should be noted that the data reflected motorist experience with a new practice and that reduction in RTOR accidents might be expected after motorists became more familiar with this form of operation.

Data from four cities, which include 53,879 accidents, indicated that 3.6 percent of all intersection pedestrian accidents involved motorists turning right on the red signal interval (under the generally permitted rule). Where right turns are permitted only when authorized by signs, 6.9 percent of the pedestrian accidents involved RTOR. Pedestrian accident experience varied widely in the cities studied.

RTOR accidents proved less severe than the average intersection accident. In four cities, the percent of RTOR accidents resulting in personal injury was in each case less than the percent of all intersection accidents in which people were injured. In Virginia the cost of all intersection property damage accidents averaged $538 per accident, but only $229 per RTOR accident.

The State of Colorado summary of 1970-75 RTOR accident types indicated that angle and same direction sideswipes predominated - 40.7 and 36.2 percent respectively. Rear end collisions were 13.6 percent of the total.

In another analysis 65 percent of RTOR accidents involved collisions between vehicles turning on red with those moving legally on the cross streets within their green phase of the signal cycle. These would not have occurred if the turning vehicle had properly yielded as required. In 18 percent of such accidents right turning vehicles struck left turning vehicles from the opposite direction that were moving on a left turn phase of the signal cycle. Five percent were rear end collisions occurring when motorists right turning on red stopped abruptly and were struck by vehicles from behind. The fourth major type of accident involved right turning vehicles striking pedestrians. Other types of accidents involving RTOR movements were relatively infrequent.

Because idling at intersections is shortened, RTOR saves fuel and reduces emissions. A computer simulation based upon an 18 intersection grid network indicated that RTOR could reduce fuel consumption by 2.6 percent on streets with bus operation and by 7.8 percent on those without. A Virginia study estimated the practice could reduce fuel consumption in that State by 3.6 million gallons of fuel per year.

Thus it has generally been the consensus of researchers using data from many localities in the United States that the RTOR practice reduces travel time, fuel consumption, and undesirable emissions while not significantly degrading the safety of signalized intersection traffic operation. But recent investigations by Galin and Baumgartner indicate that this practice could lead to a weakening of positive traffic control at intersections.

Dr. Galin further analyzed the data used in the FHWA research and that furnished by others. He questioned the interpretation or relevance of some of these data (49) and concluded that the practice could increase the number or severity of accidents. Dr. Galin presented opinion polls of drivers and pedestrians which indicated that a significant number of persons regard the practice as dangerous. He advocated continued study of this practice, based on surveys of uniform design that represent nationwide experience. His research also expressed concern that the permitting of RTOR could weaken conformance to traffic signal control. This question of compliance was addressed in a Maryland study (50) based on data collected during four succeeding time periods to detect changes in compliance during "learning periods." Analysis concluded that:

- Non-conformance to the requirement to stop was not only significant, but that it increased over time.
- While non-conformance was high, the number of unsafe turns was very low.

The Maryland study recommended that the places where RTOR is permitted (or specifically prohibited) should be established using well-conceived engineering warrants, that proper use be sufficiently understood and complied with by the motoring public, and that rules of operation should be adequately enforced.

There is a general agreement that there are conditions where RTOR should not be allowed (51). These would include:

1. Intersections where visibility is less than desirable sight distance minimum.
2. Intersections with more than four approaches or where geometrics cause additional conflicts.
3. Where there is an all pedestrian phase in the traffic signal control.
4. Where the intersection is within 200 feet of a railroad grade crossing and the signal controller is preempted during train crossings.

Flashing Beacons

The rules related to observance of flashing red indications are similar to those for stop signs and, under certain conditions, are used in conjunction with stop signs at isolated intersections or intersections having sight distance obstructions.

Results of a 1970 North Carolina State University study (52) of accidents before and after installation of flashers at stop sign controlled rural intersections are shown in Table 26. The authors state that there was a statistically significant decrease in accident rates on the aggregate sites, on three and four leg approaches and at channelized intersections. Most noticeable was the decrease in single vehicle accidents.

TABLE 26 - Change in Accident Experience with Addition of Flashers at Stop Sign Controlled Rural Intersections

Intersection Type	Percent Change: Total	Single Vehicle	Left Turn	Rear End	Angle	Other
4 Leg	-18	-62	-24	- 5	- 18	-
3 Leg	-65	-62	-	-100	-100	-50
Channelized	-47	-63	+70	- 63	- 50	-32
Non Channelized	+24	-50	+ 1	+ 3	+ 88	+32
TOTAL OF ALL	-27	-62	-13	- 33	- 21	-17

SOURCE: Reference 52

Results of a similar study in California (10) of changes in accident patterns as a result of installation of flashing beacons at stop sign controlled intersections can be summarized:

- o Total accidents decreased 43%
- o Single vehicle accidents decreased 67%
- o Left turn accidents decreased 39%
- o Rear end accidents decreased 17%
- o Angle accidents decreased 45%
- o Other two vehicle accidents decreased 47%

The severity of accidents was also reduced:

- o Property damage accidents decreased 34%
- o Injury accidents decreased 51%
- o Fatal accidents decreased 80%

There was a marked decrease in both day and nighttime accidents, those in the day decreased 43%, those at night decreased 46%.

A comparison of safety impacts for different types of flasher control is shown in Table 27.

It is interesting that the addition of four-way red flashers has an effect somewhat similar to that of traffic signal control: that angle collisions are reduced but rear end accidents increase significantly. The decrease in severity of accidents and in the number occurring in daytime and nighttime hours was quite similar to the averages previously described for all accidents.

Table 28 indicates that the California study did not find a significant difference in effect between flashers that were installed at channelized intersections and those at non-channelized intersections.

An interesting facet of the California study was a comparison of the impact on accident rates produced when four-way red flasher - four-way stop control was installed at intersections with various previous forms of traffic control. This is shown in Table 29.

The California study also analyzed the before and after severity of accidents, as a result of installing flashing yellow beacons at the approaches of intersections. While there was an increase in personal injury accidents, property damage accidents decreased 41 percent and there was a 100 percent decrease in fatalities.

TABLE 27 - Change in Accident Rates at Intersections With Addition of Flashing Beacons

	Percent Change		
	Red-Yellow Flashers		
Accident Type	3-Leg	4-Leg	4-Way Red Flashers
Single Vehicle	-29	-82	- 52
Multiple Vehicle:			
Left Turn	- 7	-44	- 82
Rear End	-46	-	+100
Angle	-33	-14	- 82
Other	-25	-63	- 73

SOURCE: Reference 10

TABLE 28 - Change in Accident Rates With Red-Yellow Flashing Beacons Added At Channelized and Non Channelized Intersections

	Percent Change	
	3-Leg	4-Leg
Channelized	-51	-26
Non Channelized	-54	-38

SOURCE: Reference 10

TABLE 29 - Change in Accident Rates When Four-Way Stop Control With Flashing Beacons Are Added to Intersections With Various Types of Traffic Control

	Percent Change				
	Accident Type		Severity		
Previous Control	Single Vehicle	Multiple Vehicle	Property Damage	Injury	Fatal
2-Way Stop	- 30	-71	-57	-71	-100
4-Way Stop	-100	- 7	+70	-65	-100
Red-Yellow Flashers	- 10	-87	-76	-95	-100

SOURCE: Reference 10

A comprehensive experiment at two sight-restricted rural intersections in central Maine (53) tested the effect of beacons versus standard signing on the approach speed of vehicles. At the intersection of a major with a minor roadway having two-way stop sign control, six alternative conditions were established as shown in Figure 3.

Sufficient data were gathered by roadside interviews with motorists and by electronic detectors to produce results significant to the 95 percent confidence level. Findings were summarized:

- The use of signs in condition 4 and 6 produced better recall of the signs and of the intersection;
- The more emphatic conditions (4, 5, and 6) were more effective in reducing speeds than the standard warning signs (conditions 1, 2, and 3). There was little difference in the effect produced by variations within each of the two groups. It can be generalized that conditions 4, 5, and 6 did significantly better in terms of awareness and actual speed reduction.

In earlier research at the Maine Facility, a 1977 study by Goldblatt (54) evaluated a flashing red beacon placed facing side road traffic. It produced lower speeds on the approaches when it operated continuously but not when it was actuated.

Research by King et al. (55) concluded that there does not seem to be any advantage in actuating advance warning beacons. This study, directed toward ways to improve the safety and capacity on two-lane rural highways, summarized the state of the art on flashing beacons. It concluded that in most of the nine studies reviewed, significant accident reduction had been obtained after the installation of standard flashing beacons at intersections. The authors state that beacons are often effectively used at intersections of deficient geometry where physical or economic restraints prevent correction through reconstruction.

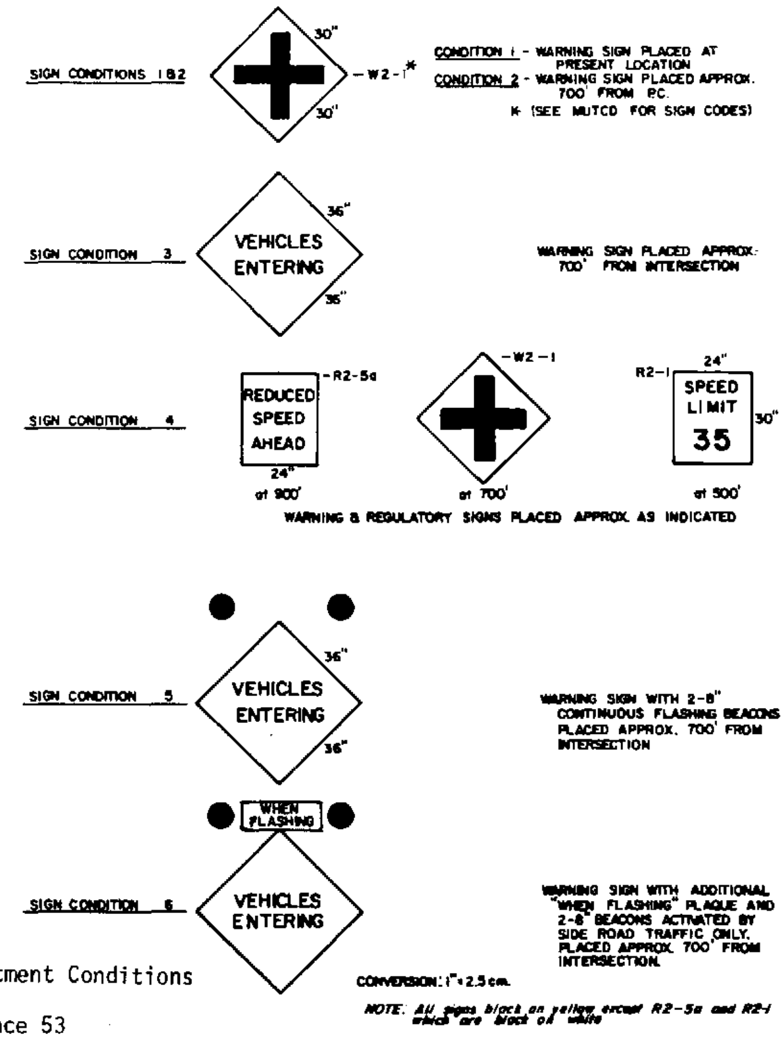

Figure 3. Treatment Conditions

SOURCE: Reference 53

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CHAPTER 6 - INTERCHANGES

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INTRODUCTION

The development of the Nation's freeway system has enabled transportation engineers to observe the safety and operational characteristics of freeways under a wide range of conditions. Freeway safety has now been observed to be largely affected by the design and operation of freeway interchanges.

A critical review has been conducted of research related to traffic safety at interchanges. This chapter provides information regarding the extent of the safety problems. The results of safety research performed to develop improvements to interchange safety are cited. Brief operational information is given only where necessary to describe the safety research situation being cited. References related to interchange planning, design, and operation for use as background information are listed with the "Additional References" at the end of the chapter.

GENERAL ACCIDENT EXPERIENCE

Interchange Spacing

The operational characteristics of a freeway depend to a large degree on the location and spacing of interchanges along the facility. Widely spaced interchanges do not adequately serve people living near the freeway nor develop the potential use of the facility. Too many interchanges in close sequence result in friction, inefficiency, loss of speed and capacity, and unsafe operations. The problem is magnified in built-up areas within cities where traffic demand is highly concentrated. Too close interchange spacing results in many "short trip" users who could have remained on the arterial street system.

A good indication of the relationship between interchange spacing and accidents is provided in research by Cirillo (1) as summarized in Table 1. The results reported indicate the proximity of a study unit to an interchange having a substantial effect on the accident rate. As the study unit along the freeway was located farther away from an exit ramp downstream or upstream, the accident rate decreased. This was particularly evident in urban areas, with a decreasing accident rate for a distance of approximately 2 miles from the ramp. The highest accident rate along the freeway occurred within 1,000 feet of an exit ramp nose or an entrance ramp merging end.

The tabular values in Table 1 portray much higher rates of accidents on urban than rural facilities. A pronounced variation in the range of accidents occurs along the freeway between interchanges in urban areas. Apparent is an overall decrease in accidents with an increase in interchange spacing. In rural areas, as compared to urban areas, the level of accidents is much lower at and between interchanges, and is considerably less sensitive to interchange spacing. A 1-mile interchange interval in urban areas shows an approximate range of 115 to 130 accidents per 100 million vehicle-miles. A 2- to 4-mile spacing in rural areas indicates a range of 70 to 80 accidents per 100 million vehicle miles. Comparison of data reveals a ratio of urban to rural accidents along freeways on the interstate system of about 1.7 between interchanges and close to 2.0 at and through interchanges.

Traffic Volume

The most important factor contributing to accident rates at and within interchanges is traffic volume. Cirillo et al. (2) developed a series of regression models from interstate accident and geometry data for various interchange types. These models are shown in Figure 1 and are cited at appropriate locations in this chapter as specific interchange types are discussed. Of all variables included in the models, mainline Average Daily Traffic (ADT) was found to be the most important predictor of accidents at interchanges. The geometric elements also included in the models are of elemental nature, such as lane width, shoulder width, lighting intensity, and presence of guardrails. An indication of the mainline ADT relation to accidents for seven of the interchange forms is presented in Figure 2.

In a study of cloverleaf interchanges, Foody and Wray (3) noted accident experience was strongly related to peak ADT on the main facility and ramps. A recent research study of entrance ramps reported by Transport Canada (4) found a strong correlation between the number of merging accidents per year on a ramp and the average daily ramp volume.

Lundy (5) reported 3 years of experience on 659 miles of four-, six-, and eight-lane freeways in California. The accident rates for each classification will normally increase with an

TABLE 1 - Accident Rate by Proximity to Interchange Exit and Entrance Ramps

EXIT SIDE			ENTRANCE SIDE		
Distance to down stream exit-ramp nose	Accidents (Number)	Accident Rate(a)	Distance to up-stream entrance-ramp nose	Accidents (Number)	Accident Rate(a)
URBAN			URBAN		
Less than .2 miles	722	131	Less than .2 miles	436	122
.2 - .4 miles	1,209	127	.2 - .4 miles	1,156	125
.5 - .9 miles	786	110	.5 - .9 miles	655	105
1.0 -1.9 miles	280	75	1.0 -1.9 miles	278	84
2.0 -3.9 miles	166	63	2.0 -3.9 miles	151	59
4.0 -7.9 miles	19(b)	69	4.0 -7.9 miles	200	75
RURAL			RURAL		
Less than .2 miles	160	76	Less than .2 miles	117	80
.2 - .4 miles	459	75	.2 - .4 miles	482	82
.5 - .9 miles	559	69	.5 - .9 miles	560	72
1.0 -1.9 miles	479	69	1.0 -1.9 miles	435	64
2.0 -3.9 miles	222	68	2.0 -3.9 miles	169	51
4.0 -7.9 miles	46	62	4.0 -7.9 miles	62	40

(a) Accidents per 100 million vehicle miles
(b) Small sample size

SOURCE: Reference 1

List of Model Equations

Y = Number of Accidents   N = Number of Observations

R^2 = Square of Multiple Correlation Coefficient

(1) Full Cloverleaf (with no collector-distributor roadway)

$Y = -3.7 + 1.3X - 0.025C$ $N = 186$ $R^2 = 0.80$

(2) Partial Cloverleaf

$Y = -1.6 + 0.24X + 2.9Z - 0.17F$ $N = 191$ $R^2 = 0.69$

(3) Three-leg or Trumpet

$Y = 0.41 + 0.20X + 0.17J$ $N = 160$ $R^2 = 0.53$

(4) Full Diamond

$Y = -1.0 + 0.31X + 2.0Z - 1.0A + 0.14B - 0.0045D - 0.11F - 0.11F - 0.51G + 0.61H$ $N = 681$ $R^2 = 0.89$

(5) Half Diamond

$Y = -0.64 + 0.15X + 1.27 + 0.50A + 0.14B - 0.0064D$ $N = 94$ $R^2 = 0.86$

(6) Full Slip Ramp

$Y = 2.9 + 2.0X - 0.067C - 0.0013E$ $N = 96$ $R^2 = 0.76$

List of Independent Variables

X = Average daily mainline traffic volume (thousands of vehicles)
Z = Average daily traffic volume exiting interstate (thousands of vehicles)
A = X · Number of businesses per one hundred feet on crossroad
B = X · Area type (1 = rural, 0 = urban)
C = X · Percent commercial vehicles, day
D = X · Percent commercial vehicles, night
E = X · Size of interchange (feet) ("Index" of area consumed by interchange)
F = X · Lighting intensity (foot-candles)
G = X · Type of crossroad (1 = divided, 0 = undivided)
H = X · Number of lanes in crossroad (1 = four or more, 0 = two)
J = X · Type of interchange (1 = trumpet, 0 = three-leg)

Figure 1. Model Equations for Estimating Annual Number of Accidents at Interchanges

SOURCE: Reference 2

increasing ADT. The rates of increase per 10,000 vehicle increase in ADT are four-lane, 0.240 accidents/MVM; six-lane, 0.094 accidents/MVM; and eight-lane, 0.078 accidents/MVM. As the ADT increases, the difference in rates between the three classifications becomes greater. This relationship introduces the possibility of significantly reducing the total number of freeway accidents by increasing the number of traffic lanes, even though the increase is not required by traffic volumes. Charles et al. (6) studied the effects of exiting vehicles on freeway operations. Observations of lane changing near exits found the percentage of exiting vehicles undergoing a "high accident risk" increasing dramatically as traffic volume increased. "High accident risk" was defined in terms of permissible braking delays of 1 second or less created by acceptance of short gaps.

Operational Uniformity

Important operational features of freeways are those communicative aspects which tend to clarify and simplify operations through uniformity of design and driver expectancy. One such significant characteristic is consistence in the design of successive interchanges along a freeway with respect to driver orientation and maneuver in exiting the freeway.

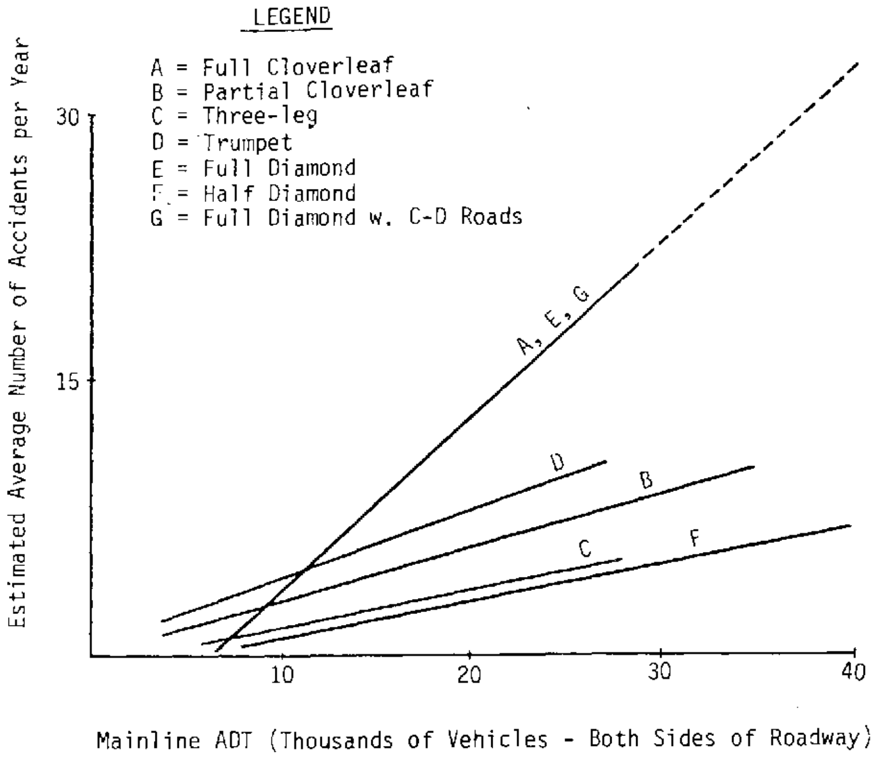


Figure 2. Comparison of Estimated Number of Annual Accidents by Interchange Type

SOURCE: Adapted from Reference 2

The principle of a single-exit design on the right, in advance of the crossroad to achieve uniformity was introduced by Leisch (7). While direct research on this specific configuration is not known to have been performed, indications of effectiveness of "single-exit in advance of crossroad" design can be derived from the literature.

Lundy (8) studied exits of diamond interchanges and those of cloverleafs with collector-distributor roads which represent a single exit on the right, in advance of the crossroad structure. Accident rates were found to be 0.67 and 0.62 per million vehicles, respectively. Cloverleaf loop ramps exiting on the right beyond the crossroad structure have accident rates of 0.88 per million vehicles. Substantially lower accident rates for diamond interchanges than for cloverleaf interchanges, as reported by Cirillo et al. (2), may be considered a rough indication of accident rates associated with single and multiple exit arrangements. A study in California by Johnson (9) shows nearly 65 percent of all the fatal accidents on California freeways occurring at grade separations involving abutments, piers, and railings as vehicles passed under or over the structures. This further highlights the vulnerability of ramps situated beyond the crossroad. As a result of diagnostic field studies in Texas, it has been postulated by Woods et al. (10), using the unfamiliar driver as the logical design driver, that all freeway exits would be expected on the right and the driver would expect to turn right in advance of the interchange structure.

INTERCHANGE TYPES

A wide variety of interchange forms is used on rural and urban freeway systems. Each form produces unique operating characteristics and applicability to certain situations. Factors which usually contribute to selection of an interchange type include types of intersecting facilities, traffic volume, nature and volume level of turning movements, location (rural vs. urban), and relationship to other nearby interchanges. Area development characteristics, available right-of-way, political considerations, and local citizen inputs are often factors. The sensitivity of many of these factors to the safety of an interchange was not known when much of the Nation's freeway system was built. In addition, changes in travel patterns have resulted in many freeway corridors carrying significantly higher volumes than were originally anticipated. Therefore, it has been possible to study the performance of various interchange types with respect to their accident potential under a variety of operating and environmental conditions.

A study by Lundy (8) provides one measure of the relative safety of different interchange types. He collected accident data for 10 basic ramp types. His findings, summarized in Table 2, indicate ramps of diamond and directional interchanges are the safest types. Loop and cloverleaf ramps without collector-distributor roads, trumpet, scissors and left-hand ramps have the highest accident rates.

TABLE 2 - Accident Rates by Type of Freeway Ramp

Ramp Type	Accident Rates* ON	OFF	ON & OFF
Diamond Ramps	0.40	0.67	0.53
Trumpet Loop Ramp	0.84	0.85	0.85
Cloverleaf Ramps	0.72	0.95	0.84
Cloverleaf Ramps With C-D Roads	0.45	0.62	0.61**
Loop Ramps Without C-D Roads	0.78	0.88	0 83
Cloverleaf Loops With C-D Roads	0.38	0.40	0.69**
Left Side Ramps	0.93	2.19	1.91
Direct Connections	0.50	0.91	0.67
Buttonhook Ramps	0.64	0.96	0.80
Scissors Ramps	0.88	1.48	1.28

* Per Million Vehicles (Rates Do Not Include Crossroad Accidents and Freeway Mainline Accidents Within the Interchange Area)
** On and Off Rates Include Accidents on C-D Roads
C-D Collector-Distributor

SOURCE: Reference 8

Cloverleaf Interchange

Cloverleaf interchanges have been widely used at freeway-freeway and freeway-arterial junctions in both rural and urban areas. After many years of operational experience, serious problems inherent in the form of the cloverleaf have resulted in the comparatively poor safety of this form of interchange.

The major safety problem associated with the cloverleaf, the combination of low speeds and short weaving distances has been extensively studied. This problem is particularly serious where cloverleafs are used as a major interchange, in which high volume, high-speed merges and diverges are expected.

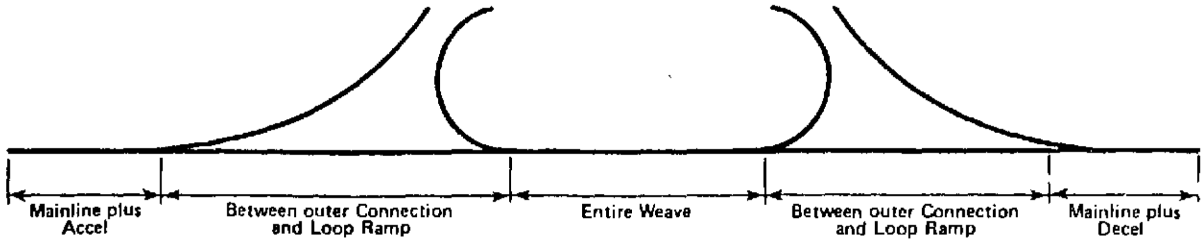
Tait et al. (11) studied a sample of cloverleafs identified by State highway agencies as being "problem locations." Common difficulties at these locations were inadequate weaving sections and acceleration/deceleration lengths. These conditions created weaving conflicts, queuing on the ramp, and use of the shoulder, all of which played roles in the high accident experience. Hansell (12) also noted a high incidence of weave-related accidents on three cloverleaf interchanges which had no collector-distributor roads.

Foody (3) did not find the cloverleaf design to have any one design feature, such as the loop ramp, which experiences an accident frequency disproportionate to that of the other design features. The increase in interchange accident frequency, experienced primarily on the mainline, occurs with increasing mainline traffic volume. The weave section, when defined to consist of both the acceleration half and the deceleration half, does experience the greatest increase in accident frequency with increasing ADT of the various mainline elements. This is not unexpected since this portion of the interchange must accommodate both entering, exiting, and through traffic. All of the mainline design elements experience significant increases in accident frequency. Thus the safety problem with the cloverleaf interchange is not an accident problem resulting from a hazardous design but a capacity problem resulting from a design inadequate for high speed, high volume operation. A series of models was developed, shown in Figure 3, to calculate the annual accidents expected for mainline design elements of cloverleaf interchanges.

The analysis performed by Cirillo et al. (2) as part of a study of the accident characteristics of Interstate Highways (as previously cited) includes in Figure 1 model equations for estimating the number of annual accidents for cloverleaf as well as other types of interchange.

Partial Cloverleaf Interchange

Partial cloverleaf interchanges of various types are found both on rural and urban freeways. The partial cloverleaf form is effective in situations in which turning movements off the crossroad are high. Partial cloverleaf forms in which the loop ramps are not adjacent and no weaving occurs provide an operational advantage over full cloverleafs.



y = Annual number of accidents per interchange

x_1 = Entering Volume (peak, one-way, ADT)
x_2 = Peak ADT on "mainline plus accel" section
x_3 = Peak ADT on OC Off-ramp
x_4 = Peak ADT on loop off-ramp
x_5 = Peak ADT on OC on-ramp
x_6 = Peak ADT on loop on-ramp
x_7 = Length (in 10's) of "mainline plus accel" section
x_8 = Length (in 10's) of OC off-ramp
x_9 = Length (in 10's) of "mainline plus decel" section
x_0 = Total length (in 10's) of all mainline sections

<u>"Mainline plus Accel" Section (R^2 = 0.442)</u>

$$y = -0.27849 + 0.0000128 x_7 x_1 - 0.0000050\, x_5 x_1 + 0.0000004\, x_1 x_1 + 0.005042\, x_5$$

<u>"Entire Weave" Section (R^2 = 0.656)</u>

$$y = 0.116565 + 0.0000056\, x_6 x_1 - 0.0000167\, x_4 x_1 + 0.001806\, x_1$$

<u>"Mainline plus Decel" Section (R^2 = 0.545)</u>

$$y = 0.097271 + 0.0000259\, x_9 x_1 + 0.0000026\, x_3 x_1 - 0.0000079\, x_8 x_1$$

<u>"Between OC and Loop Ramp" Section (R^2 = 0.417)</u>

$$y = -0.379804 + 0.002503\, x_1 - 0.0000046\, x_0 x_1$$

<u>Entire Mainline Considered as Single Unit (R^2 = 0.697)</u>

$$y = 0.917214 + 0.0000016\, x_1 x_1 + 0.009235\, x_3$$

Figure 3. Model Equations for Estimating Annual Number of Accidents for Mainline Design Elements of Cloverleaf Interchanges

SOURCE: Reference 3

While the partial cloverleaf is a common interchange form, limited research has been performed on its safety characteristics. A number of studies of wrong-way driving including Parsonson and Marks (13) and Scifres (14) have noted the potential for such movements on partial cloverleaf interchanges in rural areas. Cirillo's (2) model for accident predictions for partial cloverleafs is included in Figure 1.

Diamond Interchange

The diamond form is the most common interchange type. Diamond interchanges are found frequently in rural areas and to a considerable extent in urban areas. Safety problems associated with the diamond form relate mostly to the crossroad and its intersections with ramp terminals. Harwood (15) studied existing interchanges which underwent rehabilitation to solve safety and/or operational problems. Of 20 full diamond interchanges studied, 10 were found to have inadequate spacing between the intersection of a ramp and an adjacent local street. Eleven ramps experienced queuing problems caused by excessive delays to exiting vehicles. This situation created a rear end accident problem on the freeway and/or ramp at four of the locations. Wrong-way movements were identified as being a problem at two locations.

Studies by, Gabriel (16), Parsonson and Marks (13), Scifres (14), Tamburri and Theobald (17), and Vaswani (18) have investigated wrong-way movements at diamond interchanges. Tamburri and Theobald (17), reported 37.5 percent of wrong-way driving incidents on freeways occurring at diamond ramps. Half diamonds, because they do not provide for all movements, are especially prone to wrong-way movements. Scifres (14) also noted the susceptibility of diamond interchanges to wrong-way movements. Vaswani (18) studied 288 incidents of wrong-way driving observed in Virginia over 7 years on interstate highways where 117, or 41 percent resulted in an accident. Scifres (14) found 29 out of 37, or 78 percent of the wrong-way accidents on controlled access facilities in Indiana resulted in an injury or fatality. A number of countermeasures and design principles have evolved from the need to reduce wrong-way movements at diamond and other interchange types. Channelization of ramp terminals, freeway entrance signs, provision for crossroad medians, and careful design of the median openings have been noted as positive solutions to the problem.

Cirillo's (2) regression model for prediction of annual accidents includes diamond interchanges as shown in Figure 1. The independent variables include traffic volumes and the geometric and environmental characteristics.

"T" AND "Y" INTERCHANGES

"T" and "Y" interchanges, including the three-leg directional or trumpet form, are usually used at freeway-to-freeway interchanges. The trumpet form is also appropriate for arterial highway or street junctions with a freeway. A major problem with numerous existing three-leg directional interchanges is the use of left-hand ramps to reduce structural and right-of-way costs. The poor safety history of left-hand ramps will be cited later. Modern three-leg directional interchange designs utilize right-hand ramps, with apparent improved safety.

Cirillo et al. (2), in Figure 1, shows the accident rate at "T" and "Y" interchanges to be a function of traffic volume only. Lundy (8) in Table 2 indicates accident rates on trumpet ramps to be somewhat higher than on direct connections.

DIRECTIONAL INTERCHANGE

Directional interchanges on which the major turning movements are made on direct connections rather than loop ramps can take many forms. They can include collector-distributor roads and weaving sections.

Lundy's (8) analysis of individual ramps indicates the expected safety experience of directional interchanges; directional ramps and ramps along collector distributor roads have relatively low accident rates. Properly designed directional interchanges provide relatively safe movement of traffic as shown in Table 2.

MAJOR VERSUS MINOR INTERCHANGE

Major interchanges (freeway-freeway) typically involve high total traffic volumes and a higher proportion of high turning volumes.

Major interchange forms include directional interchanges, with high-speed ramp geometry. They may include loop ramps with and without internal collector-distributor roads.

Minor interchanges between freeways and collectors or arterials are usually of the diamond or cloverleaf interchange type. A relatively smaller proportion of the traffic through a minor interchange uses either exit or entrance ramps.

Using the FHWA data base, maintained for the Interstate System Accident Research Program, Taylor et al. (19) examined the differences in accident rates between major and minor interchanges. Findings shown in Table 3 indicate accident rates for major and minor interchanges have an irregular pattern when stratified by urban and rural locations. Accident rates at major interchanges are lower in urban areas than in rural areas, whereas the opposite is true for minor interchanges. Since turning volumes on major interchanges are usually much heavier than on minor ones, Taylor finds the major interchanges to be more hazardous due to the numerous merges and diverges. The analysis supports this except for the urban-minor case, where the rate is the highest of all four interchange classes. The explanation for this high rate lies in the types of ramp connections with the cross streets. These would typically be diamond connections with high volume, at-grade junctions controlled by signals or stop signs. Accidents in this area are classified as "interchange" accidents increasing the accident rate for the urban-minor interchange. Extremely low cross street volumes decrease the hazard at rural-minor locations.

TABLE 3 - Accident Characteristics of Major and Minor Interchanges

Interchange Type	Location	Number	Accident Rate*	Injury Rate*	Fatality Rate*
Minor	Urban	529	240	155	3.1
	Rural	1059	122	81	4.3
Major	Urban	15	174	112	2.9
	Rural	22	225	149	4.1

*Per 100 Million Vehicle Miles

SOURCE: Reference 19

Pigman et al. (20), in Table 4, summarize the differences in accident experience between major and minor interchanges in urban, suburban, and rural areas of Kentucky. The effects of volume and interchange spacing are partially responsible for the high accident rates associated with interchanges in urban areas. Interchange accidents were found to occur more frequently on the exit ramp than on the entrance ramp. On both the exit and entrance ramps, the largest number of accidents were of the rear end type. On entrance ramps, rear end accidents were the second most frequent, followed by angle accidents between a vehicle that was leaving the

TABLE 4 - Accident Summary From Kentucky Interchange Study

Population Areas	Number of Accidents	Number of Interchanges	Accidents per Interchange	Average AADT	Average Accident Rate*	Interchanges Per Mile
Urban	948	72	13.2	68,047	0.53	0.86
Suburban	82	20	4.1	31,678	0.36	0.32
Rural	114	79	1.4	17,638	0.22	0.16
TOTAL	1,144	171	6.7	40,502	0.45	0.26

*Accidents Per Million Vehicles

SOURCE: Reference 20

ramp and a vehicle on the main line. This merging created the largest number of accidents. On exit ramps, rear-end accidents were much more numerous than any other type. These accidents were caused, in most cases, by drivers who were not properly slowing when exiting. Some of the most severe accidents involved hitting fixed objects.

Summary of Interchange Type Safety

Figure 1, as mentioned previously, depicts the equations resulting from an analysis of the accident characteristics of various interchange types by Cirillo et al.(2). The regression equations permit predictions of annual accidents for the several types of interchanges discussed previously. Further perspective is given by the summary in Table 5 of accident rates for a number of interchange types.

The models may be used for direct comparison between interchange types because of the already noted strong relationship between traffic volume and accident rate.

TABLE 5 - Estimated Annual Accidents, Injuries, and Accident Rates by Interchange Type

Interchange Type	Annual Average Accidents Number	Annual Average Injuries Number	Accident Rate*
Full Cloverleaf with No C-D Roadway	19.3	12.9	1.69
Full Cloverleaf With At Least One C-D Roadway	14.3	6.2	1.45
Partial Cloverleaf	5 3	3.7	.94
Three-leg or Trumpet	4.0	2.6	.80
Full Diamond	4.2	2.8	1.02
Half Diamond	2.9	2.0	.25
Full Slip Ramp Diamond	9.9	5.5	1.23
Half Slip Ramp Diamond	4.9	1.9	.89

*Accidents per Million Vehicles

C-D - Collector-Distributor

SOURCE: Reference 2

INTERCHANGE ELEMENTS

An interchange can be described as a set of geometric elements, each with its own safety and operating characteristics. These elements include the exit ramp, entrance ramp, ramp proper, weaving sections, and the crossroad including its intersections with the ramps.

Research to date on the safety characteristics of specific interchange elements is limited. Cirillo et al.(2) advise the overpowering effect of traffic volume on accident rates makes identification of geometry effects extremely difficult. In addition, special factors often play a role. For example, the Cirillo study Table 5 shows the half-diamond to be the "safest" interchange type, although this interchange is not recommended because of the potential for wrong-way movements. The relative quality of geometry on the Nation's freeway system is high, resulting in further difficulty in identifying "hazardous" versus "safe" sets of conditions. As a result of these problems many researchers have concentrated on the operational effects of various interchange elements by observing erratic maneuvers, lag, gap times, and/or speeds. Studies involving safety related research performed in combination with operational observations and experience on interchanges, are cited in the following discussion of interchange elements.

Ramp Terminals

The arrangement of ramp terminals--including sequencing, spacing, and placement of entrances and exits--is an important determining factor in interchange operation and accident experience. Ramp sequences which create weaving sections, such as occur on cloverleaf interchanges, and designs mixing ramp terminals on left and right sides of the freeway can contribute to unsafe operations. Spacing of successive entrances or exits must take into consideration the lengths necessary to safely accomplish lane changing and deceleration or acceleration associated with diverging and merging.

EXIT RAMPS

One of the most critical elements of an interchange is the exit ramp, including the deceleration lane, gore area, and ramp proper. Good design of all these elements is required to enable drivers to place their vehicles in the proper lane in advance of the exit, leave the freeway easily at traffic stream speed, and decelerate at a comfortable rate to reach the ramp's posted speed.

The influence of the exit ramp has been shown to extend as much as 1 mile upstream of the point of divergence. A study by Charles et al. (6) of vehicle behavior upstream of exits found on six- and eight-lane freeways through vehicles moved to left-hand lanes (away from the exit lanes) with the greatest frequency 3,600 to 4,800 feet from the exit. Exiting vehicles change to the right lane at distances of 1,800 to 2,400 feet under low and medium volumes and up to 3,600 feet under high volume conditions.

The critical nature of the exiting maneuver at major interchanges is emphasized by Taylor et al. (19) who determined, on the average, deceleration areas (which include the exit ramp) experience a 44% greater accident rate than acceleration areas. Lundy (8) determined, on the average, exit ramps had a rate of 0.95 accidents per million ramp vehicles, compared to a rate of 0.59 for entrance ramps.

Deceleration Lanes

The most important element of the exit ramp, in terms of safe operation, is the length of deceleration lane available to drivers exiting the freeway. Cirillo et al. (2) studied a subset of ramps with extremely high accident rates.

The primary causes of such histories were high traffic volumes and insufficient deceleration lengths. These findings agree with Lundy (8), who noted exits with deceleration lengths greater than 900 feet had lower accident rates. Another study by Cirillo (21) of the same interstate data base indicates that a combination of high exiting traffic volume and short deceleration length produced very high accident rates. Table 6 demonstrates these findings.

TABLE 6 - Accident Rates for Deceleration Lanes Of Various Lengths

Length of Deceleration Lane	Percent Diverging Traffic					
	<2	2.0 to 3.9	4.0 to 5.9	6.0 to 7.9	8.0 to 9.9	>10
	Acc.* Rate	Acc.* Rate	Acc.* Rate	Acc.* Rate	Acc.* Rate	Acc.* Rate
<200	62	65	119	151	196	259
200 to 299	58	69	125	140	178	227
300 to 399	39	60	123	124	172	200
400 to 499	33	60	109	129	151	176
500 to 599	29	58	127	105	129	200
600 to 699	25	41	88	120	118	149
>700	39	48	79	111	112	148

*Number of Accidents per 100 Million Vehicles

SOURCE: Adapted from Reference 21

Single and Two-Lane Exits

Two-lane exits are frequently used to handle large volumes of exiting traffic. Operational research has shown the use of two-lane, rather than one-lane ramps, for volumes in excess of 1,500 vehicles per hour, results in superior operations. Martin et al.(22) observed marked deceleration along the mainline and poor distribution across the lanes at high volume, one-lane exit ramps. Auxiliary lanes and two-lane exits are recommended by Martin as a way of avoiding the congestion created at such locations.

Right and Left-Hand Exits

The existence of left-hand exits on the freeway system violates the concepts of operational uniformity and design consistency. Drivers tend to anticipate right-hand exits and position vehicles accordingly. The presence of an occasional left-hand exit conflicts with driver expectancy. This results in excessive lane changing and mixing of decelerating or slower speed traffic with high-speed traffic in the left lane. Also, a left-hand exit may generate wrong-way movements on the cross street because the exit may appear to be a right-hand entrance ramp to an unfamiliar or confused motorist. The adverse effects of left-hand exits on safety have been studied by a number of researchers. Findings from three studies are briefly summarized in Table 7, indicating left-hand exits are at least twice as hazardous as right-hand exits. References cited are Lundy (8), Northwestern University (23), and Taylor et al.(19).

Decision Sight Distance

Adequate sight distance to the exit is critical. Sight distances on the order of 1,500 feet were found by Leisch et al.(24) to be necessary for drivers to recognize the exit, make a navigational decision, and exit safely. This extra distance in excess of the stopping sight distance, is referred to as "decision sight distance."

A number of studies have addressed the operational consequences of limited sight distance. Taylor and McGee (25) studied erratic maneuvers at eight exit ramps. A significant contributor to the high rates of such maneuvers was insufficient sight distance to the gore area. B. Goodwin and Lawrence (26) and D. Goodwin (27) studied lane drops at exit ramps. Lane changes associated with moving out of a lane drop required an average time of 7 seconds. When perception and decision times are added to the maneuver, 12 to 15 seconds of sight distance are required as a minimum. D. Goodwin (27) observed erratic maneuvers increased in frequency at exit locations where lanes were dropped and sight distances were restricted by crest vertical curves.

ENTRANCE RAMPS

The importance of good design of entrance ramps is apparent due to the inherent difficulty of the merging maneuver. Entrance ramp accidents usually involve two or more vehicles. Martin et al.(22) found entrance ramps, particularly those with high volumes, produced more congestion problems on freeways than exit ramps. The consequences of these congestion problems are shock wave effects on the mainline, which create the potential for rear end and sideswipe accidents.

A study of 10 entrance ramps by Transport Canada (4) in Toronto illustrates the basic nature of accidents associated with the merging maneuver. A summary of 156 merging accidents taken from this study is shown in Table 8. A total of 84.6 percent of the accidents involved one or two vehicles. Sideswipe, rear end, and angle accidents predominated. Accident rates at the 10 locations ranged from 1.27 to 3.61 per million ramp vehicles.

Acceleration lanes

Three studies have observed the safety benefits of long acceleration tapers at entrance ramps. Recent Canadian research (4) resulted in the development of a series of regression equations which predict annual merging accidents. The simplest of these equations, is shown below.

$$Ay = K + 0.70\ Vr - 0.80L$$

where
Ay = Merging accidents per year
K = 4.55 if degree of curve = 0^0
K = 5.52 if degree of curve = 3^0
K = 8.29 if degree of curve = 4^0
Vr = Average daily ramp volumes (thousands of vehicles per day)
L = Length of full width acceleration lane; measured from merging end to start of taper (hundreds of feet).
% STD Error 22.6
$R^2 = 0.929$

Lundy (8) found accident rates at ramps with acceleration lane lengths of at least 750 feet to be safer than ramps with shorter lengths. Short lengths in combination with high speeds at the merging end were found to be particularly hazardous. At such locations the driver is left with little time to complete the merge or adjust speed. This reduced margin for error resulted in the high accident rates observed. The overall relations are shown in Figure 4.

Cirillo et al. (2) evaluated the geometry of entrance ramps with extremely high accident rates. High traffic volumes in combination with short acceleration lengths were common to these locations.

A number of studies of the operational quality of entrance ramps have also shown the desirability for acceleration lengths of at least 700 to 800 feet. Wattleworth et al. (28) judged 700 feet to be a minimum length for good operation. Buhr et al. (29) found acceleration lengths above 800 feet resulting in smooth merging of traffic streams with minimal effect on mainline speeds.

TABLE 7 - Comparison of Left and Right-Hand Exit Ramp Accident Experience

Source	Left-Hand Accident Rate	Right-Hand Accident Rate	Ratio LH/RH
Lundy (8)	2.19/MV	0.95/MV*	2.31
North-western Univ. (23)	2.17/MV**	0.92/MV**	2.31
Taylor (19)	2.12/Unit***	1.14/Unit***	1.86

MV - Million Vehicles

* Average for all exit ramps

** Figures are for ramp volumes of 8000 ADT, all freeway volumes; difference in rates is significant at = .05.

*** Accidents per deceleration unit (deceleration lane and taper); no volume data given.

TABLE 8 - Merging Accident Characteristics (156 Accidents at 10 Entrance Ramps)

Impact Type	Percent of Accidents
Side Swipe	16.0
Rear End	53.2
Angle	21.8
Other	9.0
Severity	
Property Damage	63.5
Personal Injury	35.9
Fatal	0.6
No. Vehicles Involved	
One	16.7
Two	67.9
Three	13.5
Four or More	1.9

SOURCE: Reference 4

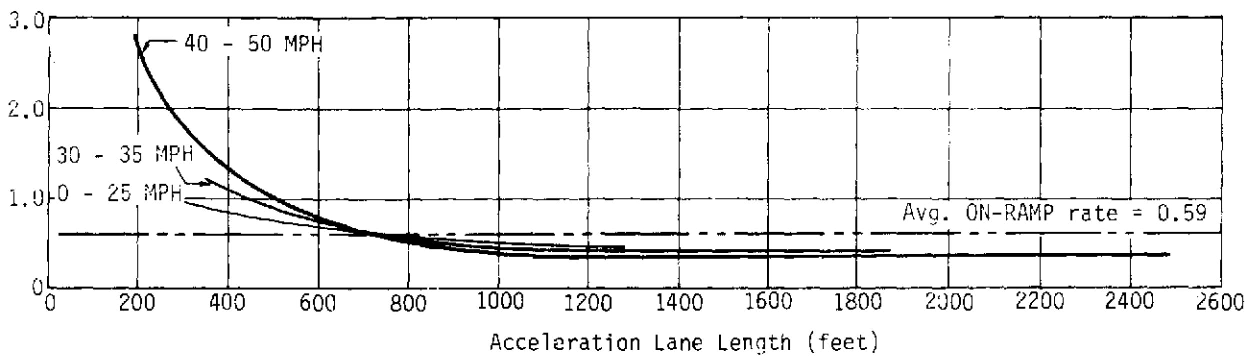


Figure 4. On-ramps: accident rate vs acceleration lane length (233 ramps, 491 accidents, 622.5 MV)

SOURCE: Reference 8

Single and Two-Lane Entrances

Two-lane entrance ramps can have the same relative advantages over single-lane ramps as are provided by 2-lane exits. Martin et al. (22) found improved lane distribution, reduced congestion, and better service to the street system resulting from use of two-lane entrance ramps.

Right- and Left-Hand Entrances

The safety problem created by left-hand entrance ramps is of the same magnitude as left-hand exit ramps. Drivers on the freeway in the left lane do not expect merging into their lane to take place. Entering drivers must merge into the higher speed traffic prevailing on the left of the freeway. This results in greater speed differentials between entering and freeway traffic. This requires entering drivers to wait for longer gaps or accept short gaps for merging. A difficult and unsafe situation is created. This problem is particularly critical for low-speed trucks which enter the freeway on the left.

All of these problems, inherent to left-hand entrances regardless of their geometry, result in accident rates 60 percent higher than those for right-hand entrances. Table 9 illustrates the relative hazard of left-hand entrance ramps versus right-hand entrance ramps as summarized from three studies. Studies cited are Lundy (8), Northwestern University (23), and Taylor et al. (19).

TABLE 9 - Comparison of Left- and Right-Hand Entrance Ramp Accident Experience

Source	Left-Hand Accident Rate	Right-Hand Accident Rate	Ratio LH/RH
Lundy (8)	0.93/MV*	0.59/MV*	1.58
Northwestern Univ. (23)	1.55/MV**	0.97/MV**	1.60
Taylor (19)	9.33/Unit***	0.76/Unit***	12.28

* Average for all (right or left) entrance ramps.

** For all ramp and mainline volumes; difference is significant at .05

*** Limited left-hand sample (3 ramps). Accidents per deceleration unit (deceleration lane and taper); no volume data given.

Angle of Convergence

Several operational/safety studies considered the effects of the angle of convergence of the ramp on operational quality. Wattleworth et al. (28) found smooth flow achieved at merges in which the angle of convergence was not over 3 degrees. Conversely, angles of 10 degrees or more produced poor flow. Buhr's findings (29) support these conclusions. Acceleration noise was greater on ramps with high angles of convergence than on other ramps with similar geometric and volume conditions. Drew et al. (30), in examining driver gap acceptance, found a much higher rate at the lower convergence angles as illustrated by Figure 5. This indicates a smoother flow, less stopping, and less likelihood of rear end collisions.

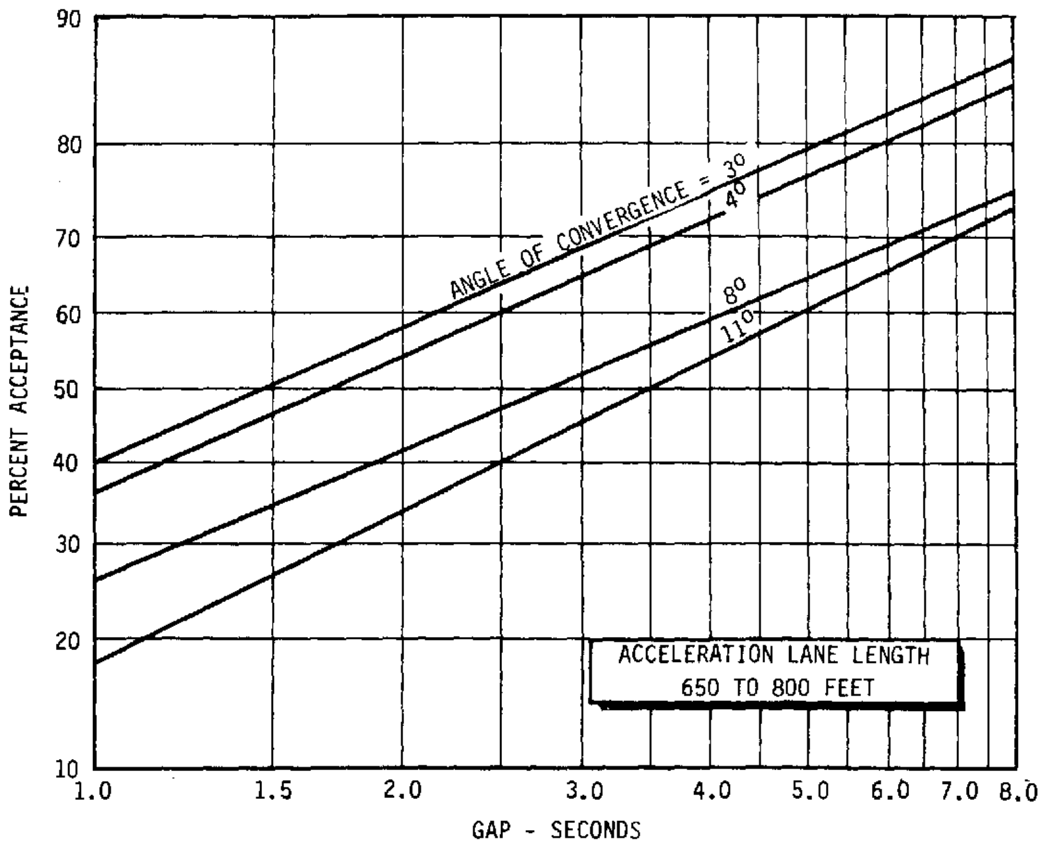


Figure 5. Effect of Convergence Angle on Gap Acceptance Characteristics

SOURCE: Reference 30

Ramp Proper

The geometry of the ramp proper of interchange ramps also affects safety and operations. Horizontal and vertical alinement, cross section, and roadside considerations are all important.

HORIZONTAL ALINEMENT

The horizontal alinement of the ramp proper is particularly important because of the speed changes which take place on the ramp. Overly sharp curvature, particularly at or near the nose or merging end of the ramp, makes transitioning from high to low speed difficult.

The effect of curvature on ramp accident rates is shown in Tables 10 and 11 from a study by Yates (31) on outer connection and loop ramps of urban and rural interchanges.

Gray and Kauk (32) studied the operational characteristics of two loop ramps. Ramp A contained a minimum radius of curvature of 200 feet. Ramp B was a "humpback" design using a tangent between two curves, of minimum 125 foot radius, in the same direction. The study of speeds, lateral placement, and acceleration produced the following findings:

o Ramp A was judged to be more conducive to ease of operation than Ramp B, based on analyses of speed changes along the ramp and vehicle lateral placement.

o Ramp A produced a rate of change of lateral acceleration half that of Ramp B indicating that Ramp A was more comfortable to drive.

VERTICAL ALINEMENT

The grade of the ramp as it leaves the nose, just prior to the end of the merging area, has an important effect on ramp operations. Operational studies have verified the desirability of designing interchanges with the crossroad over (above) the freeway for the following grade related reasons:

(1) With the crossroad over the freeway, the exit ramp and gore area will normally be visible for sufficient distance. The freeway over the crossroad frequently results in the exit gore being hidden beyond the mainline upgrade or crest.

(2) By placing the crossroad over the freeway, the designer uses gravity and sight distance to assist the operation of both accelerating vehicles (on a downgrade) at entrance ramps and decelerating vehicles (on an upgrade) at exit ramps.

Wattleworth et al. (28) observed the effect of grade on merging. As the effective grade of

TABLE 10 - Accident Rates on Outer Connections by Curvature and ADT

Accidents Per 100 Million Vehicles

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

[a]Less than 10 study units

SOURCE: Reference 31

TABLE 11 - Accident Rates on Loops by Curvature and ADT

Accidents Per 100 Million Vehicles

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

[a]Less than 10 study units

SOURCE: Reference 31

the entrance ramp relative to the mainline increased, the distance to the merging point along the freeway increased. At locations in which the ramp was lower than the freeway (relative upgrade on the ramp), ramp drivers could not select gaps and adjust speed until they were near the end of the merging area. This resulted in the need to make more speed adjustments on the acceleration lane.

Excessive grades can have adverse effects on ramps in general. Cirillo's (2) analysis of high-accident ramps attributes some causative effect to excessive grades on accidents at slip ramps, outer connections, and direct or semi-direct connections. At loop ramps with excessive grades, the profile is judged to have considerable causative effect on the accident experience.

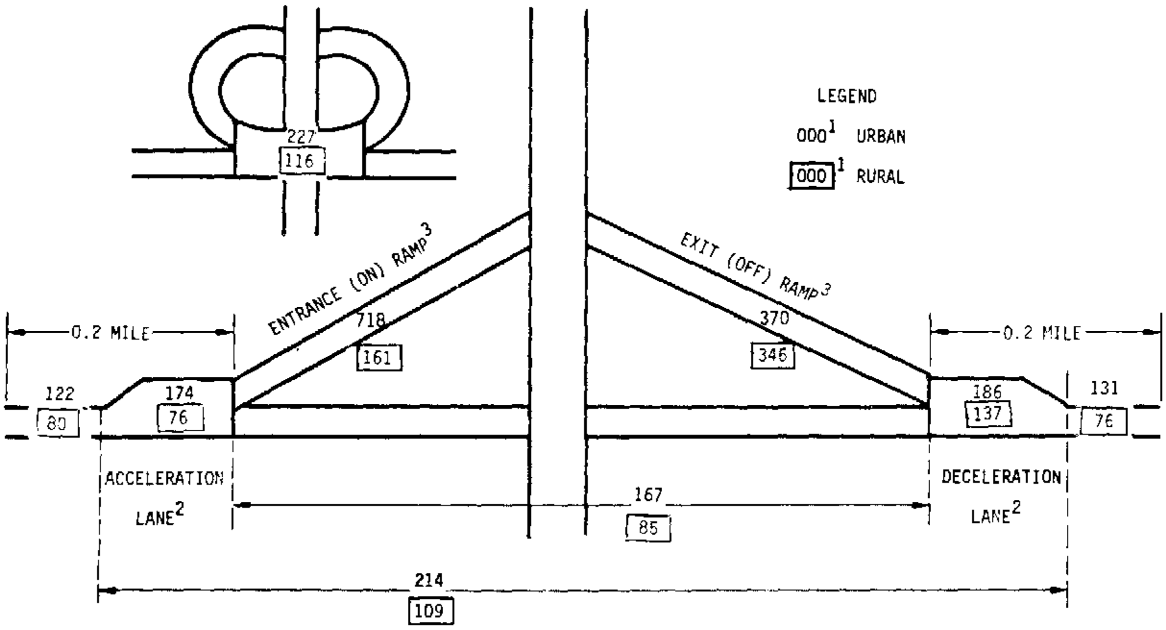
CROSS SECTION AND ROADSIDE

The cross section of ramps has been studied by a number of researchers. Wattleworth et al. (28), being interested in the relative effect of pavement width on ramp merging, observed a narrow 9-foot wide ramp contributing to difficulty in merging compared to a 12-foot wide ramp. Cirillo et al. (2) and Transport Canada (4) cite an association between shoulder width and accidents. Wider shoulders may produce somewhat safer ramps.

The safety effect of a clear roadside is apparent from several studies. Lundy (8) observed greater than half the accidents at exit ramps were single-vehicle or off-the-road accidents. In Harwood's (15) study of rehabilitated interchanges, single-vehicle accidents were a safety problem at 5 of the 40 interchanges. Significant accident reductions were achieved by lengthening ramp tapers, ramp widening, and removal of fixed objects.

SUMMARY BASIC ELEMENTS

Figure 6, taken from a study by Cirillo (1), summarizes accident rates within each of the basic elements of the interchange discussed in the previous sections. The higher overall rates for urban areas reflect the relationship of volume to accident rates previously discussed. Higher volumes generate increased vehicle-to-vehicle conflicts and higher accident rates. Exit ramps in rural areas exhibit an accident rate nearly comparable with those in urban areas, an indication of the single vehicle run-off-the-road nature of exit ramp accidents. For entrance ramps, where conflicts are the predominant factor, urban entrance ramps can be seen to have a much higher accident rate than rural entrance ramps where vehicular interactions would be much less likely.



[1] Accident rate per 100 million vehicles of travel.

[2] Rate includes accidents along adjacent mainline.

[3] Rate does not include accidents at intersection with crossroad.

Figure 6. Accident Rate By Type of Interchange Element

SOURCE: Reference 1

Selected Special Elements

The remaining discussion in this section deals with selective elements of interchanges. These elements may or may not be present at all interchanges. They include auxiliary lanes, lane drops, weaving sections and collector-distributor roads. Their presence in combination with the basic interchange elements (exit and entrance terminals and the ramp proper) can have significant safety impacts.

AUXILIARY LANES AND LANE DROPS

Auxiliary lanes and lane drops are special features of freeway systems. Auxiliary lanes are used between closely spaced interchanges to increase capacity and reduce lane changing and weaving. Martin et al. (22) observed the effectiveness of auxiliary lanes in his study of California freeways. He concluded auxiliary lanes should extend to the next exit ramp or for a minimum of 2,500 feet.

TABLE 12 - Erratic Maneuver Rates at Exit Ramps With and Without Lane Drops

Site	Erratic Maneuver (EM) Rates*: Exiting EM as a Percent of Exiting Volume	Through EM as a Percent of Through Volume
	Locations With Lane Drops	
I-283N to I-83, US 322 Right Ramp	1.39	2.60
I-79S at Exit 18	0.00	0.79
I-76W at Exit 13	0.89	0.16
I-95S at I-695	0.95	0.74
	Locations without Lane Drops	
I-83S at Exits 27-28	2.18	0.55
US 322W to I-83, I-283 Left Ramp	3.42	0.02
Right Ramp	1.27	0.29
I-283N to I-83, US 322 Left Ramp	9.18	2.02
I-76E at Exit 14	0.40	0.08
Baltimore-Washington Expressway to Harbor Tunnel Thruway	0.87	0.08
Harbor Tunnel Thruway to Baltimore-Washington Expressway, Exit 15	1.11	0.36

*EM Rates -- Percent of vehicles performing one of the following maneuvers: Cross Gore Point or Area, Stop in Gore, Back Up, Slow Suddenly, Lane Change (to exit), Swerve, and Stop in Shoulder.

SOURCE: Reference 25

Lane drops occur at interchanges as well as along the mainline. Because lane drops are unusual and unexpected features, proper warning and design of the lane drop is critical. D. Goodwin's study of lane drops (27) points out a common characteristic of hazardous lane drops as poor or insufficient sight distance of less than 1,500 feet. D Goodwin also recognized dropping a lane at an exit without an optional lane to be a violation of lane balance. Many drivers who normally travel in the right lane do not expect to have to exit and are thus forced to change lanes. This occurs regardless of the sight distance. Taylor and McGee (25) also noticed this problem in a study of erratic maneuvers at gore areas. Table 12 shows erratic maneuver rates for the through drivers at some exits, in which a lane is dropped, exceeded those for the exiting drivers. In other locations studied, the reverse was true. The presence of the lane drop was an unexpected occurrence which affected the entire stream of traffic. When these unusual situations are imposed on the unfamiliar driver and/or when visibility is poor because of darkness or poor weather, the potential for an accident is further increased. Taylor found a substantial portion of interchange accidents were preceded by an erratic maneuver. Therefore, it is believed a reduction in the frequency of erratic maneuvers is likely to result in a reducion in the number of accidents. This is indicated in Table 13.

TABLE 13 - Summary of Accident Data as Related to Erratic Maneuvers (EM)

Site	Accidents (2 Year Period): Total	EM Prior to Accident	EM % of Total
	Locations With Lane Drops		
I-79S at Exit 18 for Greentree and Crafton	47	9	19%
I-76W at Exit 13 for Churchill	26	5	19%
I-95S at I-695	10	7	70%
	Locations Without Lane Drops		
I-76E at Exit for Business US 22	20	4	20%
Baltimore-Washington Expressway to Harbor Tunnel Thruway	11	3	27%
Harbor Tunnel Thruway to Baltimore-Washington Expressway, Exit 15	12	8	67%

Erratic Manuever (EM) - Vehicle performing one of the following maneuvers: Cross Gore Point or Area, Stop in Gore, Back Up, Slow Suddenly, Lane Change (to exit), Swerve, and Stop in Shoulder.

SOURCE: Reference 25

RAMP SEQUENCES

The arrangement of ramps, both within and between interchanges, can have a significant effect on the safety and operational quality of the freeway. An important aspect of ramp arrangement is the sequence of exit and entrance ramps. An entrance ramp followed by an exit ramp may create a weaving section. Where volumes are high and the distances between the ramps are short, weaving sections create safety problems to be discussed in the following sections. The relationship of accident rate to volumes and lengths of weaving sections at cloverleaf interchanges is shown in Figure 2 by Foody (3). Cirillo's studies (2) also showed, as volumes increase and weaving section lengths decrease, accident rates increase dramatically.

Sequencing of successive entrances or exits can also affect operations. Exit ramps in series with close spacing can result in poor volume distribution across the lanes, with more vehicles in the right lane preparing to exit. Martin's studies (22) demonstrated successive exit ramps should be spaced 1,200 to 1,500 feet apart to prevent these problems. Similar operational problems result where successive entrance ramps cause excessive congestion and poor distribution of lane volumes. Martin's studies showed 1,500 to 1,700 feet of separation between successive entrance ramps necessary to achieve good operation.

WEAVING SECTIONS

Weaving sections present the greatest source of operational and safety problems on urban freeways. Weaving sections can be created by exit and entrance ramps in series within an interchange or between two interchanges. Such sections with heavy traffic volumes create congestion and back-ups on ramps and mainline due to the high incidence of lane changing with forced flow conditions. This situation in turn results in rear end, merge, and angle accidents.

A recent study by Foody (3) of 32 cloverleaf interchanges along freeways provides some insight into the hazards presented by weaving sections. Of the 1,815 observed accidents at these interchanges, 28 percent occurred within the weaving section or on the loop ramps which make up the weave. Also accident patterns changed as traffic volumes increased. Table 14 shows the percentage of accidents in the weave section to be twice the percentage found at low volume interchanges. Two important conclusions with respect to cloverleaf interchanges illustrate the problems associated with weaving sections in general:

- Of all mainline elements which comprise the cloverleaf, the weave section experiences the greatest increase in accident frequency with increasing ADT.
- Safety problems with cloverleafs result from designs which are inadequate for high-speed, high volume operation. The major element of these designs which affects operations is the weaving section.

Weaving sections are particularly hazardous when insufficient length is provided for the weaving maneuvers. Appropriate lengths of weaving sections are a function of the number of lanes on the freeway, total traffic volume, and each ramp volume. Cirillo et al. (2) evaluated the geometry of weaving sections with unusually high accident rates. A combination of high traffic volume and short weaving length was common to these locations. Tait et al. (11) studied six cloverleaf interchanges identified as being problem locations. At three locations, extremely short weaving sections from 420 to 650 feet had the effect of backing up traffic on the ramps and mainline. This created severe speed reductions and forced-flow conflicts.

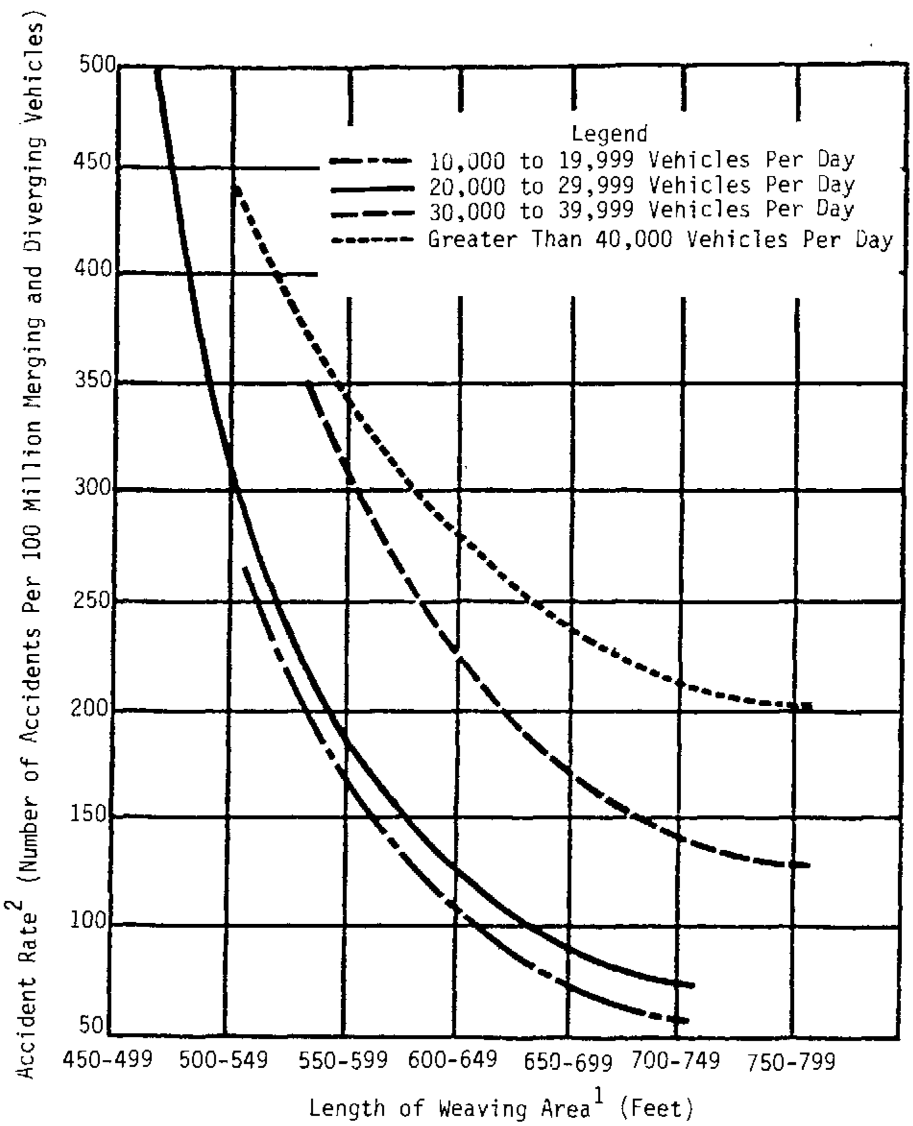
TABLE 14 - Distribution of Accidents by Location Within Cloverleaf Interchanges

		High Volume Interchange ADT >16,000	Low Volume Interchange ADT<16,000
Mainline*	Deceleration	13%	9%
	Weaving	23%	11%
	Acceleration	14%	10%
	Between Ramps	17%	12%
	Sub-Total	67%	42%
Ramps	Outer Conn. Off	7%	8%
	Loop Ramp On	5%	4%
	Loop Ramp Off	5%	8%
	Outer Conn. On	4%	7%
	Sub-Total	21%	27%
Crossroad		12%	31%
	Total	100%	100%

*Exiting and entering traffic included for mainline

SOURCE: Reference 3

Figure 7, from a study by Cirillo (21), shows the significance of high volume and weaving length in accident occurrence at weaving sections of cloverleaf interchanges.



1 Length Measured from Merging End of Entrance Loop Ramp to Nose of Exit Loop Ramp

2 Includes Accidents Along Mainline

Figure 7. Accident Rate and Length of Weaving Area

SOURCE: Reference 21

COLLECTOR-DISTRIBUTOR ROADS

Collector-distributor (C-D) roads are employed through and between interchanges to facilitate weaving maneuvers. The operational and safety effects of C-D roads derive from the separation of the lower speed weaving maneuvers from the higher speed through traffic. Hansell (12) studied erratic maneuvers and accidents at eight interchanges with similar traffic and geometric characteristics. Five of the eight were served with C-D roads, the remainder were not. The superior operational quality provided by C-D's was shown by lower erratic maneuver rates on those interchanges with C-D roads. More importantly, an evaluation of 3 years of accident data revealed significantly lower weave-related accident rates on the C-D equipped roads.

A recent evaluation of the Interstate Accident Data Base by Morganstein and Edmonds (33) showed somewhat higher accident and injury rates being expected at interchanges without C-D roads. This data is summarized in Table 15.

TABLE 15 - Accident, Injury and Fatality Rates at Interchanges With and Without Collector-Distributor-(C-D) Roadways

	Accident Rate*	Injury Rate*	Fatality Rate*
Interchange With C-D Roadways	8.16	2.03	0.17
Interchange Without C-D Roadways	9.48	3.59	0.16
TOTAL	9.37	3.47	0.16

*Per Million Vehicles Through the Interchange

SOURCE: Reference 33

CROSSROAD AND ASSOCIATED RAMP TERMINALS

One of the most important factors in the safe operation of an interchange is the handling of the crossroad, including intersections with the ramps. In rural areas, accident problems relate to the crossing conflicts at unsignalized ramp terminal intersections, and the potential for wrong-way movements from undivided crossroads. Crossing conflicts are also a problem in urban areas, particularly when sight distance is restricted, ramp volumes are high, and/or the crossroad is a four-lane facility.

A good indication of the seriousness of the crossroad accident problem at interchanges is given by the Harwood study (15) of 40 interchange rehabilitation projects. In 34 of the projects, a portion of the rehabilitation was devoted to the ramp terminal intersections. Among the problems cited as being common to many crossroads and ramp terminals are:

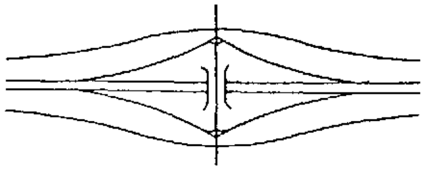
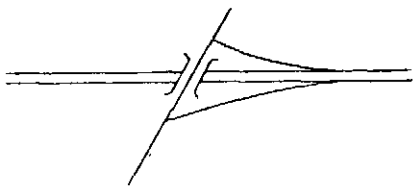
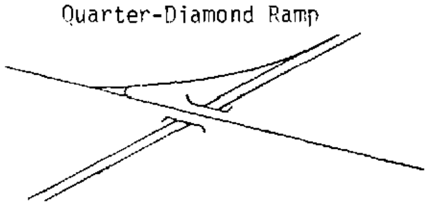
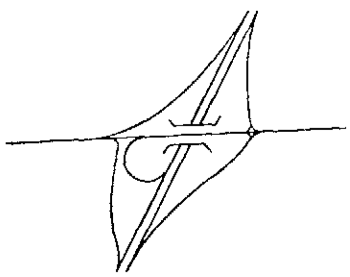
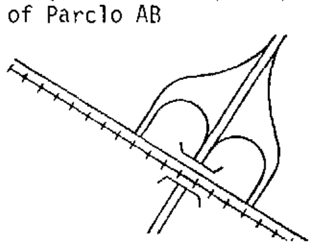
- Rear end, left turn, right turn, and angle accidents at crossroad ramp terminals.
- Excessive delay for off-ramp traffic.
- Congestion on arterials between free-flow ramp terminals and adjacent intersections.
- Rear end and angle accidents in weaving areas between free-flow ramp terminals and adjacent intersections.

Cirillo's (2) analysis of interchange accidents suggests the design of the crossroad and the extent of activity along it are correlated with accidents. The model for full diamond interchanges (Figure 1) utilizes independent variables describing the number of businesses along the crossroad, presence of a median on the crossroad, and number of lanes on the crossroad.

WRONG WAY MOVEMENTS

The problem of wrong-way movements originating at ramp terminals is discussed by a number of authors. The use of certain interchange forms in particular situations can lead to confusion and a higher incidence of wrong-way movements. This is a particularly significant factor in rural areas at night. Parsonson and Marks (13) observed high rates of wrong-way movements at partial cloverleaf interchanges where the exit ramp terminal is adjacent to the loop entrance ramp terminal. Scifres (14) examined the characteristics of crossroad terminal sites which seemed to generate wrong-way movements. Findings show a lack of channelization was a frequent characteristic. Approximately 3/4 of all wrong-way movements studied were found to have occurred at night, during periods of low volume with little activity along the crossroad to provide artificial light or clues about the location of the ramp.

Vaswani (18) and Parsonson and Marks (13) identify crossroad countermeasures which alleviate wrong-way movements. Figure 8 summarizes Parsonson's findings based on a before and after analysis of frequencies of wrong-way movements.

INTERCHANGE FORM	COUNTERMEASURES APPLIED	WRONG-WAY RATES (MONTHLY) BEFORE	AFTER
Diamond Ramp; Close Frontage Road 	• Large Pavement Arrows • 24" Stop Bar • DO NOT ENTER Sign (R5-1) • Guide Sign	2.9	1.4
Half-Diamond Ramp 	• Standard MUTCD Arrows • WRONG WAY Sign (R 5-1a) • DO NOT ENTER Sign (R 5-1) • NO RIGHT TURN and NO LEFT TURN Signs (R 3-1 and R 3-2)	4.6	2.4
Quarter-Diamond Ramp 	• Large Pavement Arrows • KEEP RIGHT Sign (R4-7)	3.5	2.0
Diagonal and Loop Ramps of Partial Cloverleaf 	• Large Pavement Arrow • 24" Stop Bar • DO NOT ENTER Sign (R5-1)	8.1	0
Diagonal and Loop Ramps of Parclo AB 	• 24" Stop Bar • Large Pavement Arrow • Trailblazer • Ceramic Buttons	3.2	1.8

*Numbers in parentheses refer to MUTCD identification numbers.

Figure 8. Effectiveness of Countermeasures For Wrong-Way Movements

SOURCE: Adapted from Reference 13

OPERATIONAL SAFETY

Signing, delineation marking schemes, and control measures such as ramp metering all affect the operations and safety of interchanges and freeway systems. In turn, the design of the interchange affects the selection and safety effectiveness of a signing/delineation scheme or control measure.

In this section operational and safety effectiveness of freeway information and control devices is discussed.

SIGNING

The complexity of factors which contributes to safe operations at interchanges serves to confound attempts to attribute safety effectiveness (in terms, say, of accident reductions) to signing alone. Many authors have pointed out the relationship between the form of an interchange and adjacent roadway systems to directional signing requirements. Designs which are operationally unsound, with right and left-hand exits, close interchange spacing, and no lane balance, are very difficult to sign effectively without overloading the driver with information. It is therefore extremely difficult to separate the effect of such signing overloads from poor design. In addition, it should be recognized signing is primarily for unfamiliar drivers, who may be a small percentage of the facility traffic. Thus, safety benefits of signing schemes become even more difficult to identify. For these and other reasons, researchers have studied the effectiveness of interchange signing in terms of operational measures, such as lane changes, erratic maneuvers, and other conflicts. In addition, studies of human factors requirements have been related to signing practice to develop measures of signing quality.

One indication of the effect of signing on freeway operations is provided by the Taylor and McGee (25) study of exit behavior. Erratic maneuvers at 10 locations were observed. Drivers who made such maneuvers were interviewed directly after the maneuver. More than half of the exiting drivers who made erratic maneuvers indicated the signs were not clear or did not meet their expectations. The results of the interviews were used to classify causes of erratic maneuvers and determine their relative frequency. Table 16 shows 53 percent of the erratic maneuvers attributed to an information deficiency.

TABLE 16 - Frequency of Factors Cited by Drivers As Causing Erratic Maneuvers

Data from Nine States

Factor	Total Frequency	Percent of Total
Driver-Related Problem:		
Distracted or Inattentive	16	8
Last-Minute Change of Mind	17	9
Not Sure of Direction	38	20
		37
Information Deficiency:		
Sign Legend	48	27
Insufficient Advance Warning	38	20
Inadequate Sign Visibility	9	5
Inadequate Markings, Delineators	1	1
		53
Geometric Deficiencies		
Visibility of Ramp Area	4	4
Other Inadequate Geometrics	6	6
		10

SOURCE: Reference 25

DIAGRAMMATIC SIGNING

In response to problems at unusual locations, diagrammatic signs have been tried and studied for their effectiveness. Roberts et al.(34) studied erratic maneuvers at freeway exits during peak periods. The study findings revealed the following:

- Conventional signs were more effective in reducing critical maneuvers at splits for parallel roadways.
- Diagrammatic signs reduced stopping and backing maneuver rates at right-hand ramps, but had no effect on unusual exit gore maneuver rates.

Lunenfeld and Alexander (35) determined left-hand exits at which a lane is dropped, due to their unusual nature, should be signed with advance and overhead mounted exit signs of a diagrammatic type. Mast and Kolsrud (36) confirmed the effectiveness of such signing at left-hand exits and prescribed locations where diagrammatic signs would be most effective. Hanscom (37) studied the effectiveness of diagrammatic signs at certain complex, multiple-exit interchanges. Reductions in weaving rates over the gore area of 22 percent were observed as well as a significant decrease (16.6 percent) in stopping and backing maneuvers.

Taylor and McGee (25) demonstrated the need for special signing at exit locations with a combination of unusual characteristics. Operations at lane drops and exits with restricted sight distance were significantly improved through installation of "EXIT ONLY" signs, standardization of signs, and additional advance signing.

Pavement Marking and Delineation

A limited number of studies have been performed to evaluate the effectiveness of certain marking schemes at interchanges. Taylor and McGee (25) noted the effectiveness of gore markings and lane lines in reducing the erratic maneuvers across exit gore areas. In a separate research effort, Taylor and McGee (38) studied the effect of pavement coloration schemes at lane drops and exits on encroachments and lateral placement of the vehicles. A significant reduction in encroachments on the restricted area was achieved at 7 of 10 sites through the use of yellow colored pavement. At five of eight sites, the yellow colored pavement produced significant shifts in the lateral placement of the right front wheel. Studies of speeds, lane changes, and brake light applications at exit ramps with colored pavements failed to produce changes in observed vehicle behavior.

In a study of auxiliary lanes and lane drops at exits, Martin et al. (22) concluded auxiliary lanes should have special delineation to differentiate them from the through lanes of the freeway. Contrast treatment was not necessarily the answer, but use of special striping or dots may prove to be. Further suggested treatments are presented by Leisch (39) and Taylor and McGee (25).

A number of authors have studied wrong-way movements at ramp terminal intersections and have suggested delineation schemes, including wide stop bars, arrows on the pavement, and signing. Gabriel (16) and Parsonson and Marks (13) suggested such countermeasures following independent analyses of wrong-way movements at interchanges.

RAMP CONTROL

Metering of vehicles entering freeways has been used in many locations to improve traffic flow. Metering has been instituted at individual locations, where poor ramp geometry and/or high volumes have resulted in congestion and safety problems for merging vehicles. System-wide use of metering is also common. In such situations, metering on a series of ramps regulates the flow of entering traffic thereby keeping freeway flow at or below available capacities, and reducing total delay. On the individual ramps, metering provides a separation between ramp vehicles, which reduces rear end type collisions.

Information from the Everall study (40) in Table 17 summarizes the effectiveness, in terms of travel time and accident reduction, of a series of ramp metering projects in the late 1960's.

The accident reductions shown in Table 17 are indicative of potential "order of magnitude" benefits only. Other studies have verified the safety effectiveness of properly designed metering systems. A study of the Dallas North Central Expressway Corridor by Cima et al. (41) showed annual accident reductions during peak periods of 20 to 30 percent.

Cima (42) studied merge related conflicts and accidents during peak periods of demand at a single ramp. A before and after analysis made use of 4 years of accident data for each period. Ramp metering produced a 35 percent reduction, 32 accidents before to 21 after, in merge related accidents significant at the .05 level. Conflicts at the merge point were reduced 11.6 percent following introduction of metering. McCasland's (43) study of accidents at two entrance ramps is consistent with the Cima findings. While the after period in McCasland's study was only one year, ramp metering was indicated to have had a positive impact on safety at the study locations.

TABLE 17 - Benefits Achieved in Ramp Metering Projects

Location	Number of Ramps	Change In Vehicle Hours of Travel Time (Annual)*	Change In No. of Accidents (Annual)
Atlanta	1	+8,200	-70
Minnesota	2	-5,640	N/A
Los Angeles (Harbor Freeway)	6	-108,300	N/A
Detroit (John Lodge Freeway)	8	-225,000	N/A
Chicago (Eisenhower Expressway)	8	-64,000	-51
Houston (Full Ramp Control)	8	-72,670	-40

* Net Increase (+) or decrease (-) in total travel time along the freeway and ramps attributed to implementation of metering.

N/A - Not Available

SOURCE: Reference 40

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CHAPTER 7 - ONE-WAY STREETS AND REVERSIBLE LANES

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INTRODUCTION

For over 50 years one-way streets have been used, to an increasing degree, for the primary purpose of increasing capacity and operating speeds, and reducing delay on specific pairs of streets and street networks. Where one-way streets have been used, improved safety and a decrease in accidents have also resulted in many of the one-way systems. Reversing traffic on all or some traffic lanes at specific times has also increased capacity and speed, reduced delay, and improved safety. Implementation of reversible lanes requires more attention to traffic control features than does one-way street operation. This chapter cites representative studies of one-way street and reversible lane research with emphasis on the safety aspects.

ONE-WAY STREETS

GENERAL CHARACTERISTICS

One-way streets have a number of characteristics which should enhance safety.

1. Fewer points of potential conflict exist at intersections.
2. The chances of head-on and sideswipe accidents may be greatly reduced due to no opposing traffic.
3. Turning vehicles can be passed reducing the possibility of rear end collisions.
4. Signals can be timed for progressive movement. This reduces the number of stops and keeps vehicles in orderly groups with well defined intervals between groups for pedestrian and vehicle crossings.

The following material dealing with one-way streets has been drawn, in large part, from Chapter 10, "One-Way Streets and Parking" (1), of "Traffic Control and Roadway Elements - Their Relationship to Highway Safety/Revised." The one-way street portion of that chapter was originally prepared by Peter A. Mayer, then a Traffic Research Engineer with the Highway Users Federation for Safety and Mobility.

In 1929, Eno (2) described the early application of one-way streets in New York (1907), Boston (1908), Paris (1909), and Buenos Aires (1910). He proposed extensive implementation, including a one-way system in New York City for east-west streets between 14th and 59th Streets.

The purpose of one-way streets, Eno advised, was to avoid confusion and to better utilize narrow one and two-lane streets. This early vision recognized the potential of one-way streets. Every major study completed since has shown one-way streets improving transportation efficiency. Several early studies also attributed large accident reductions to one-way streets. In addition to the safety improvement, most studies found substantial improvements in travel time, street capacity, and vehicle delay. However, there is inadequate nationwide data on how many one-way streets are in use, the magnitude of improvements in operation, and the accident reduction being achieved.

One-way streets generally reduce accidents but not for all situations. Accident reductions of from 10 to 50 percent have been reported. Examples do exist where certain types of accidents and accident rates have increased. Research has identified one-way streets as an important technique for improved traffic operations and safety on urban streets although the possibility for increased accidents does exist.

Consistent characteristics of accidents on one-way streets have been reported.

- In most cases, rear end, sideswipe meeting, turning, parking, and pedestrian accidents can be expected to decrease.
- Accidents that involve turns from the center lane may increase.
- Accident severity generally decreases.
- There is almost always a reduction in total accidents after the first year of operation.
- Mid-block accidents generally are reduced more than intersection accidents.

Because problems do arise with the initial operation of one-way streets, special attention should be given to advance publicity, proper and adequate application of traffic control devices, and enforcement.

The following sections describe the important findings of studies evaluating the safety relationships of one-way streets in terms of accident characteristics and traffic control.

Accident Experience From 1930 Through 1972

Early progress in implementing one-way streets was reported in 1937 and 1938 by Canning (3, 4) from studies presenting the reasons for and the effects of designating existing two-way streets for the use of traffic in one direction. One-way streets were found to increase capacity and speed as well as reduce accidents. Progressive signalization was also facilitated. Examples of data are presented from a number of cities to confirm these findings. In Detroit, Mich., 0.625 miles of one street were changed from two-way to one-way in 1930. Table 1 summarizes the results of the change.

Canning reported that when three streets in Washington, D.C. were made one-way in 1935 for peak hour operation, accidents increased 90 percent. However, in Philadelphia, Pa., he reported a material reduction in "at intersection" and "between intersection" accidents for similar streets operating one-way as compared to two-way.

TABLE 1 - Effect of Change From Two-Way to One-Way (Detroit, Mich.)

	Two-Way	One-Way	Percent Change
Free Moving Lanes (Number)	2	3	+50%
Traffic Volume, (max. per hour)	680	980	+44%
Average Speed, (mph)	14	22	+57%
Number of Accidents (per year)	54	38	-30%

SOURCE: Reference 4

In Table 2, data from Chicago, Ill., shows the improved operation of three locations. The total number of accidents on all three streets decreased from 993 to 553 or nearly 45 percent. These streets may have aspects of reverse lane operation, but are being cited here since they are referenced to Canning's review of early one-way street operations.

TABLE 2 - Effects on Directional Flow by Reducing Number of Opposing Lanes (Chicago, Ill.)

Location	Number of One-Way Lanes		Max. Hourly Traffic Volume		Average Speed (mph)	
	Before	After	Before	After	Before	After
Sheridan Road	2	3	1700	2630	9	30
Warren Boulevard	2	4	1300	2900	14	28
Lake Shore Drive	4	6	3375	7120	10	35

SOURCE: Adapted from Reference 4

Faustman (5) reported a conversion to one-way streets in Sacramento, Calif., showing a 14-percent accident reduction the year following conversion. All accidents in the city increased 16.6 percent.

Sometimes accident rates increase immediately following such conversions and then decrease as time passes. Not all situations show accident reductions, although in general there is significant improvement. The differences may be due to a number of variables including signing, parking changes, and the addition of traffic signals or other modification in traffic control.

In 1957, Modesto, Calif., found an operational problem after initiating one-way operation on 5 miles of eight arterial streets in the central part of the city. In the first year of one-way operation, Carmody (6) reported the number of accidents decreased from 356 to 322. The number of injury accidents increased from 48 to 77. In the second year (7), injury accidents were reduced from 77 to 49. The total number of accidents were down 28 percent from the last year of two-way operation with 24 percent more traffic volume. The improvements in the second year of the after period were attributed to informing motorists of operational problems through newspapers, the driver licensing agency, and police at the site. A reduction in pedestrian accidents was also reported. Modesto also experienced accidents caused by turns from the center lane. During the first year of one-way operation, there were 62 accidents of this type or 20 percent of all the accidents. After the program of publicity and education during the second year of one-way operation, there was a 50 percent reduction in wrong lane turn accidents.

An early study of the safety effectiveness of one-way streets was conducted in Oregon. Peterson (8) reported 12 Oregon cities operating one-way street pairs on the congested sections of State highway through central business districts during the 1940's and 1950's. The accident and severity rates, as well as other characteristics of these one-way streets, are shown in Table 3. Eight cities reported statistically significant accident reduction rates after the one-way systems were installed. The average accident severity rate (casualties per 100 million vehicle miles) was reduced significantly in five cities, but did not change significantly in seven cities. In an attempt to discover a reason for the variation in results, the relationships between the change in accident rate, traffic volume, and length of one-way streets were reviewed. No relation was detected between safety and these factors.

TABLE 3 - Characteristics of One-Way Streets and Accidents - Oregon

	Before & After Period (Years)	Average Daily Traffic		Study Length (Miles)	Accident Rate(a)		Percent Change	Severity Rate(b)		Percent Change
		Before	After		Before	After		Before	After	
1. Astoria	3	8,700	9,370	.46	61.9	53.2	-14(d)	634	771	+22(c)
2. Coos Bay	3	9,980	15,960	.78	49.9	21.8	-56(e)	476	220	-54(e)
3. Corvallis	3	8,040	9,325	1.21	48.6	31.2	-36(e)	495	497	+ 1(c)
4. Eugene	1	8,200	6,040	1.86	73.3	37.3	-49(e)	790	463	-41(c)
5. Lebanon	3	6,440	8,630	.66	47.8	39.1	-18(e)	444	338	-24(c)
6. Medford	1	11,680	11,090	2.24	16.8	9.3	-45(e)	226	99	-56(d)
7. Pendleton	3	6,430	7,560	1.23	44.4	48.2	+ 8(d)	450	433	- 4(c)
8 Redmond	3	4,120	7,240	1.16	30.4	17.9	-42(e)	294	240	-19(c)
9. Salem	1	19,600	20,500	3.18	44.1	42.1	- 4(d)	570	418	-27(d)
10. Springfield	3	14,500	16,800	1.47	26.6	16.0	-40(e)	407	266	-35(e)
11. The Dalles	3	8,780	17,300	.74	52.1	34.7	-33(e)	479	233	-51(e)
12. Tillamook	3	5,840	6,880	.79	41.4	38.8	- 6(d)	297	572	+92(c)
Average					41.3	30.2	-27	464	343	-26

(a) Accidents per million vehicle miles
(b) Casualties per 100 million vehicle miles
(c) Change not statistically significant
(d) Change statistically significant
(e) Change highly significant

SOURCE: Reference 8

In the before and after study of the 12 Oregon one-way pairs, intersection accidents were studied separately to determine if the reduction of the possible points of conflict on one-way streets is effective in reducing intersections accidents. As shown in Table 4, summarizing six of the cities, the accident rate at intersections was reduced from 18.36 to 13.52 accidents per million vehicle miles (acc/MVM) or 26.4 percent. However, the reduction in non-intersection accidents was even larger, from 20.27 to 11.65 acc/MVM or 42.5 percent. Rear end, turning, and pedestrian accident rates at and between intersections decreased, as did those for sideswipe, parking, and backing between intersections. Rates for other types of accidents remained substantially unchanged.

The Oregon study concluded a reduction in accidents can be expected between and at intersections after establishing one-way streets. The greater percentage reduction occurs between intersections. This Oregon finding that the non-intersectin accident rate showed a greater percentage reduction than did the intersectional accident rate is deserving of considerable emphasis. In addition, one-way streets also yield less congestion, greater capacity, freer movement, and reduced travel time. In the 12 cases cited, the study included, besides one-way routings, construction of various new connections, some channelization, improvements at selected intersections to allow easy turning and, in some instances, additional traffic signals to keep platoons defined.

In 1950, a one-way street system was established in Portland, Oreg. As reported by Fowler (9), it was located in the central west side business area and included over 21 miles of streets covering 280 city blocks. The excellent results from the earlier one-way pair were largely responsible for the quick public acceptance of the complete grid system. In every case the one-way street carried traffic volumes in excess of the previous two-way volumes with appreciably greater freedom of movement. Volumes doubled on some streets. Traffic signals provided a 15 mph progression in all directions. This permitted the average speed to increase from 7.9 mph to 14.2 mph. The peak hour speed increased from 5.8 mph to 11.6 mph. Most types of accidents were reduced. Angle collisions were reduced two-thirds and turning collisions reduced one-half. Although the total number of accidents was greatly reduced, the number of injuries dropped from 241 to only 213 due to the higher speeds. The comparison of accidents in 1949 with those in 1951 is shown in Table 5.

The curves in Figure 1, from 1964 accident data, show the lower accident rates for signalized intersections in the central business district (CBD) compared to the higher rates for signalized intersections outside of the core area.

TABLE 4 - Intersection and Non-Intersection Accidents (Six Oregon Cities)

	Time Period (Years)	Number of Accidents	Accident Rate(a)	Percent Reduction in Rate
INTERSECTION				
Before One-Way	3	969	18.36	
After One-Way	3	1024	13.52	26.4
NON-INTERSECTION				
Before One-Way	3	1069	20.27	
After One-Way	3	883	11.65	42.5

(a) Accidents per million vehicle miles

SOURCE: Reference 8

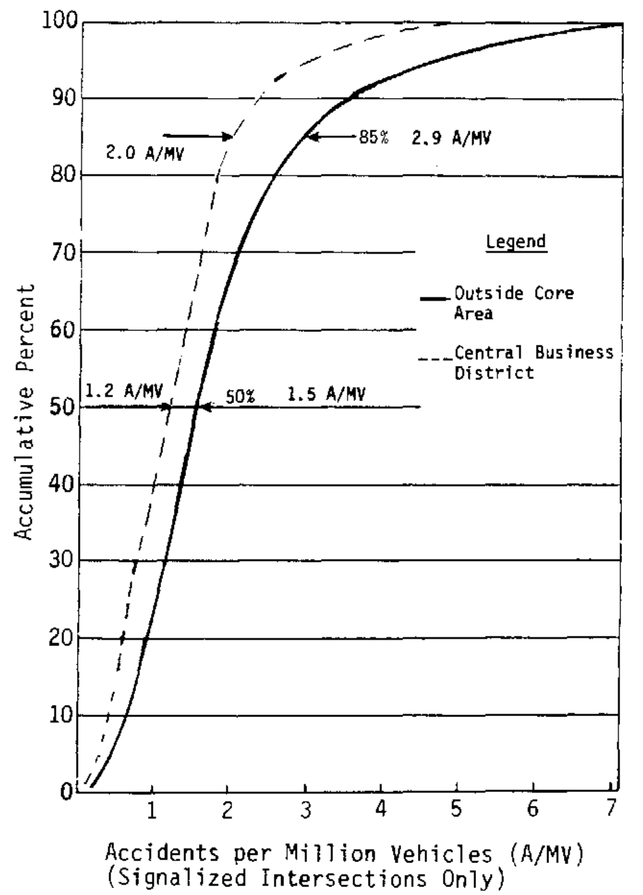

Figure 1. Signalized Intersection Accident Rates (Central Business District vs Outside Core Area)

SOURCE: Adapted by M. J. Rosenbaum from curves of 1964 data furnished in July, 1982 by Bureau of Traffic Engineering, Portland, Oregon.

TABLE 5 - Accidents Before and After One-Way Grid (Portland, Oregon)

Type of Collision	Intersectional Before 1949	Intersectional After 1951	Intersectional % Change	Non Intersectional Before 1949	Non Intersectional After 1951	Non Intersectional % Change
Angle	1461	500	-66	12	3	-75
Rear End	430	202	-53	293	169	-42
Turning	1075	627	-42	95	42	-56
Parking	122	112	-8	1381	819	-40
Sideswipe Meeting	8	2	-75	34	7	-79
Sideswipe Overtaking	198	125	-37	689	574	-17
Head On	4	3	-25	18	2	-89
Non-Collision*	140	85	-39	167	89	-47
Pedestrian	215	115	-46	22	11	-50
Total	3653	1771	-51	2711	1716	-37

*Backing, Fixed Object, Misc.

SOURCE: Reference 9

In 1956, Dallas, Tex., put several streets into one-way operation with extensive advance publicity and preparations prior to the changeover (10). Improved traffic operations were reported. A 6-percent reduction in accidents (from 311 to 293) was found in a before and after study. Dallas also reported an increase in accidents involving turns from the wrong lane (from 8 to 68). This indicated a need to monitor operations and be prepared to identify and correct problems found following the installation of one-way systems.

In San Francisco, Calif., the accident rate on one-way streets was found by Marconi (11) to be lower than on a comparable two-way street. The accident rate was 12.3 acc/MVM on one-way streets and 28.7 on two-way streets. Marconi also reported using two reflectorized one-way signs on the approach to a one-way street as described in the Manual On Uniform Traffic Control Devices (12). With this standard configuration, only 6 of 22,000 accidents involved motorists traveling the wrong way on a one-way street. Where there was a tendency for wrong way travel, "No Right Turn" or "No Left Turn" signs were needed in addition to standard one-way signing.

Ewens (13) reported a study in Hamilton, Ontario, Canada, of 27 miles of one-way street, 12 miles being arterial and 15 miles in the central business district and residential areas. Results showed reductions in most types of accidents and improvements in traffic capacity and travel time. Due to driver unfamiliarity during the first 6 months of operation, accidents increased 5 percent (from 632 accidents to 663) over the same period before the one-way installation. The greatest problem involved motorists turning left from the center of the one-way street during the first few weeks of operation. This was only temporary. In a comparable 6-month period 3 years later, there were 523 accidents, a 17 percent reduction from the period before one-way operation. This reduction of accidents on one-way streets occurred during a period of substantial accident increase on other streets in the city. As shown in Table 6, a drop in pedestrian accidents was an important aspect of the accident reduction attributed to the platooning of vehicles and the creation of safe gaps in traffic for pedestrian use.

During a 2-year period in London, England, ending in 1963, one-way streets were put into operation at 24 locations involving 31 miles of street. The one-way street operation, as reported by Duff (14), reduced injury accidents 19 percent and pedestrian accidents 38 percent. Detailed traffic and accident data for six of these one-way streets are given in Table 7. In most cases, the accident reductions occurred with increases in traffic volume and vehicle travel on the one-way streets.

TABLE 6 - Accidents and One-Way Streets (Hamilton, Ontario, Canada)

Type and Location of Accidents	Before(1) Period (1955-1956)	After Periods(1) (1956-1957)		(1959-1960)	
	Number	Number	Percent Change(2)	Number	Percent Change(2)
Pedestrian Accidents					
One-Way	87	78	-10	29	-67
All Streets	140	177	+26	227	+62
All Accidents					
One Way	632	663	+ 5	523	-17
All Streets	1,989	2,278	+15	2,789	+40

(1) Six-month period November-April
(2) Percent change from before period

SOURCE: Reference 13

TABLE 7 - Accident Changes and Traffic Characteristics on One-Way Streets (London, England)

Street	Miles	Average Weekday Traffic (% Change) Volume	Vehicle Miles	Travel Time (% Change) Offpeak Each Direction		PM Peak Each Direction		Accidents (% Change) Injury	Pedestrian
Tottenham Ct. Rd.(1)	5.1	+4	+8	-49	-34	-43	-14	-21	-33
Baker St.(1)	2.1	+2	+3	-48	-35	-65	-55	+ 4	-8
Earls Ct. Rd.(2)	6.3	+10	+12	-33	-15	-27	-16	-27	-18
Kings Xing(1)	2.5	-2	+18	-28	0	-27	+40	-33	-40
Bond St.(2)	1.3	+9	+14	-26	-38	-15	-38	0	0
Piccadilly(1)	1.3	-4	0	-19	-12	-5	-12	-14	-38

(1) 6-months before and after
(2) 3-months before and after

SOURCE: Reference 14

In 1965, Denver, Colo., converted 3.3 miles of Broadway and Lincoln Streets from two-way to one-way operation. Broadway, a primary arterial street with strip business development, leads to the central business district. A study, reported by Bruce (15), was made of the accident experience at 10 high accident intersection locations along Broadway. With the same traffic volumes before and after the conversion, nine of the 10 locations had a reduction in accidents. The overall reduction for the group of intersections was 29 percent.

In January 1966, New York City converted Fifth and Madison Avenues to one-way operation (15). The changes extended 6.5 miles on Fifth Avenue, from Washington Square to 138th Street, and 5.7 miles on Madison Avenue, from 23rd Street to 135th Street. Average daily traffic volumes ranged from 5,000 to 28,000. The effects of these changes are summarized in Table 8. In a short 5-month before and after study, both streets showed an overall reduction of 153 accidents, or 27 percent, although two sections of Fifth Avenue showed a slight increase in the number of accidents. Midblock collisions decreased 57 percent.

In 1967, Washington, D.C., compared the relative safety of one-way and two-way streets (16). The study was limited to similar one-way and two-way street sections approximately 1 mile long. However, the travel on the one-way section was 13 percent greater. This study found the one-way street having an accident rate of 31.6 acc/MVM, 30 percent lower than the rate for the comparable section of two-way street. The safety advantage of the one-way over the two-way street was restricted to intersections as shown in Table 9.

A 1-year before and after study of sections of Michigan State highways in Lansing and Kalamazoo was reported by Enustun (17). Accidents on sections changed from two-way to one-way operation were compared to the accidents on adjacent sections remaining two-way. As shown in Table 10, a substantial reduction in accidents resulted on the one-way sections. Midblock locations showed the greatest relative safety improvement. The sections remaining two-way did not have comparable safety improvement.

TABLE 9 - Intersection and Midblock Accidents (Washington, D.C.)

	Accidents			
	Intersection		Midblock	
	Injury	Total	Injury	Total
Two-Way (2-yr. Before)	64	212	10	70
One-Way (2-yr. After)	50	175	12	70
Percent Change	-22%	-17%	+20%	0

SOURCE: Reference 16

TABLE 8 - Accidents and One-Way Streets (New York City)

Street and Length Made One-Way		Number of Accidents: Angle	Rear End	Turning	Other	Pedestrian	Total Accidents	Total Injured	Accident Rate(1)
Madison Avenue 23rd St. to 135th St. 5.7 miles	Before Period	23	49	53	67	54	246	167	16.7
	After Period	23	34	24	45	32	158	101	9.3
	Percent Change	0%	-31%	-49%	-33%	-41%	-36%	-40%	-44%
Fifth Avenue Washington Sq. to 38th Street 6.5 miles	Before Period	40	65	68	84	63	326	190	20.4
	After Period	38	53	52	73	45	261	156	13.7
	Percent Change	-5%	-18%	-23%	-13%	-29%	-18%	-18%	-32%
Both Streets	Before Period	63	114	121	151	117	572	357	18.6
	After Period	61	87	76	118	77	419	257	11.6
	Percent Change	-3%	-24%	-37%	-22%	-34%	-27%	-28%	-38%

(1) Accidents per million vehicle miles

SOURCE: Reference 15

TABLE 10 - Before and After Accidents on One-Way and Two-Way Street Segments (Michigan State Highways)

Location of Accidents	Lansing		Kalamazoo	
	Two-Way Street to One-Way Street	Two-Way Street No Changes	Two-Way Street to One-Way Street	Two-Way Street No Changes
Signalized				
Before	69	55	147	56
After	46	61	125	58
% Change	- 33.3	+ 10.9	- 15.0	+ 3.6
Nonsignalized Intersections				
Before	36	22	19	2
After	38	30	21	0
% Change	+ 5.6	+36.4	+10.5	-
Midblock				
Before	65	44	180	
After	32	43	111	22
% Change	-50.8	- 2.3	-38.3	- 8.3
Total Accidents(a)				
Before	173	121	357	82
After	133	134	267	80
% Change	- 23.1	+ 10.7	-25.2	- 2.4

(a) Includes accidents in addition to those tabulated by location

SOURCE: Adapted from Reference 17

Light condition (daylight vs. night) was not an element included in most studies of one-way streets. With one-way operation the accident causal factor of oncoming headlights would be eliminated. The comparison of daylight and night accidents in Table 11 indicates the potential safety benefits of one-way operation at night. In both Lansing and Kalamazoo, night accidents were reduced on sections changed from two-way to one-way. The reductions were of the same or greater relative magnitude than those for daylight. Neither city showed a similar night safety improvement on the study sections remaining two-way.

Relationship to Pedestrian Safety

Pedestrians have usually benefited from conversions from two-way to one-way street operation. For example, in Baltimore (18) when four streets were so converted, pedestrian accidents were reduced from 44 to 35. In New York City, the conversion of north-south avenues in Manhattan to one-way operation has consistently been followed by decreases in pedestrian accidents of from 10 to 30 percent. Wiley (19) attributes this to the tendency of progressive signal timing to group vehicles into platoons, creating clearly defined gaps which pedestrians can use for crossing. Also, fewer vehicle stops are required resulting in fewer violations of red signal indications.

The operational problem of conflicts on one-way streets due to turns from the wrong lane suggests a need for improved signing and traffic markings to inform motorists of the one-way operation. This may possibly include supplemental signing before major intersections, directing left turning traffic to the left lane.

REVERSIBLE LANES

This section includes a summary of a number of reversible lane installations. A brief description of the operational modifications and the resulting changes in safety (accident reduction or increase) are included. Some of the material in this section has been taken from a "Survey of Reverse Lanes" prepared by Lalani (20) prior to continuing studies being conducted in Phoenix, Ariz.

Arlington, Virginia

A 1.5 mile section of a 28-foot-wide width three-lane highway (U.S. Routes 29 and 211) extending westward from Key Bridge into Arlington County carried, in 1949, in excess of 20,000 vehicles per day (21). Traffic was expected to materially increase upon completion, in Washington, D.C., of the Whitehurst Freeway on the east side of the Potomac River. The highway is intersected

TABLE 11 - Daylight and Night Accidents (Street Segments of Michigan State Highways)

Light Condition	Lansing: Two-Way Street to One-Way Street	Lansing: Two-Way Street No Changes	Kalamazoo: Two-Way Street to One-Way Street	Kalamazoo: Two-Way Street No Changes
Daylight				
Before	123	94	232	52
After	96	97	193	52
% Change	-21.9	+ 3.2	-16.8	0
Night				
Before	39	22	111	26
After	31	31	63	24
% Change	-20.5	+40.9	-43.2	-7.7
Twilight				
Before	11	5	14	4
After	6	6	11	4
% Change	-45.4	+20.0	-24.4	0

SOURCE: Adapted from Reference 17

by numerous lateral streets serving local residential areas. While there were no really large volumes on any of these intersecting streets, volumes were sufficiently large to result in considerable accumulated delay to side street traffic. The situation was regarded as intolerable by nearly all users of the side streets. With no funds available for additional construction, it was agreed the artery must be signalized to apportion some of the side street delay to the artery by giving side street traffic more opportunity for entrance. The arterial directional movements during peak hours were found to be exceptionally unbalanced. Eleven intersections were signalized and controlled by master equipment to provide two inbound lanes during the morning peak, two outbound lanes during the afternoon peak, and two-way center lane use during offpeak periods.

A comprehensive study of lane use, travel time, capacity, overall volumes, parallel route use, and delay was conducted three months before signals were installed. A similar followup study was conducted 9 months after signal operation began.

The signalization was found to handle larger volumes, require slightly longer travel time, and result in more orderliness of all movements, more overall standing delay, more traffic accidents, little or no diversion to parallel routes, limited illegal use of lanes, remarkable acceptance by users, and vastly improved public relations.

Entirely too many drivers violated red light indications. During the before and after periods reported accidents increased from 11 to 35 or 218 percent.

A reversible lane system was installed in 1974 on 2.5 miles of Wilson Boulevard, an urban arterial (20). The system uses 22 spans of lane control signals. An amber "X" is displayed for clearance prior to reversing a lane. During offpeak periods, two lanes operate in each direction. During peak hours, directional lights change to provide a "bus only" lane and two other lanes for peak hour flow. The remaining lane operates counter to the peak hour flow. Bus travel times decreased substantially in both the a.m. and p.m. peak periods. The passenger car volumes also increased. A 1-year before and after study showed accidents increased 30 percent. Many of these accidents were caused by drivers turning left from the wrong lane. To reduce accidents, p.m. peak hour operation was shortened from 2-1/2 to 2 hours. An additional span of lane control signals has been installed in a retail area with high turning accidents. Signing and pavement marking for priority lanes were changed to include the diamond symbol. Signs regulating use of the curb lane during reversible hours were also changed to include the diamond symbol. Accidents declined since the high point during the 1-year period after installation.

ATLANTA, GEORGIA

A three-lane section of Memorial Drive, with two lanes marked outbound and one lane marked inbound, operated parallel to I-20 as a major arterial for commuting motorists (20). Delays were frequent due to restricted travel lanes inbound and the independently signalized intersections using outdated equipment.

improvements included use of the center lane as a reversible lane, interconnection of the traffic signal system, and modernization of signal head displays.

Following improvements, studies showed significant improvement in travel times. A 25-percent reduction in morning and afternoon peak hour travel times and a 5 to 10 percent reduction in offpeak travel times resulted. Accidents decreased by 25 percent.

Chicago, Illinois

Chicago used overhead lane signals on the Hollywood Ridge system for a number of years with very good results (22). Four-lane streets provided three lanes in the peak direction. Lane signals were the "red X" and "downward pointing green arrow." The entire system was slightly more than 1 mile long. Studies showed two-way volumes increasing by 50 percent from 2,250 to 3,550 vehicles per hour during the peak hour. No significant changes in accidents were reported. In over 9 years of operations, there were no head-on collisions involving any of the reverse lanes.

Chicago's eight-lane Lake Shore Drive, using three sets of hydraulically operated divided fins, provided an example of another specialized reversible lane technique that was probably not applicable to most situations. This reverse lane is now being abandoned due to accident and maintenance problems along the remaining sections of the reverse lanes where the hydraulic fins were not used (20).

Detroit, Michigan

Grand River Avenue, with strip commercial development, has operated for many years with four lanes in the peak direction and three lanes in the opposing direction during peak hours (22). Throughout the 13 miles large permanent 3 foot by 3 foot signs are hung over the center 10-foot left turn lane at two block intervals with alternating messages for the inbound traffic:

- o THIS LANE THRU TRAFFIC ONLY 7-9 A.M. MON THRU FRI
- o NO LEFT TURN 7-9 A.M. 4-6:30 P.M. KEEP OFF 4-6:30 P.M. MON THRU FRI
- o LEFT TURN ONLY THIS LANE EXCEPT 7-9 A.M. 4-6:30 P.M. MON THRU FRI

The outbound signs have the times reversed.

Before and after studies found peak direction travel time reduced. Volume in the peak direction increased 41 percent. Initially, during peak hours, traffic volumes were 40,000 to 50,000 vehicles per day and heavy enough to discourage violation of the left turn restrictions. A new freeway has caused volumes to decline to 30,000 vehicles per day with an increase in the number of left turn violations.

Detroit also installed a combination sign and signal arrangement over the center lane of Michigan Avenue, a five-lane roadway (22). The sign displayed either a red X with "NO LEFT TURN" legend, a green arrow with "NO LEFT TURN" legend, or no symbol with the "ONLY LEFT TURN" legend. This last legend was for offpeak periods allowing the center lane to be used for two-way left turns. Parking was prohibited at all times. Traffic volumes in the morning peak direction increased from 4 to 20 percent and in the evening peak period up to 76 percent. Travel times during these two periods decreased 19 and 20 percent, respectively. Average speeds increased 23 percent. Most of an overall 19-percent accident decrease was due to a 93-percent accident decrease associated with parked vehicles due to the full-time prohibition of parking. The decrease in nonparking accidents was 4 percent.

Lexington, Kentucky

Agent and Clark (23) reported on a reversible lane system on Nicholasville Road in Lexington. Before the change, two lanes served each direction with the center lane being used for two-way left turns. A reversible lane system was installed on a 2.6-mile section operating with three lanes in one direction during the peak hours. There was an increase in accidents from 360 to 399 (11%) during the operation of the reversible lanes. The increase in accidents during reversible lane operation was identical to the increase during other times. Two types of accidents were attributed to the reversible lane operation. One involved drivers desiring to make a left turn moving into the left turn lane far upstream from the left turn location. This usually occurred during evening operation in the offpeak direction as drivers attempted to avoid long delays. The other types involved drivers attempting to turn left into a driveway across three opposing lanes of traffic. A summary of accident type and severity is given in Table 12.

Los Angeles, California

Los Angeles was the first city to use reversible lanes during peak periods (22). The lanes were delineated mostly by manually placed cones and signs. The first installation was made in 1928. By 1967 there were approximately 13 miles of streets using reverse lanes. In general the accident rate per million vehicle miles was much less on those streets than on major streets where lane reversal was not used. Field observations indicated more satisfactory operation in terms of smoothness of flow and frequency of stops. Decreases in travel times ranged from 1 to 15 minutes.

TABLE 12 - Summary of Accidents by Accident Type and Severity
(Lexington, Kentucky)

	Number of Accidents							
	AM Peak*		PM Peak**		Offpeak		Total	
	Before	After	Before	After	Before	After	Before	After
Accident Type								
Angle	9	7	33	37	109	117	151	161
Rear End	18	18	27	37	84	100	129	155
Same Direction Sideswipe	7	4	11	13	42	42	60	59
Opposite Direction Sideswipe or Head-On	0	1	0	5	1	5	1	11
Fixed Object or Single Vehicle	1	0	1	1	11	7	13	8
Bicycle	0	0	1	0	1	2	2	2
Pedestrian	2	0	1	0	1	3	4	3
Accident Severity								
Property Damage Only	27	25	59	70	202	210	288	305
Possible Injury	3	3	10	13	22	29	35	45
Non-Incapacitating	6	1	3	8	15	23	24	32
Incapacitating	1	1	2	2	10	14	13	17
Fatality	0	0	0	0	0	0	0	0
Totals	37	30	74	93	249	276	360	399

* Monday through Friday, 7 - 9 a.m. (carries the lowest directional volume in this direction.)

** Monday through Friday, 4 - 6 p.m.

One year before and after periods

SOURCE: Reference 23

Memphis, Tennessee

Upchurch (24) reviewed a reversible lane study conducted in Memphis. Lane use control signals using the "red X" and "green arrow" were installed on a 4-mile segment of Union Avenue, an arterial used by commuters to reach the central business district. The six-lane avenue included two 12-foot curb lanes and four 10-foot center lanes numbered one to six as shown in Figure 2. Land use along Union Avenue was generally commercial. The westward end of the study section included a large medical complex. The avenue was also a major bus transit route. The reverse lanes operated as a four-to-two split during peak hours and as three lanes in each direction during all other times. Parking was prohibited at all times. The overhead lane control signals were located about every 650 feet over lanes 2, 3, 4, and 5.

Supplementary traffic control signing was installed throughout the study section. The entire length included "NO PARKING OR STANDING AT ANY TIME" signs. The speed limit was 35 miles per hour. Left turns at most signalized intersections were prohibited at all times with "NO LEFT TURN THIS LANE" signs suspended over the relevant lanes. These were provided to prevent left turns across four lanes of opposing traffic during periods of reverse lane operation.

A review of 817 accidents for 1972 provided collision diagrams and written descriptions by which the nature of the facility could be judged a contributor to accidents. These reports showed 137 accidents (16.8 percent of the total) did have a direct relation to the reversible nature of the facility. One hundred and eleven accidents related to a left turn being made across a lane designated for flow in the same direction, as shown in Figure 2. Five accidents resulted from confusion between a traffic signal and a lane control signal.

Ninety additional accidents occurred at signalized locations where a motorist disregarded a red light.

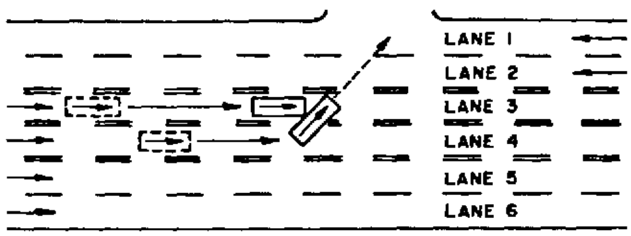

Figure 2. Reversible Lane Configuration and Typical Left Turn Collision, Memphis, Tenn.

SOURCE: Reference 24

Head-on accidents or sideswipes resulting from vehicles traveling in opposite directions in the same lane were not a problem. Only one accident of this type was recorded. Observation of compliance or noncompliance to the overhead lane markers showed that compliance was generally good.

Confusion between lane control markers and traffic signals at intersections was found to be a problem contributing to at least a small number of accidents. In planning new reversible lane facilities, lane control markers should be located so that horizontal and vertical curvature are not in the driver's same line of sight as traffic signals but far enough from intersections so they cannot be confused with traffic signal indications.

Milwaukee, Wisconsin

Milwaukee used overhead lane signals and manually placed cones on a six-lane street designated as an "interim freeway terminal distributor system" (25). This seven-block stretch of West Clynborn Street was placed in reversible lane operation in 1963. Two-way traffic volumes were 55,000 vehicles per day, with up to 4,000 vehicles per hour in the peak direction. No changes in accident levels were reported.

Newark, New Jersey

Newark's reverse lane operation (26) used cones and signs on 0.6 miles of Broad Street, a major north-south arterial street serving downtown Newark. Approximately 37,000 vehicles per day used this experimental section. Peak hour a.m. and p.m. volumes exceeded 1,750 vehicles southbound and 2,700 vehicles northbound.

Greatly improved quality of flow resulted for southbound traffic in the a.m. peak hour. This included savings of 3 minutes per mile in trip time, 2 minutes per mile in delay time, and 3.8 fewer stops per mile. Average speeds increased from 8.09 to 13.67 mph, with a 10-percent increase in traffic volume. No accidents were reported due to the reversible lane operation. Some of the portable signs were struck during hours of darkness as indicated by physical evidence. The reverse operation is now permanently installed using overhead signs and signals.

Phoenix, Arizona

1. A 1.38-mile section of 15th Avenue in Phoenix was placed in reversible lane operation in 1959 (20). The 36-foot pavement was striped as three 12-foot lanes. Overhead signs facing in both directions indicated when the center lane was to be used by southbound traffic during a.m. hours and by northbound traffic during p.m. hours. Average daily traffic volumes increased 10 percent from 1957 to 1962. A 4-year accident study showed accidents had increased 54 percent. There was a 214-percent increase in left turn accidents and a 140-percent increase in rear end accidents.

These occurred mostly at intersections. The left turn accidents were caused by vehicles attempting to turn left from the curb lane colliding with the reverse lane traffic traveling in the same direction. The rear end accident increase was not unexpected. The reversible lane type operation provided three lanes of moving traffic, instead of two, creating more opportunity for rear end collisions. In addition, motorists attempting to turn left across two opposing lanes of traffic were thought more likely to slow or stop more suddenly than if only one opposing lane had to be crossed.

A followup study of accidents in 1974 indicated the left turn accident problem had not subsided and the rate of accidents was still high on this reversible lane. By examining the time at which the accidents occurred, it became apparent the problems were related to offpeak hours when the use of the center lane as a two-way left turn lane was not thoroughly understood by some motorists.

The reverse lane operation on 15th Avenue was abandoned because of traffic safety considerations.

2. A barricade controlled reversible lane operation was initiated on Washington Street in Phoenix in 1978 (20). Prior to inception of the reversible lane, the roadway was striped for three lanes of eastbound traffic, two lanes of westbound traffic, a continuous two-way left turn channel in the center of the roadway, and left turn pockets at signalized intersections. For the reverse lane operation, the roadway was restriped so the left turn channels of unsignalized intersections became part of a continuous left turn lane. When reverse lane operations were begun, portable barricades were placed between eastbound and westbound traffic. The two-way left turn lane and the adjacent eastbound lane became two additional lanes for the use of westbound a.m. peak traffic. Additional signs were erected to enforce this lane utilization each weekday between 6:30 and 9 a.m. A two-man team requires 2 hours to place and remove the barricades.

Peak hour volumes in the 6:30 to 9 a.m. period increased by 11 percent in 3 months (seasonally adjusted). The potential increase in volume is limited by the west end of the reverse lane operation where the four traffic lanes available westbound in the morning are required by geometrics to narrow to two lanes before widening out into a five-lane one-way street. A lane usage study shows the reverse lane adjacent to the barricades, separating the four westbound lanes from the two eastbound lanes, is used by only 6 percent of the 6:30 to 9 a.m. peak period westbound traffic. The probable reasons are the left turns still being permitted from this lane and the drivers' dislike for the lane because of proximity to the barricades. The eastbound traffic in the offpeak direction on this section of reverse lane is not permitted to turn left at some intersections. Analysis of before and after accidents indicated left turn and angle accident increases. These involve vehicles entering or leaving local streets and driveways having to turn across or merge with four lanes of westbound traffic rather than two. The number of these accidents was small due to limited operation both in length of roadway and hours of operation.

3. A reversible lane was put into operation on Seventh Avenue in Phoenix in 1979 (20). The roadway was, for the most part, originally striped for three lanes northbound and two lanes southbound with separate left turn lanes at intersections. The reverse lane striping converted all separate left turn lanes into part of a continuous two-way left turn lane for offpeak use. During peak hours, this lane operated as the reversible lane. Figure 3 shows the before and after lane configurations. Left turns were prohibited at almost all signalized intersections during reverse lane operation to prevent the reverse lane from being blocked by vehicles waiting to turn. The overhead and supplemental signs used are shown in Figures 4 and 5, respectively. A press release and leaflets sent out with water billings advised the public of the upcoming change. Accident data were analyzed for matching 7-month before and after periods. Results showed:

- Accidents increased overall 20.3 percent.
- Signalized intersection accidents remained unchanged.
- Midblock and nonsignalized intersection accidents increased 36.4 percent.
- No significant change was found in weekday a.m. peak periods and all offpeak hours.
- An accident increase of 173.3 percent was found during weekday p.m. peak periods with midblock and nonsignalized intersection accidents increasing 316.6 percent during these periods. This was caused mainly by a dramatic rise in sideswipe and improper left turn accidents.

A summary of the number of accidents by accident type during the 7-month before and after study is given in Table 13.

As a result of the increase in accidents, concentrated police enforcement was instituted during the p.m. peak period resulting in 402 citations being issued. Table 14 gives the number of violations by type. An educational campaign was conducted including a post card survey of observed license plates, a telephone survey, public service announcements on local television and radio stations, city personnel participation at public meetings, and the mailing of a new leaflet. The foregoing campaign did not have the desired effect as shown in Table 15, a summary of the final before and after accident study.

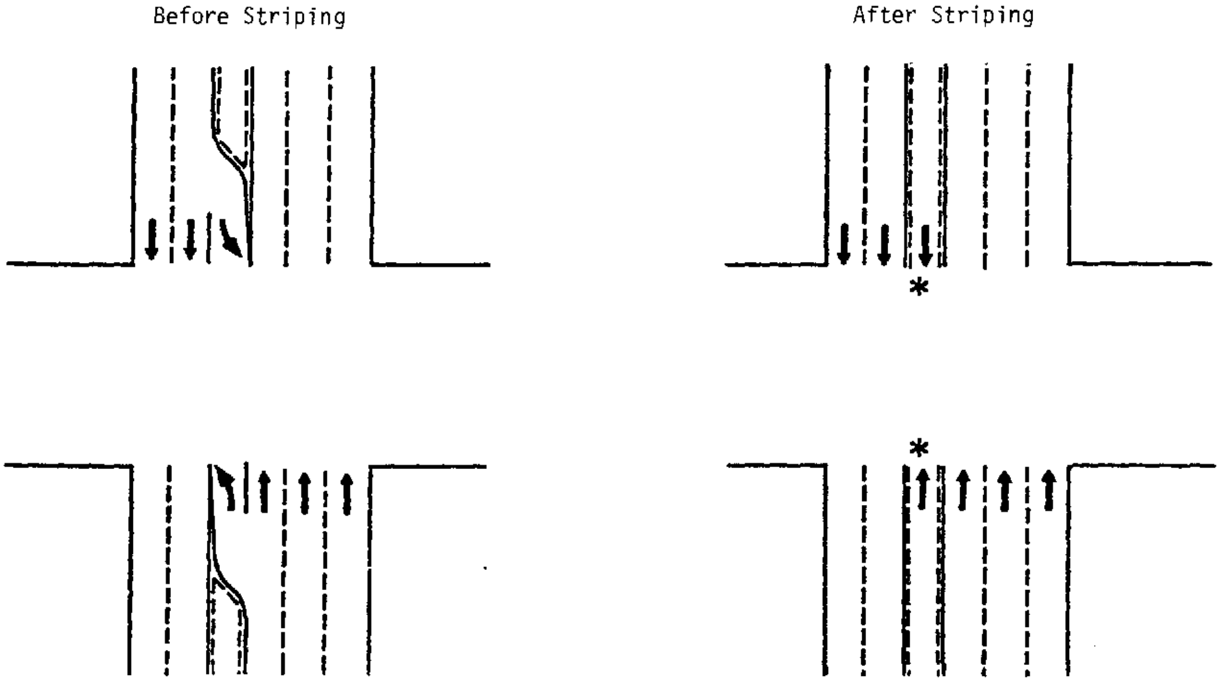

Figure 3. Lane Configuration Changes on Seventh Avenue for Reversible Lane Operations, Phoenix, Ariz.

SOURCE: Reference 20

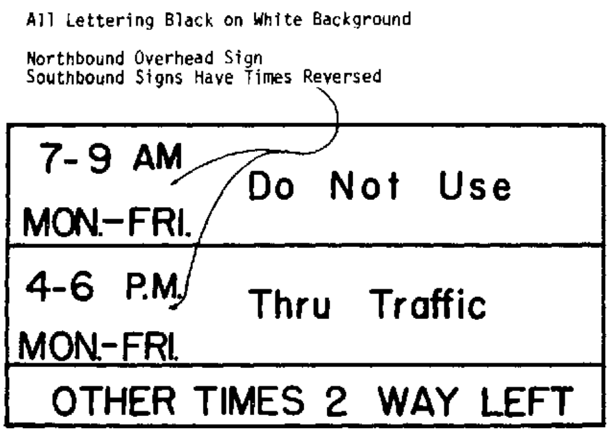

Figure 4. Overhead Sign Used on Seventh Ave. Reversible Lane (Phoenix, Ariz.)

SOURCE: Reference 20

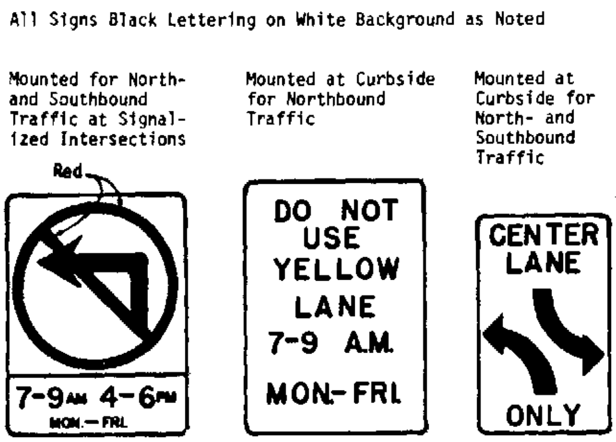

Figure 5. Supplementary Signs used on Seventh Ave. Reversible Lane (Phoenix, Ariz.)

SOURCE: Reference 20

TABLE 13 - Seven-Month Before and After Study of Accidents on Seventh Avenue (Phoenix, Ariz.)

Manner of Collision	Number of Accidents							
	7 - 9 a.m.		4 - 6 p.m.		Offpeak		All Accidents	
	Before	After	Before	After	Before	After	Before	After
Single Vehicle	1	0	1	0	5	7	7	7
Angle	7	4	9	12	52	51	68	67
Left Turning	4	2	7	6	22	25	33	33
Rear End	9	6	9	16	48	34	66	56
Head-On	0	0	0	1	1	1	1	2
Sideswipe	0	5	3	17	8	7	11	29
Pedestrian	1	0	1	1	2	2	4	3
Improper Left Turn	0	2	0	29	2	3	2	34
Injury	10	10	14	18	55	58	79	86
All Accidents	22	19	30	82	140	130	192	231

Before period from January 11, 1978, to July 31, 1978
After period from January 11, 1979, to July 31, 1979

SOURCE: Reference 20

TABLE 14 - Citation Types Issued During Concentrated Police Enforcement on Seventh Avenue

Type of Violation	Number of Citations	Percent
Failure to obey no left turn signs at signalized intersections	240	59.7
Entering yellow lane against overhead sign	106	26.4
Using private property to make a left turn	16	3.9
Driving without current vehicle registration	9	2.3
Driving without a current drivers license	15	3.7
Failure to yield from a stop sign	4	1.0
Failure to yield right-of-way when turning left	3	0.8
Running a red light	3	0.8
Unsafe lane change	1	0.2
Driving too fast for conditions	4	1.0
Failure to drive in one lane	1	0.2
Total	402	100.0

SOURCE: Reference 20

TABLE 15 - Final Before and After Study of Accidents on Seventh Avenue (Phoenix, Ariz.)

	Number of Accidents							
	7 - 9 a.m.		4 - 6 p.m.		Offpeak		All Accidents	
Manner of Collision	Before	After	Before	After	Before	After	Before	After
Single Vehicle	0	2	1	0	7	5	8	7
Angle	13	10	10	14	67	65	90	89
Left Turning	2	1	14	6	27	23	43	30
Rear End	7	14	11	26	44	38	62	78
Head-On	1	0	0	0	0	0	2	0
Sideswipe	2	6	2	15	16	9	20	30
Pedestrian	0	0	0	1	0	2	0	3
Improper Left Turn	0	5	1	27	3	4	4	36
Injury	5	15	15	27	61	59	84	101
All Accidents	25	38	39	89	164	146	229	273

Before period from April 1, 1978, to December 31, 1978
After period from April 1, 1979, to December 31, 1979

SOURCE: Reference 20

Tucson, Arizona

Three techniques for controlling reversible lane traffic flow have been employed in Tucson. Each makes provision for offpeak operation of the center lane for two-way left turns.

An unpublished report by Nassi (27), compared the use of "signs only" with "control signals and supplementary signs" and "manually placed cones and supplementary signs." His study of accidents for the three operational techniques found the reversible lane related accident rate to be basically equal regardless of the type of traffic control or age of the reversible lane operation. The majority of the accidents are left turn related, even though the left turns are restricted on all reversible lanes.

Table 16 depicts the percentage of accidents by accident type and total accident rates for each of the reversible lane traffic control techniques used. A rating of "Advantages/Disadvantages" for the three techniques is given in Table 17.

Brief descriptions of the techniques are cited in the following paragraphs.

1. A variable message sign and signal system was installed in 1971 on a 2-mile section of "Broadway," a major arterial. Traffic control included changeable message signs reading "LEFT TURN ONLY" over the two-way left turn lane during offpeak hours in conjunction with a flashing amber "X". For reversible lane operation, this changed to "NO LEFT TURN" with an associated "green arrow" or "red X" according to the peak flow direction being accommodated. Pavement markings for the center reversible lane were of the two-way, left turn yellow lane type. Traffic volumes have increased 23 percent and travel speeds 4 mph (statistically significant). Accidents have decreased in midblock locations 35 percent and increased 24 percent at intersections, mainly due to rear end accidents increasing at a particular intersection. There have been no accident increases associated with the reversible lane operation.

TABLE 16 - Comparison of Reversible Lane Traffic Control Accident Data (Tucson, Ariz.)

Accident Type	Percentage of Accidents Signs & Signals	Signs & Cones	Signs Only
Rear End	44	19	35
Sideswipe Out of Reversible Lane	9	8	30
Sideswipe Into Reversible Lane	13	19	9
Left Turn from Reversible Lane	4	8	4
Left Turn Across Reversible Lane	9	27	13
Head On	17	3	0
Fixed Object	0	8	0
Other	4	8*	9
Total	100	100	100
Reversible Lane Related Accident Rate per MVM	.861	.804	.705

*One accident involved a vehicle striking the cone setting truck

MVM - Million Vehicle Miles

SOURCE: Reference 27

TABLE 17 - Rating of Advantages/Disadvantages of Reversible Lane Traffic Control Techniques (Tucson, Ariz.)

	Signs Only	Signs& Cones	Signs& Signals
Initial Installation	5	4	1
Annual Cost	5	1	2
Travel Time Reduction	3	3	3
Operating Speed	3	3	3
Reversible Lane Accident Rate	3	3	3
Traffic Control Device Visibility	3	2	3
Perceived Understanding	3	3	3
Left Turn Violations	3	3	3
Driver Delay Before Usage	2	5	4
Failure to Activate	5	1	1
Vandalism	5	2	5
Hazard to City Personnel	5	1	4
Holiday Exceptions	1	5	5

5 4 3 2 1

← Better ↑ Worse →

No Difference

SOURCE: Reference 27

2. A 4-mile reversible lane was installed in 1975 and operated by signs and the manual setting of traffic cones daily to identify the reversible lane. This technique is used on a street where peak hour traffic uses the reversible lane as a two-way left turn lane during offpeak hours. Peak hour traffic must travel to the left of the centerline for approximately one-half the length of the system. Traffic cones are manually set daily from 7 a.m. to 9 a.m. and 4 p.m. to 6 p.m., identifying the lane boundaries during the peak periods. Three teams of two men each require 40 minutes to setup or pickup the traffic cones. Traffic delays are sometimes caused during cone placement. Since the cones can be blown over or knocked out of place, the crews patrol the reversible lane approximately every half hour.

Traffic volumes have increased 27 percent and travel speeds increased 4 mph (statistically significant). Accidents overall have not changed. Segment accidents have decreased 19 percent while intersection accidents have increased 19 percent. The changes in accidents are not related to the reverse lane operation.

3. The most recent reversible two-way lane operation in Tucson operates with signs alone. Drivers are given notice of the reversible two-way left turn lane operation with the use of overhead lane control type signs and supplemental side mounted restricted lane control signs. The overhead signs are spaced approximately one every 0.25 miles with the supplemental side mounted signs generally placed between them. An advance warning sign with beacons is only activated when the reversible lane is in operation.

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CHAPTER 8 - PRIORITY FOR HIGH-OCCUPANCY VEHICLES (HOV)

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INTRODUCTION

Priority measures are incentives to increase the use of high-occupancy vehicles (HOV) such as buses, vans, and cars used for carpools. These measures alter the design and/or operation of streets and freeways to cause a reduction in HOV travel times and/or an improvement in schedule reliability. Such measures have been applied increasingly throughout the world. Freeway priority measures are now in operation in at least 16 United States cities. Over 30 cities have implemented arterial street priority measures. Freeway priority measures have been implemented primarily since 1970 (1, 2, 3). Buses, the dominant form of public transportation in the United States, carry over two-thirds of total transit patrons (4). Priority measures, for buses on arterial streets have existed since 1939 (5). Although the total number of implemented bus priority measures is not large, an increasing number of projects are being considered as greater emphasis is given to transportation system management, energy availability, and air quality. As of April 1980, there have been 79 HOV projects implemented nationwide with an additional 66 projects planned or under study (6).

Nine alternative priority measures are addressed in this chapter. Four relate to arterial streets and five relate to freeways. Examples of each measure are presented in Table 1.

TABLE 1 - Significant Examples of Priority Measures

TYPE OF TREATMENT	SIGNIFICANT EXAMPLES
ARTERIAL RELATED	
Signal Preemption Systems - Buses receive preferential treatment through limited signal control at signalized locations.	N.W. 7th Avenue, Miami, Florida North Central Corridor, Dallas, Texas Concord, California
Concurrent Flow Lane - An arterial street lane designated for HOVs but not physically separated from the general traffic lanes. HOV travel is in same direction as general traffic.	South Dixie Highway, Miami, Florida Kalanianaole Highway, Honolulu, Hawaii
Contraflow Lanes - An arterial (or one way street) lane designated for HOV travel in a direction opposite that of the adjacent general traffic lanes.	N.W. 7th Avenue, Miami, Florida South Dixie Highway, Miami, Florida Ponce de Leon/Fernandez Juncos, San Juan, Puerto Rico
Separate HOV Facility - An arterial street designated for exclusive use of buses, sometimes called a transit way.	Nicollet Mall, Minneapolis, Minnesota Portland Mall, Portland, Oregon Chestnut Street, Philadelphia, Pennsylvania
FREEWAY RELATED	
Priority Entry - A separate ramp or a by-pass of ramp metering designated for HOVs.	I-5, Seattle, Washington I-605, Los Angeles, California I-35W, Minneapolis, Minnesota
Toll Plaza Lane - Lane(s) designated for HOV entry to toll facility.	San Francisco-Oakland Bridge, Oakland, California.
Concurrent Flow Lane - A freeway lane designated for HOVs but not physically separated from the general traffic lanes. HOV travel is in the same direction as general traffic.	U.S. 101, San Francisco, California Moanalua Freeway, Honolulu, Hawaii I-95, Miami, Florida
Contraflow Lane - A freeway lane designated for HOV travel in a direction opposite that of the adjacent general traffic lanes.	I-495 (Lincoln Tunnel), New Jersey Long Island Expressway, New York New York I-45, Houston, Texas
Separate HOV Facility - Lanes physically separated from other freeway lanes and designated for HOV use exclusively.	I-395, Washington, D.C. San Bernardino Busway, Los Angeles California South PATway Busway, Pittsburg, Pennsylvania.

Signal preemption systems enable buses to control cycles at signalized intersections, providing a potential reduction in bus travel time. Some 36 systems have been installed throughout the world ranging from single intersections to 62 intersections in Dallas, Texas (7). The most effective application of this technique is for express buses. Preemption can provide a 10% to 15% reduction in bus travel time in downtown areas. As much as a 30% reduction in travel time can be achieved for express buses traveling in a reserved lane (1). An example of a preemption system (N.W. 7th Avenue, Miami, Florida) is provided in Figure 1.

Two basic systems have been employed to preempt the signal by approach buses. One system involves a rapidly flashing light mounted on top of the bus. The light flashing about 20 flashes a second is sensed by a detector mounted at the signal location to note when the bus is approaching. The other system uses a small radio transmitter mounted under the bus with the signal being received by a loop antenna in the pavement on the approach to the signalized intersection. Both systems are commercially available.

Operational and Accident Data

A detailed evaluation of the N.W. 7th Avenue bus priority system in Miami was recently completed (8, 9). A total of 37 signals were equipped with preemption equipment. Some of the more pertinent data associated with that system is presented in Table 2. In Miami safety increased with the introduction of express bus service and signal preemption. However, new signals, pavement markings, and signing were also installed at the same time. Five bus accidents occurred on the project. Although this results in a bus accident rate in excess of the countywide transit system rate, this is not a statistically significant difference. The total facility accident rate decreased with the introduction of express buses and signal preemption. The decrease in accident rates reflected in Table 2 is statistically significant: There were no major changes in either the percentage of injury accidents or the type of vehicle involved in the accidents. At intersections, there was an increase noted in the percentage of accidents involving vehicles traveling in the same direction, and there was also an increase in the percentage of accidents involving vehicles traveling in opposite directions.

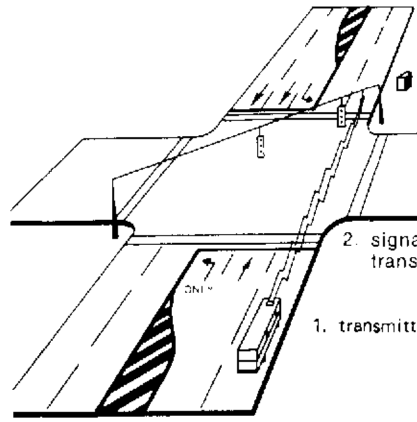


Figure 1. Bus Priority Signal Installation with Flashing Light Bus Transmitter

SOURCE: Reference 8

A demonstration project (10) conducted on eight intersections in Sacramento, Calif., concluded that introduction of the Greenback Lane with unconditional signal preemption for express buses:

- o Reduced bus trip time by 23 percent.
- o Improved reliability of bus trip time and scheduling.
- o Improved the quality of traffic flow along the route, including a decrease in delay to all vehicles.
- o Did not increase delay to cross street traffic.

Similar favorable results were found on 12 intersections with buses in Concord, Calif.:

- o Reduced bus trip time by 10-20 percent (0.8-1.2 minutes/mile).
- o Buses encountered zero delay at intersections 50-60 percent of the time.
- o Non HOV and cross street delays were negligible.
- o Average of four buses per hour in each direction along the project.

The Greenback Project in Sacramento, Calif., had no change in traffic accident rates, nor was there a significant change in the Concord, Calif., rates (10, 11).

TABLE 2 - Operational and Accident Characteristics of Miami, Florida, N.W. 7th Avenue, Signal Preemption System, 1977

VARIABLE	UNIT	BEFORE	AFTER	BEFORE	AFTER
Critical Peak Period		7 - 9 AM	7 - 9 AM	4 - 6 PM	4 - 6 PM
Length of BP System	Miles	-	9.9	-	9.9
Total Peak Directional Lanes	Lanes	2	2	2	2
Volume, All Lanes	Vehicles	1,461	1,655	1,825	1,905
Volume - Buses	Vehicles	-	23	-	21
Bus/Total Volume	%	-	1.4	-	1.1
Auto Occupancy - All Lanes	PPV	1.3	1.3	1.5	1.4
Person Throughput - All Lanes	Persons	1,895	2,777	2,641	3,221
Person Throughput - Buses	Persons	-	673	-	570
Bus/Total Throughput	%	-	24.2	-	17.7
Speed - Automobile	MPH	21.0	23.0	19.8	23.1
Speed - Bus	MPH	22.7	28.1	20.1	26.8
Travel Time - Automobile	Minutes	28.3	25.8	30.0	25.7
Travel Time - Bus	Minutes	26.2	21.1	29.6	22.2
No. of Accidents	No.	30	18	31	30
Accident Rate	Acc/MVM	11.0	3.3	9.4	4.8

SOURCE: Reference 8

CONCURRENT FLOW ARTERIAL LANE

A concurrent flow lane is a curbside or median lane not physically separated from other general traffic lanes and designed for use by HOV in the same direction of flow as general traffic lanes. A representative cross section of this priority treatment measure, a Washington, D.C., central business district (CBD) street, is depicted in Figure 2.

Curbside lanes have historically been implemented in areas such as the CBD to provide improved transit circulation. Right turning vehicles are frequently allowed to use these lanes. Median concurrent flow lanes reduce HOV travel times allowing those vehicles to bypass congestion in general traffic lanes. Left turning vehicles are sometimes allowed to use median concurrent flow lanes. The median lanes primarily serve express bus operations, while curbside lanes serve local bus operations.

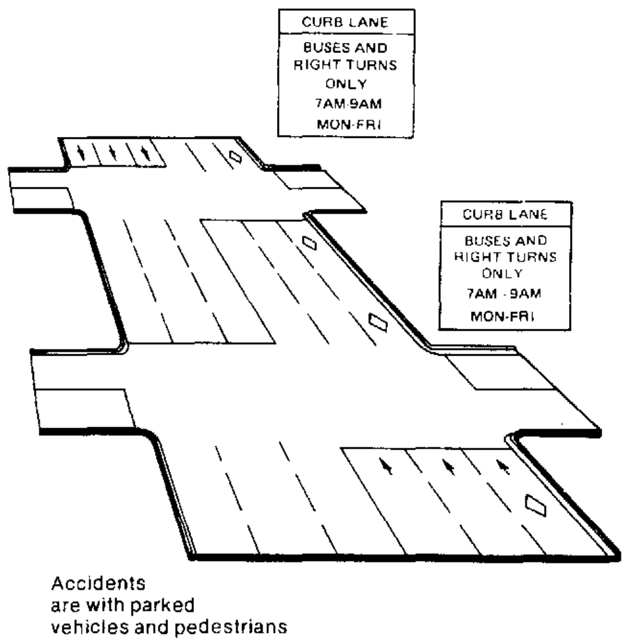


Figure 2. Typical Curb Bus Lane on Arterial Street

SOURCE: Reference 8

Operational and Accident Data

Peak period operational data (the time period during which the impact of the HOV lane is greatest) are presented in Table 3 for four selected concurrent flow projects. Additional operational data from these and other projects are highlighted below.

- o Where the HOV lane provides no travel time advantage, as in the Washington CBD and N.W. 7th Avenue projects, there is nearly a zero violation rate by through-moving vehicles. Where the HOV lane has a travel time advantage, as in the U.S. 1/South Dixie Highway and Kalanianaole (Hawaii) Highway projects, closer enforcement of the HOV restrictions is necessary. These projects had a violation rate of 5 and 10 percent, respectively (8, 9). These two HOV projects also permitted carpools in the HOV lane, whereas the other two HOV projects did not.
- o A study of curbside concurrent flow lanes, lanes that were taken away from general use in Dallas (Tex), found that the bus priority lanes did not adversely affect the operation of non-priority vehicles (12).

Detailed accident data have been collected for four projects (8). Pertinent data pertaining to accident rates, are documented in Table 3.

Miller et al. (8) tentatively identifies contributing causes to the wide range in accident rates associated with median HOV lanes. As volume in the HOV lane increases, general motorists may become more aware of the presence of the priority lane, resulting in a lower HOV lane accident rate. There may also be a direct relationship between the restriction of crossing movements and vehicular safety in the HOV lane. The project with the lowest bus accident rate -- U.S. 1/South Dixie Highway -- prohibits left turns from the facility.

In addition to the common accident types (rear end, sideswipe, and right-angle) experienced on facilities with median HOV lanes, curb lane projects encounter additional accidents involving parked vehicles and pedestrians. These types of accidents accounted for 14% and 2% of total facility accidents, respectively (8).

TABLE 3 - Operating and Accident Characteristics of Selected Arterial Street Concurrent Flow Lane Projects

VARIABLE	UNIT						
Project Location		WASHINGTON, D.C. PROJECT (a)	KALANIANAOLE HIGHWAY HAWAII	US 1/SOUTH DIXIE HIGHWAY, MIAMI, FLORIDA (b)		NW 7TH AVENUE, MIAMI, FLORIDA	
HOV Application		BUS-ONLY	BUS/2ppv	NONE	BUS/2ppv CARPOOL	NONE	BUS-ONLY
Duration		1976	7/74-12/76	8/74-1/75	4/76-3/77	1/74-8/74	1/75-3/76
Location of HOV Lane		Curb	Median	None	Median	None	Median
Critical Peak Period		6:30-9:30AM	6-8AM	7-9AM:4-6PM	7-9AM:4-6PM	7-9AM	7-9AM
Length of HOV Lane	Miles	3.6	0.5	-	5.5	-	2.7
Total Peak Directional Lanes	Number	4	3	3	3	2	3
Number of HOV Lanes	Number	1	1	-	1	-	1
Volume - All Lanes	Vehicles	4,352	5,538	10,664	11,709	1,389	1,610
Volume - HOV Lanes	Vehicles	141	1,138	-	2,834	-	23
Volume - HOV Lanes (bus only)	Vehicles	141	18	-	51	-	23
HOV Lanes/Total Volume	%	3.2	20.5	-	24	-	1.4
Auto Occupancy - All Lanes	ppv	1.6	1.7	1.3	1.2	1.2	1.3
Auto Occupancy - HOV Lanes	ppv	-	3.3	-	2.36	-	-
Person Throughput - All Lanes	Persons	13,121	10,390	13,330	16,232	1,722	2,698
Person Throughput - HOV Lanes	Persons	6,438	4,400	-	6,716	-	667
HOV Lanes/Total Throughput	%	49.1	42.3	-	41.4	-	24.7
Speed - General Lanes	MPH	24	17.4	19.4	18.5	24.4	26.9
Speed - HOV Lanes	MPH	10-13	22.9	-	25.7	-	25.7
Travel Time - General Lanes	Minutes	9	1.7	17.9	17.8	6.5	5.9
Travel Time - HOV Lanes	Minutes	16-22	1.3	-	12.8	-	6.2
Violation Rate	%	-	10.0	-	5.0	-	-
Accidents AM Peak	Number	36	19	70	110	8	7
Accident Rate AM Peak	Acc/MVM	2.8	4.6ns	5.2	8.3**	11.6	4.5ns
	Acc/MPM	1.0	2.5	3.7	5.2*	9.4	3.5ns
Accident PM Peak		99	-	123	166	8	9
Accident Rate PM Peak	Acc/MVM	12.7	-	9.2	12.7**	8.5	5.1ns
Control Accident Rate	Acc/MVM	-	2.2	8.0	7.5	8.0	7.5

(a) Data represents Connecticut Avenue
(b) Before data are for three peak periods (6-9 AM and 4-7 PM) that is reduced to two hour peak periods by assuming uniform hourly rates
ns = Not significant compared to before condition
ppv = Persons per vehicle
* = 95% level of significance for difference from before condition
** = 99% level of significance for difference from before condition

SOURCE: Reference 8

CONTRAFLOW ARTERIAL LANE

A contraflow lane is an arterial street lane designated for use by HOV traveling opposite to the normal direction of traffic. A representative cross section of such a facility (Ponce De Leon Avenue, San Juan, Puerto Rico) is depicted in Figure 3.

The contraflow lane can be either a median lane on a divided highway or a curb lane on a one-way street. A contraflow curb lane on a one-way facility is generally used by local buses making frequent stops. A lane on the left side of the median of a divided street associated with express bus service is intended to provide a time advantage for HOV. The median contraflow lane operates during peak periods, while the curb lane may be either a peak period or an all-day operation. Plastic poles are sometimes used to separate the contraflow lane from the general traffic lane.

Operational and Accident Data

The safety aspects of four arterial street contraflow lane projects were evaluated in detail (8). Data for the four projects are summarized in Table 4 (8). This table presents total facility accident rates.

A wide range in accident rates exists for the contraflow lane projects on arterial streets. Accident rates during the afternoon peak exceed morning peak values. On each project, the bus accident rate during the first year of contraflow lane operation was several times greater than the control accident rate (citywide bus accident rate). However, after 5 years of contraflow lane operation on Ponce de Leon/Fernandez Juncos Avenues, the bus accident rate was less than one-half of the citywide bus accident rate

With the establishment of the contraflow project, the total facility accident rate increased on all but the Kalanianaole project (Table 4). Also, as based on vehicle-miles of travel, the increasing trend of the total facility accident rates on the HOV project is opposite to the trend of the control bases, which experienced decreasing rates during the same periods.

One project, the N.W. 7th Avenue reversible lane, traversed two different geometric sections (9). One section permitted left turns from left turn lanes at signalized intersections and experienced a total facility accident rate of 28 accidents/MVM. The other section completely prohibited left turns and experienced a total facility accident rate of 3.2 accidents/MVM.

Data indicate that the percentage of total facility accident rates that are injury producing has declined with the introduction of contraflow lanes. While left turn cutoff

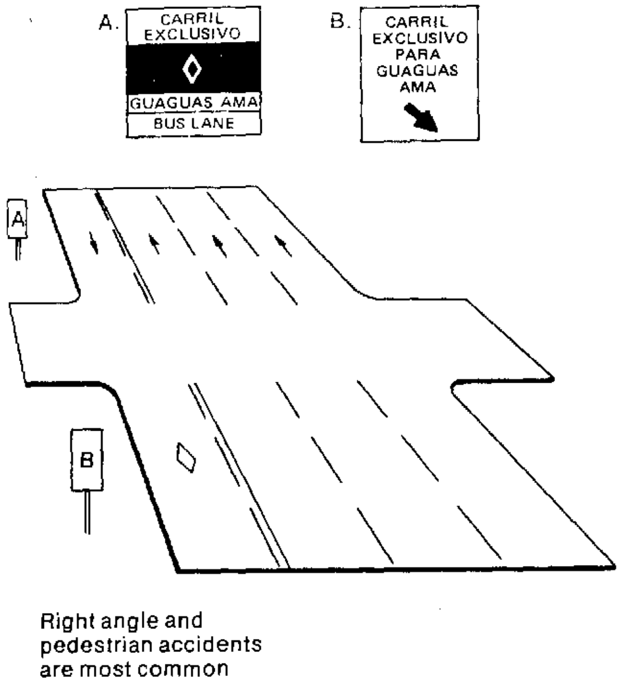


Figure 3. Contraflow Lane on One-way Street

SOURCE: Reference 8

accidents predominate on median contraflow lane projects, right-angle and pedestrian accidents were mostly prevalent on the curb contraflow lane projects. Other data suggest that, as the number of vehicles using the contraflow lane increases, the HOV lane accident rate decreases.

Accident data for the following three studies collected by Bautz (13) are presented in Table 5.

- In Louisville, Ky., it was reported that two accidents occurred in the first 2 weeks of operation. Only two more occurred during the next 15 months.
- In Indianapolis, Ind., there was no indication as to why the accident rate fell sharply in 1973. Virtually all of the accidents involved cars pulling out from side streets without looking to the left and running into the sides of buses.
- In Seattle, Wash., most accidents were caused by drivers exiting parking lots and garages and failing to look both ways.

Also, the accident rates on all projects were higher during the early stages of operation. This suggests that some adjustment period exists while motorists become familiar with the lane operation.

TABLE 4 - Operating and Accident Characteristics of Selected Arterial Street Contraflow Lane Projects

VARIABLE	UNIT										
Project Location		U.S. 1/SOUTH DIXIE HIGHWAY, MIAMI, FLORIDA (a)		KALANIANAOLE HIGHWAY HAWAII		N.W. 7TH AVENUE, MIAMI, FLORIDA A.M. Southbound Only		N.W. 7TH AVENUE, MIAMI, FLORIDA P.M. Northbound Only		PONCE DE LEON AVENUE, SAN JUAN, PUERTO RICO	
HOV Application		NONE	BUS-ONLY	BUS-ONLY	BUS/3ppv CARPOOL	NONE	BUS-ONLY	NONE	BUS-ONLY	BUS-ONLY	
Duration		7/73-6/74	7/74-6/75	8/73-9/75	9/75-12/76	8/74-1/75	1/75-3/76	8/74-1/75	1/75-3/76	1/76-10/76	
Location of HOV Lane		None	Median	Median	Median	None	Reversible	None	Reversible	Curb	
Critical Peak Period		7-9AM/4-6PM	7-9AM/4-6PM	6-8AM	6-8AM	7-9AM	7-9AM	4-6PM	4-6PM	7-9AM	
Length of HOV Lane	Miles	-	5.5	1.9	1.9	-	7.3	-	7.3	13.6	
Total Peak Directional Lanes	Number	3	4	3	3	2	3	2	3	3-4	
Number of HOV Lanes	Number	-	2	1	1	-	1	-	1	1	
Volume - All Lanes	Vehicles	14,674	14,330	3,883	4,756	1,461	1,300	1,825	1,569	5,574	
Volume - HOV Lanes	Vehicles	-	60	15	990	-	23	-	21	129	
Volume - HOV Lanes (bus only)	Vehicles	-	60	15	16	-	23	-	21	129	
HOV Lanes/Total Volume	%	-	0.4	0.4	20.8	-	1.8	-	1.3	2.3	
Auto Occupancy - All Lanes	ppv	1.4	1.6	1.7	1.9	1.3	1.3	1.5	1.4	1.5	
Auto Occupancy - HOV Lanes	ppv	-	-	-	3.3	-	-	-	-	-	
Person Throughput - All Lanes	Persons	20,250	22,640	7,410	10,070	1,895	2,413	2,641	2,900	13,749	
Person Throughput - HOV Lanes	Persons	-	1,903	680	3,930	-	748	-	710	5,798	
HOV Lanes/Total Throughput	%	-	8.4	9.2	39.0	-	31.0	-	24.5	42.1	
Speed - General Lanes	MPH	19.4	16.9	14.1	17.3	21.0	29.0	19.8	25.0	-	
Speed - HOV Lanes	MPH	-	36.7	-	22.9	-	31.7	-	28.8	12.1	
Travel Time - General Lanes	Minutes	17.0	19.5	8.1	6.6	20.9	15.1	22.1	17.5	-	
Travel Time - HOV Lanes	Minutes	-	9.0	-	5.0	-	13.8	-	15.2	67.4	
Accidents AM Peak		70	117	27	14	22	35	-	-	965	
Accident Rate AM Peak	Acc/MVM	5.2	8.8**	1.7ns	1.3ns	10.8	9.5ns	-	-	9.2**	
Accident PM Peak		123	202	-	-	-	-	23	65	-	
Accident Rate PM Peak	Acc/MVM	9.2	15.4**	-	-	-	-	9.9	14.8ns	-	
Control Accident Rate	Acc/MVM	8.0	7.5	2.3	2.2	8.0	7.5	8.0	7.5	10.1	

(a) This facility also had a concurrent flow carpool lane.
ns = Not significant
** = 99% level of significance

SOURCE: Reference 8

TABLE 5 - Contraflow Arterial Operational and Accident Data

LOCATION	LOUISVILLE, KY	INDIANAPOLIS, IN COLLEGE AVENUE	SEATTLE, WA 5TH AVENUE
Type HOV Lane	Curb on Oneway	Curb on Oneway	Curb on Oneway
Length	4 Miles	2.7 Miles	3 Blocks
Time	6AM-9AM	24 Hours	24 Hours
Period	10/71-2/73	1972-1973	1970-74
No. Accidents	4	18('72), 8('73)	12
No. Injuries	0	13('72), 2('73)	-
No. Fatalities	0	0	-
Bus Miles/Yr.	21,000	115,000	35,880
Acc. Rate/Mbm*	194	156('72), 69('73)	334

* Million bus miles
\- Not available

SOURCE: Reference 13

SEPARATE STREET HOV FACILITY

These facilities, commonly used only by transit vehicles, are frequently referred to as transitways. The most common type of separate HOV facility functions as a transit collection/distribution route and is generally created by restricting operations on a through street. Such facilities are generally located in the downtown area and are often associated with some type of pedestrian mall. These are frequently two-lane, undivided streets. Examples of such facilities can be found in Minneapolis, Minn., (Nicollet Mall); Portland, Oreg., (Portland Mall); Chicago, Ill., (Halsted and 63rd Street); and Philadelphia, Pa., (Chestnut Street). Examples in other countries include Oxford Street in London, England, and Granville Street in Vancouver, British Columbia.

Operational and Accident Data

The Nicollet Mall is an eight block treatment along Nicollet Avenue between Washington Street and 10th Street in downtown Minneapolis. Transit vehicles and taxicabs operate on a 24-foot, two-lane serpentine roadway. The remainder of the 80-foot right-of-way is a pedestrian mall (14).

Accident data have not been reported for the U.S. projects, since safety has not been considered a problem with these projects.

Oxford Street (15) in London was closed to all vehicles except buses and taxis in November 1972. Signing and enforcement were difficult since there was no nearby alternative traffic route, and many drivers had to divert at the terminus of the priority treatment.

- There was an initial violation rate of 30 percent, which dropped to 10 percent after 3 months of operation.
- The total number of injury accidents rose from 145 to 154 per year.
- The fatal and serious accidents fell from 22 to 14 per year on the busway itself.
- Injury accidents rose from 810 to 904 per year in the corridor.
- Fatal and serious accidents declined from 106 to 69.

Granville Street in downtown Vancouver was transformed into a six-block pedestrian mall (15). Electric trolley buses are used to provide transit service.

- There has been about one bus/pedestrian accident per month.
- Pedestrians tend to "jaywalk" across the narrowed street lanes.
- The quietness of the electric trolley buses give pedestrians little warning of their approach.

FREEWAY PRIORITY ENTRY FOR HOV

Preferential entry facilities for HOV can consist of either bypass ramps at metered freeway entry points or ramps for the exclusive use of HOV. A representative ramp meter bypass facility, as developed in Los Angeles, is shown in Figure 4.

Operational and Accident Data

Peak-period operational data, collected as part of the Miller et al. study (8), are presented in Table 6.

Accident data for the Los Angeles ramp bypass projects are presented in Table 7. The total number of accidents increased with the implementation of the priority measure. Limited data obtained on the Santa Monica freeway in Los Angeles suggest that the ramp meter bypass lanes for HOV have no significant effect on the safety of the freeway main lanes. There was a distinct increase in sideswipe accidents on the ramps.

In Seattle, the exclusive ramp priority measure did not result in accident characteristics that would have been caused by the HOV measure.

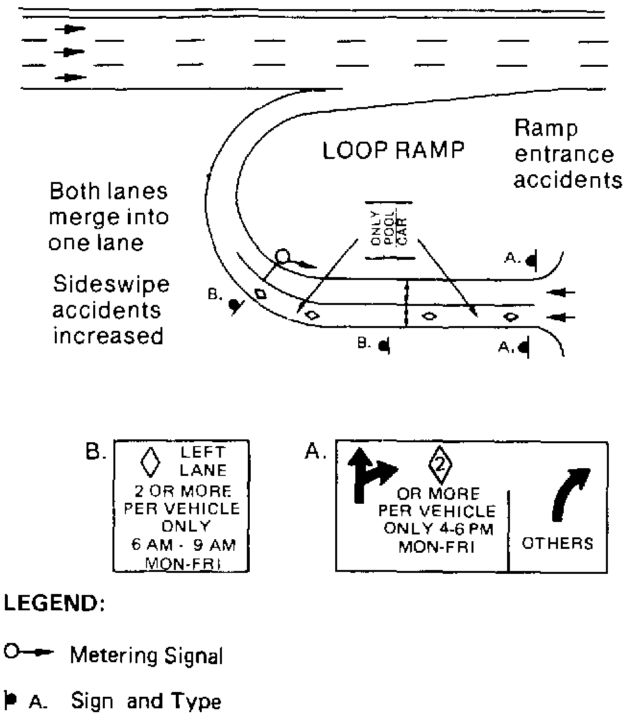


Figure 4. Freeway Ramp Meter Bypass

SOURCE: Reference 8

TABLE 6 - Operating Characteristics of Selected Priority Entry Projects

PROJECT LOCATION		LA FREEWAYS(a)	I-5 EXCLUSIVE RAMP, SEATTLE, WA.			
HOV APPLICATION		BUS/2 ppv CARPOOL	BEFORE	BUS-ONLY	BUS-ONLY	BUS-3 ppv CARPOOL(b)
Critical Peak Period		6-9AM,3-6:30PM	24 Hour	24 Hour	7-8AM	7-8AM
VARIABLE	UNIT					
Length of HOV Lane	Miles	-	0.22	0.22	0.22	0.22
Total Peak Directional Lanes	Lanes	2	1	1	1	1
Number of HOV Lanes	Lanes	1	1	1	1	1
Volume - All Lanes	Vehicles	1,409	4,650	-	-	-
Volume - HOV Lanes	Vehicles	509	-	396	70	106
Volume - HOV Lanes(bus only)	Vehicles	14	-	370	65	56
TOTAL VOLUME	%	36.1	-	-	-	-
Auto Occupancy - All Lanes	ppv	1.4	-	-	-	-
Auto Occupancy - HOV Lanes	ppv	2.1	-	-	-	-
Person Throughput - All Lanes	Persons	2,821	7,250	-	-	-
Person Throughput - HOV Lanes	Persons	1,534	-	11,431	2,086	1,954
HOV Lanes/Total Throughput	%	54.4	-	-	-	-
Travel Time Savings (Average)	Minutes	2.1	-	-	-	-
Travel Time Savings (Maximum)	Minutes	5.3	-	-	-	-
Violation Rate	%	38.3	-	3.5	7.0	4.7

(a) Data are the average of 21 ramps on Santa Monica, Golden State and Harbor Freeways
(b) Data was compiled one month after inclusion of carpools to the HOV strategy
ppv Persons per vehicle

SOURCE: Reference 8

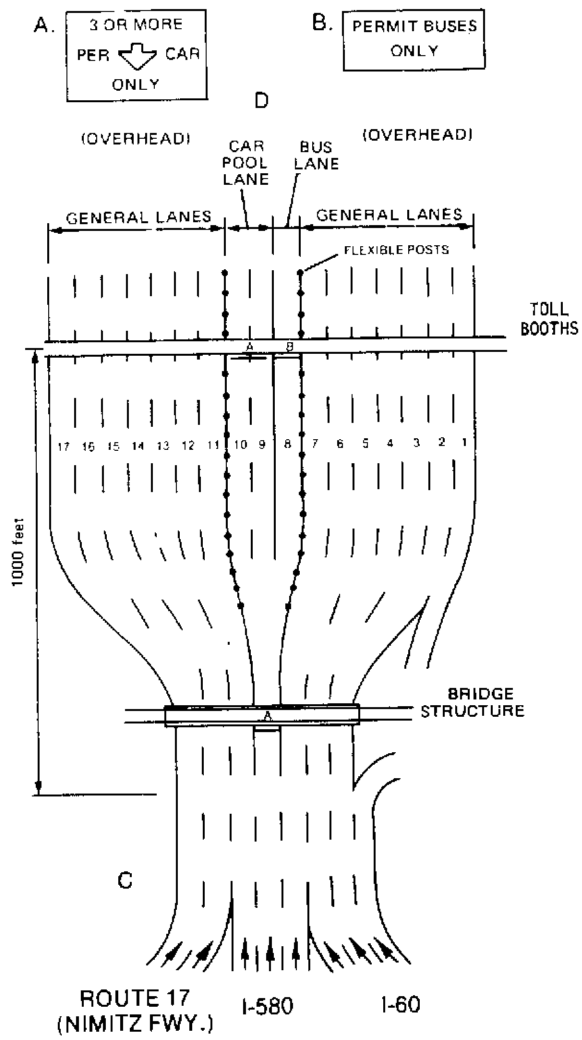
TABLE 7 - Peak Period Facility Accident Rates for Los Angeles Ramp Meter Bypass Projects

FREEWAY	RAMP	PEAK PERIOD	BEFORE RMB ACC.	BEFORE RMB ACC./YEAR	AFTER RMB ACC.	AFTER RMB ACC./YEAR	AFTER RMB VEHICLES (MILLION)	AFTER RMB ACC. RATE/MILLION VEHICLES
Santa Monica Freeway	Hoover St.	PM	1	1	3	1.7	.59	5.1
	Vermont Ave.	PM	0	0	0	0	.71	0
	Western Ave.	PM	2	2	5	2.8	.49	10.2
	Crenshaw Blvd.	PM	0	0	2	0.6	.52	3.8
	Fairfax Ave.	PM	0	0	1	0.6	.45	2.2
	Vermont Ave.	AM	0	0	0	0	.35	0
	Western Ave.	AM	0	0	1	0.9	.43	2.3
	Crenshaw Blvd.	AM	1	0.5	0	0	.29	0
	Venice Blvd.	AM	0	0	2	1.8	.25	8
	Robertson Blvd.	AM	0	0	0	0	.25	0
	Manning Ave.	AM	0	0	0	0	.49	0
	Bundy Dr.	AM	0	0	2	1.1	1.03	1.9
	Cloverfield Blvd.	AM	0	0	0	0	.20	0
Harbor Freeway	Vernon Ave.	AM	0	0	0	0	.12	0
	Florence Ave.	AM	0	0	0	0	.14	0
	EB Manchester	AM	0	0	1	2.4	.09	11.1
	WB Artesia	AM	0	0	0	0	.18	0
	EB Artesia	AM	0	0	0	0	.07	0
Golden State Freeway	Stadium Way	PM	0	0	0	0	.24	0
	EB Los Feliz	PM	0	0	1	0.7	.91	1.1
	EB Western Ave.	AM	0	0	1	0.7	.27	3.7
TOTAL	All Ramps		4	3.5	19	13.3	8.07	2.35

SOURCE: Reference 8
RMB - Ramp Meter Bypass

TOLL PLAZA HOV LANES

Since the capacity of toll plazas is less than the roadway capacity, the plazas create bottlenecks. This priority measure consists of designated freeway approach lanes that allow HOV's to bypass the queue that forms as a result of the plaza bottleneck. Representative approach lanes for the San Francisco-Oakland Bay Bridge are depicted in Figure 5. The HOV lanes are usually taken away from general traffic rather than being added lanes.



C— Accidents in weaving areas as HOV move to restricted lanes
D— Accidents when toll plaza lanes merge into 5 bridge lanes

Figure 5. Toll Plaza HOV Treatment

SOURCE: Reference 8

Operational and Accident Data

Peak period operational data and detailed accident data (8) related to the priority measure on the San Francisco-Oakland Bay Bridge are presented in Tables 8 and 9, respectively. Peak period accidents increased with each subsequent stage until metering was introduced. Daily accident rates during HOV operating conditions were significantly higher than control accident rates due to the complexity of operation in the toll plaza area. The HOV lanes split what had formerly been a homogeneous stop-and-go queue into two sections. That resulted in the queue in the general lanes being extended further upstream and also created a speed differential between the HOV lanes and the general lanes.

CONCURRENT FLOW FREEWAY HOV LANE

A concurrent flow lane on a freeway is a lane, usually the inside lane in the peak travel direction, not physically separated from the other general traffic lanes, and designated for exclusive HOV use. Access to these lanes is generally continuous. A representative cross section of this type of priority measure (Interstate 95 in Miami, Fla.) is shown in Figure 6. Concurrent flow lane projects can be created by either adding a new lane to serve as the HOV lane or by taking away an existing lane from general traffic to function as an HOV lane. The former approach has resulted in greater public acceptance of the measure and less congestion in the general traffic lanes. The lack of physical separation between HOV and general lanes associated with this measure generally causes operational and safety problems.

Operational Data

Peak period operational data, the time during which the lane is designated for use by HOV, for four concurrent flow lane projects are shown in Table 10 (8, 16-25). Comparable operational data for these and other projects not shown in Table 10 are presented below.

1. Peak period Volume

   - In Boston, Mass. on the Southeast Expressway, peak period vehicular throughput decreased by 14 percent when an existing lane was reserved for buses and carpools of 3 persons per vehicle (ppv) or more (26).

   - On the Banfield Freeway in Portland, Oreg., where an HOV lane was added by removing a shoulder and reducing lane width in the normal traffic lanes, total volume increased 3 percent during the peak period (20).

- o When HOV lanes were added to the existing lanes on Route 101 in Marin County, Calif., total peak period volume increased by 4 percent (22).

2. Person throughput

- o The total person throughput initially declined 17 percent on the Santa Monica Freeway where the HOV lane was taken from general use (17, 18).
- o On I-95 (Miami, Fla.), where the HOV lane was added to the facility, the total person throughput increased by 22 percent for buses and 3 ppv carpools and 50 percent when the carpool limitation was reduced to 2 ppv (23, 24).
- o Route 101 showed a 0.5 percent increase in throughput when only buses were allowed in the HOV lane, which increased slightly to 3.8 percent with the addition of 3 ppv carpools (22).
- o On the Banfield Freeway, person throughput increased by 9 percent for the peak period (20).

TABLE 8 - Operating Characteristics of the Toll Plaza HOV Lanes on the San Francisco-Oakland Bay Bridge

VARIABLE	UNIT	Before	Bus-Only[a]	Bus/3 ppv Carpool[a]	Bus/3 ppv Carpool[b]
Critical Peak Period	-	6-9AM	6-9AM	6-9AM	6-9AM
Length of HOV Lane	Miles	-	1.1	1.1	1.1
Total Peak Directional Lanes	Lanes	17	17	17	17
Number of HOV Lanes	Lanes	-	1	3	3
Volume - All Lanes	Vehicles	22,820	23,001	22,694	22,346
Volume - HOV Lanes	Vehicles	-	767	2,827	3,338
Volume - HOV Lanes (bus only)	Vehicles	-	542	509	406
HOV Lanes/Total Volume	%	-	3.3	12.5	14.9
Auto Occupancy - All Lanes	PPV.	na	1.31	1.42	1.50
Auto Occupancy - HOV Lanes	PPV	-	1.31[c]	3.23	3.29
Person Throughput - All Lanes	Persons	na	49,069	50,914	46,908
Person Throughput - HOV Lanes	Persons	-	19,942	26,875	23,718
HOV Lanes/Total Throughput	%	-	40.6	52.8	50.6
Speed - General Lanes[d]	MPH	na	15.1	28.6	na
Speed - HOV Lanes[d]	MPH	-	31.5	38.2	na
Travel Time - General Lanes[d]	Minutes	na	15.5	8.2	na
Travel Time - HOV Lanes[d]	Minutes	-	7.4	6.1	na
Violation Rate	%	-	29.3	7.1	5.6

a. HOV priority at toll plaza
b. HOV priority at toll plaza and metering station
c. These are violators
d. Speed and travel time based on 3.9 mile (6.3 km) section from junction of I-80 and I-580.

PPV - persons per vehicle
na - data not available

NOTE: Operating conditions are as follows:
1) Before Stage -- general operations prior to any HOV priority treatment.
2) Bus-Only Stage -- one lane (No. 8) was reserved for buses (lane numbers shown in Figure 5).
3) Bus/Carpool Stage -- in addition to the bus lane, two carpool lanes (nos. 9 & 10) were reserved for carpools of three or more persons (lane numbers shown in Figure 5).
4) Bus/Carpool and Metering Stage -- the HOV lanes are allowed non-stop passage through the metering station, which was installed to control the volume and merging as the facility narrows from 17 to five lanes.

SOURCE: Reference 8

3. Peak Period Speed

- The peak period speed in the HOV lanes was nearly 50 mph on all projects except the Southeast Expressway in Boston which had a speed of 38 mph and the Moanalua Freeway in Honolulu which had a speed of 12 mph.
- Due to the congested operation in the general lanes, all projects except the Moanalua Freeway had a speed differential of from 9 to 14 mph.

4. Violation Rate

- The violation rate (percentage of the HOV lane traffic that does not qualify to be in that lane) ranged from 7 percent on the Moanalua Freeway project to 61 percent on the I-95 project to 80 percent on the Southeast Expressway in Boston (voluntary compliance).

Accident Data

Detailed accident data have been compiled for four concurrent flow lane projects (8) and are presented in Table 10.

Of the peak period facility accident data for these projects, only one, I-95 in Miami, experienced a decrease in accident rates from the "before" condition, while the others all experienced increases. The Southeast Expressway in Boston experienced a slight, but not statistically significant, increase in accidents (26, 27).

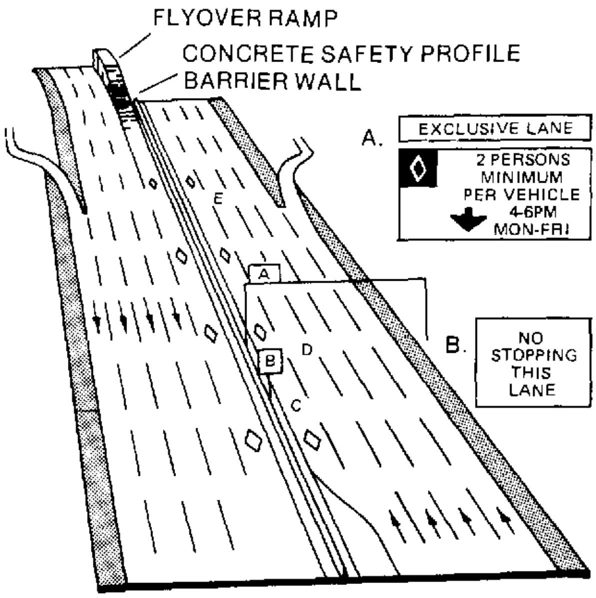
TABLE 9 - Peak-Period Facility Accident Data, San Francisco-Oakland Bay Bridge Toll Plaza Lanes

SECTION/HOV APPLICATION	AM PEAK PERIOD ACCIDENT DATA			
	TIME PERIOD	Number of Accidents	Accident Rate (a) (acc/mvm)	Accident Rate (a) (acc/mpm)
Total Length				
o Before HOV	1/70 - 4/70	9	1.8	0.9
o Bus Only	5/70 - 12/71	74	2.7	1.3 ns
o Bus/Carpool	1/72 - 2/74	146	4.0 *	1.8 *
o Bus/Carpool with Metering	3/74 - 12/76	83	2.3 ns	1.1 ns
Upstream of Toll Plaza				
o Before HOV	1/70 - 4/70	5	3.5	1.7
o Bus-Only	5/70 - 12/71	26	3.3 ns	1.6 ns
o Bus/Carpool	1/72 - 2/74	53	5.1 ns	2.3 ns
o Bus/Carpool with Metering	3/74 - 12/76	49	4.7 ns	2.3 ns
Downstream of Toll Plaza				
o Before HOV	1/70 - 4/70	3	2.3	1.1
o Bus-Only	5/70 - 12/71	19	2.7 ns	1.3 ns
o Bus/Carpool	1/72 - 2/74	41	4.3 ns	1.9 ns
o Bus/Carpool with Metering	3/74 - 12/76	20	2.1 ns	1.0 ns
Bridge Section				
o Before HOV	1/70 - 4/70	1	0.4	0.2
o Bus-Only	5/70 - 12/71	29	2.4 *	1.1 *
o Bus/Carpool	1/72 - 2/74	52	3.2 *	1.4 *
o Bus/Carpool with Metering	3/74 - 12/76	14	0.9 ns	0.4 ns

(a) Statistical significance of accident rates compared to before HOV condition
ns indicates difference is not significant
* indicates a 99% level of significance

acc - accidents
mvm - million vehicle miles
mpm - million person miles

SOURCE: Reference 8



C— Accidents from speed differential and merge into lane
D— Accidents in adjacent lane due to viewing HOV lane and/or shock waves
E— Accidents leaving HOV lane

Figure 6. Freeway Concurrent Flow Lane

SOURCE: Reference 8

The creation of speed differentials between different traffic lanes and actions such as the elimination of emergency shoulders can influence accident severity. Accidents involving personal injury increased significantly during the peak period on the Santa Monica Freeway and U.S. 101 projects. On a daily basis, the Santa Monica Freeway project showed a statistically significant increase in its peak period personal injury accident rate. The HOV lanes added to I-95 had a statistically significant decrease in daily accident rates.

Analysis of accident data on the Santa Monica Freeway showed that the relative percentage of accidents associated with vehicles which were slowing, stopping or standing still prior to collision increased significantly (16). An increase in the percentage of rear end accidents was also observed on the Banfield Freeway (20). This suggests an increase of shock wave related accidents. The same percentage trend was observed on Route 101 despite the fact that congestion was relieved through increased capacity and reduced vehicular demand. The only apparent explanation for this occurrence is that weaving to and from the HOV lane produced shock waves which led to rear end accidents.

On U.S. 101, in the San Francisco area, the accident picture remained about the same for a few years following its opening. During the evening bus-carpool lane operating hours, accidents in the adjacent general traffic lanes have increased from 37 during the 6 months prior to the opening of the bus lanes in December 1974 to 79, 77, and 76 during the same periods in 1975, 1976, and 1977. Less than 10% of the peak period accidents have involved vehicles in the HOV lane. During the morning operating hours, the number of accidents in the adjacent general traffic lanes are the same as they were prior to the HOV lane operation (24).

On Route 280 another concurrent flow bus-carpool lane in the San Francisco area, which provides a southbound HOV lane for 2 miles to bypass a congested peak period section, the lane operates 24 hours a day, 7 days a week. During the evening peak the lane averages about 150 carpools, 15 buses and 80 violators. Daily traffic using the lane is about 600 vehicles. Except for the first year of operation, the accident rate has been about the same as it was before the HOV lane was implemented. There has been only one accident involving vehicles in the express lane during 3 years of operation (24).

A different trend occurred on I-95 in Miami (25). The relative frequency of accidents involving stopped traffic declined. The relative frequency of sideswipe accidents increased. The percentage of rear end accidents declined. This combination suggests there was less of a problem with accidents related to congestion. Thus, the I-95 HOV lane appears to have had a higher relative frequency of accidents related to gaining access to, or egress from, the HOV lane by weaving across the general lanes than the other projects in which speed or congestion related problems predominated.

CONTRAFLOW FREEWAY HOV LANES

A contraflow lane is a freeway lane (commonly the inside lane in the offpeak direction of travel) designated for exclusive use by HOV traveling in the peak direction. This technique assumes that unused capacity exists in the offpeak travel direction. A representative cross section of this type of priority measure (I-45 North in Houston, Tex.) is shown in Figure 7.

In some instances, only buses are allowed to use the contraflow lane. The project on the Long Island Expressway, N.Y., also allows occupied taxis to use the lane. The I-45, Houston project allows registered vanpools, airport shuttlebuses, and intercity buses to use the priority lane and is the only contraflow lane operated during both peak periods.

TABLE 10 - Operating and Accident Characteristics of Selected Concurrent Flow Lane Projects

VARIABLE	UNIT													
Location		Moanalua Freeway (HI)		Santa Monica Freeway (CA)			Route 101 San Francisco (CA)			Interstate 95 Miami (FL)		
HOV Application		Bus/3 ppv Carpool(a)	Bus/3 ppv Carpool(b)	None	Bus/3 ppv Carpool	After Termination	None	Bus-Only	Bus/3 ppv Carpool	None	Bus/3ppv Carpool	Bus/2ppv Carpool
Time Period	--	1975	1976	to 3/76	3/76-7/76	8/76-12-76	1974	12/74-3/76	6/76-12/76	5/74-8/74	3/76-1/77	1/77-5/77
Critical Peak Period	--	6 - 8 AM	6 - 8AM	3 - 7 PM	3 - 7 PM	3 - 7 PM	4 - 7 PM	4 - 7 PM	4 - 7 PM	4 - 6 PM	4 - 6 PM	4 - 6 PM
Length of HOV Lane	Miles	2.7	2.7		12.5			3.7	3.7		6.7	6.7
Total Peak Directional Lanes	Lanes	3	3	4	4	4	3	4	4	3-4	4-5	4-5
Number of HOV Lanes	Lanes	1	1		1			1	1		1	1
Volume - All Lanes	Vehicles	7,200	6,425	28,250	21,158	28,013	13,600	13,137	13,089	11,355	12,825	15,290
Volume - HOV Lanes	Vehicles	1,220	1,850		1,853			191	647		618	2,057
Volume - HOV Lanes (bus only)	Vehicles	6	11		64			148	150		23	23
HOV Lanes/Total Volume	%	16.9	28.8		8.8			1.5	4.9		4.8	13.5
Auto Occupancy - All Lanes	ppv	1.7	1.9	1.3	1.3	1.3	1.3	1.3	1.4	1.3	1.4	1.4
Auto Occupancy - HOV Lanes	ppv	3.2	3.2		3.3			2.2	3		2.2	1.8
Person Throughput - All Lanes	Persons	12,230	12,305	35,878	29,781	36,977	24,439	24,567	25,365	14,875	18,221	22,338
Person Throughput - HOV Lanes	Persons	3,920	5,980		7,117			5,719	7,172		1,981	4,347
HOV Lanes/Total Throughput	%	32.1	48.6		23.9			23.3	28.3		10.9	19.5
Speed - General Lanes	MPH		8.9	42.1	36.0	46.3	34.1	43.3	47.6	29.6	35.6	41.6
Speed - HOV Lanes	MPH		11.5		49.6			53.4	53.4		50.0	50.4
Travel Time - General Lanes	Minutes		13.9	17.8	20.8	16.2	6.5	5.1	4.7	13.5	11.3	9.6
Travel Time - HOV Lanes	Minutes		12.9		15.1			4.2	4.2		8.0	8.0
Violation Rate	%	18.8	6.8		15.9		4.2	9.6	12.8	5.1	4.7	2.4
Accidents	Number	11	29	363	197	497	163	158	89	32	92	27
Accident Rate	Acc/MVM	2.2	6.3*	2.6	9.5**	1.9*	4.2	9.6**	12.8**	5.1	4.7ns	2.4*
Accident Rate	Acc/MPM	1.3	3.4	2.1	6.7**	1.5*	2.3	5.2**	6.6**	3.9	3.3ns	1.6**

(a) One month after opening of project (November, 1974)
(b) Two years after opening of project (October, 1976)
* 95% level of significance
** 99% level of significance
ppv Persons per vehicle

Acc/MVM Accidents per million vehicle miles
Acc/MPM Accidents per million person miles

Blank Cell - Data not available

SOURCE: Reference 8

Operational Data

Operational data for four contraflow lane projects are shown in Table 11. Additional data are highlighted below (8, 28, 29).

- o Contraflow lane widths vary. The HOV lane is 12-feet wide on the I-45 lane in Houston, the Southeast Expressway in Boston and on U.S. 101 in Marin County, Calif., 11.5 feet on the Long Island Expressway in New York, and 10.5 feet on I-495 in New Jersey. However, despite heavy bus volumes and substandard geometrics, the I-495 project has not had perceptible operational problems.

- o Delineation of the contraflow lane has been accomplished by using either traffic cones or plastic posts inserted into holes drilled in the pavement. In Boston heavy cones were placed on 80-foot centers. In New Jersey and New York, nipple-type plastic posts are inserted into holes drilled in the pavement. In Marin County, similar posts are placed 50 feet apart. These posts are placed at 40-foot intervals in Houston.

- o In the Marin County and Boston projects, buses merge back through an opening in the median barrier to reenter the normal traffic lanes. On U.S. 101 (Marin County), the crossover facility is a permanent tapered, blocked out guardrail which protects the buses from oncoming traffic in the offpeak roadway and prevents unauthorized crossovers. A 0.9-mile acceleration lane facilitates a smooth merge into the normal lanes. The crossover on the Boston contraflow bus lane was closed daily and no acceleration lane provided. The merge occurred without significant problems since traffic on the normal lanes is usually traveling very slowly during the peak period and signs instruct drivers to yield to buses. The New Jersey and New York facilities terminate at toll plazas.

Accident Data

Accident data are available for the four projects identified in Table 11. Accident rates are typically higher in the offpeak direction of travel than in the peak travel direction. On Route 101, accident rates in the offpeak travel direction increased significantly with the introduction of the contraflow lane. None of the other projects had significant increases in peak period accident rates. Tests of statistical significance were not performed on the Houston data.

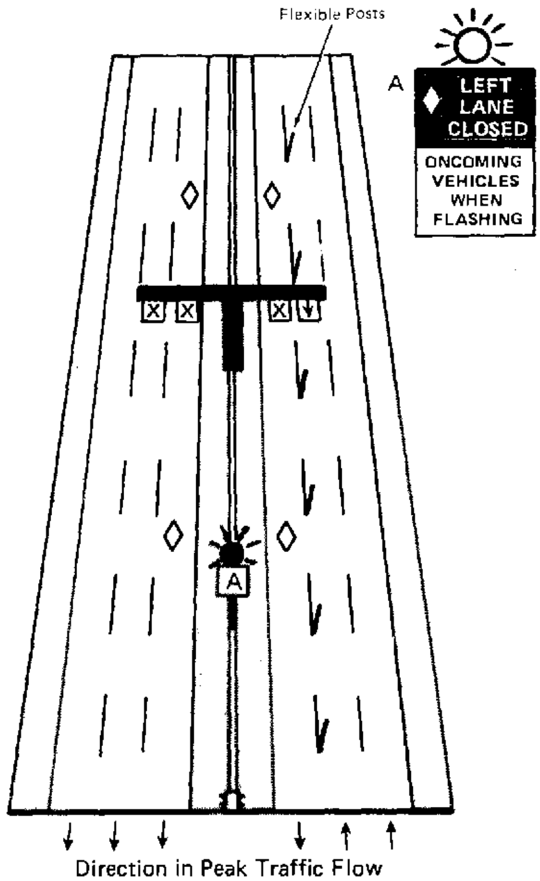


Figure 7. Freeway Contraflow Lane

Note: Accidents occurred on lane next to contraflow because of more congestion.

SOURCE: Reference 28

The introduction of the contraflow lane also increased the daily accident rate on Route 101 by a statistically significant amount. The control accident rate was experiencing a decreasing trend while the Route 101 accident rate was experiencing an increasing trend. On that facility, the contraflow priority measure resulted in a statistically significant increase in peak period, peak direction injury accidents but a decrease (not statistically significant) in injury accidents in the offpeak direction. After 1 year of operations, the accident rates for the peak direction on Route 101 had dropped to pre-contraflow lane levels.

Accident characteristic data show that, on Route 101, the contraflow lane project resulted in an increase in the percentage of accidents occurring in the interior and left lanes in the peak direction. The setup and takedown of the lane can also affect accident rates since the procedures used to set up and take down the lane alter facility operation and can impact facility safety.

TABLE 11 - Operating and Accident Characteristics of Selected Contraflow Lane Projects

VARIABLE	UNIT							
LOCATION		I-495, NJ		Long Island Expressway, NY(a)	Route 101 Marin County, CA		I-45N Houston, TX(e)	
HOV APPLICATION		NONE	BUS-ONLY	BUS-ONLY	NONE	BUS-ONLY	NONE	HOV(f)
Critical Peak Period	--	7-10AM	7-10AM	7-9:45AM	4-7PM	4-7PM	8:30AM	8:30AM
Length of HOV Lane	Miles		2.5	2.0		4.0		9.6
Total Peak Directional Lanes	Lanes	3	4	4	4	5	3	4
Number of HOV Lanes	Lanes		1	1		1		1
Volume - All Lanes	Vehicles	11,747	12,368	9,607	15,392	16,608(b)	12,600	12,600
Volume - HOV Lanes	Vehicles		1,050	307		125		242
Volume - HOV Lanes (bus only)	Vehicles	1,100(c)	1,050	300	120(c)	125		60
HOV Lanes/Total Volume	%		8.5	3.2		0.8		1.9
Auto Occupancy - All Lanes	ppv	1.5	1.6	1.4	1.3	1.3	1.4	1.3
Auto Occupancy - HOV Lanes	ppv							12.3
Person Throughput - All Lanes	Persons	63,260	61,036	23,662	24,348	26,428	17,300	20,100
Person Throughput - HOV Lanes	Persons		44,625	11,107		5,000		3,715
HOV Lanes/Total Throughput	%		73.1	46.9		18.9		18.4
Speed - General Lanes	MPH	10.0	17.2	6.7	24.0	40.0	25.0	25.0
Speed - HOV Lanes	MPH		22.4	34.3		36.9(d)		54.0
Travel Time - General Lanes	Minutes	14.7	8.7	17.9	10.0	6.0	22.6	22.6
Travel Time - HOV Lanes	Minutes		6.7	3.5		6.5		10.4
Peak Period Direction - Acc.	No.		51	42	58	148	46	36
- Rates	Acc/MVM		3.1	2.1	2.2	2.2ns	1.8	1.5
	Acc/MPM		0.8	0.9	1.4	1.2ns		
Off Peak Direction - Acc	No.		32	57	56(e)	194	54	58
- Rates	Acc/MVM		3.6	5.4	2.9	3.8*	2.2	3.0

(a) No before data available
(b) Freeway improvements resulted in increased auto volumes during HOV application.
These data exclude the effects of the concurrent HOV
lane project added later in the north end
(c) Buses in general lanes in period prior to HOV
(d) Lower contraflow lane speed due to uphill grade and improvements in general lanes
(e) HOV lane operates during both peak periods. Only morning data shown in this table
(f) Bus and registered vanpool, airport shuttle bus, intercity bus
PPV - Persons per vehicle
MVM - Million Vehicle Miles
MPM - Million Person Miles
Acc - Accidents
* - Signifcant difference from before condition at 95% confidence level

SOURCE: Reference 8 and 27 (data for I-495 updated to 1980 by Port Authority of N.Y., and N.J.)

The apparent danger of a head-on collision at high speeds is a deterrent to potential violators of the contraflow lane restrictions. Thus the accident rates have generally been low, though the severity index (injuries/accident) tends to be high (14).

One major safety problem concerning contraflow measures involves the need to set up and take down the safety poles used to designate the lane. A second problem involves the reduction in roadway capacity in the offpeak direction of travel that results from designating a contraflow lane. This capacity reduction generally leads to increases in accident rates in the offpeak travel direction.

SEPARATE FREEWAY HOV FACILITY

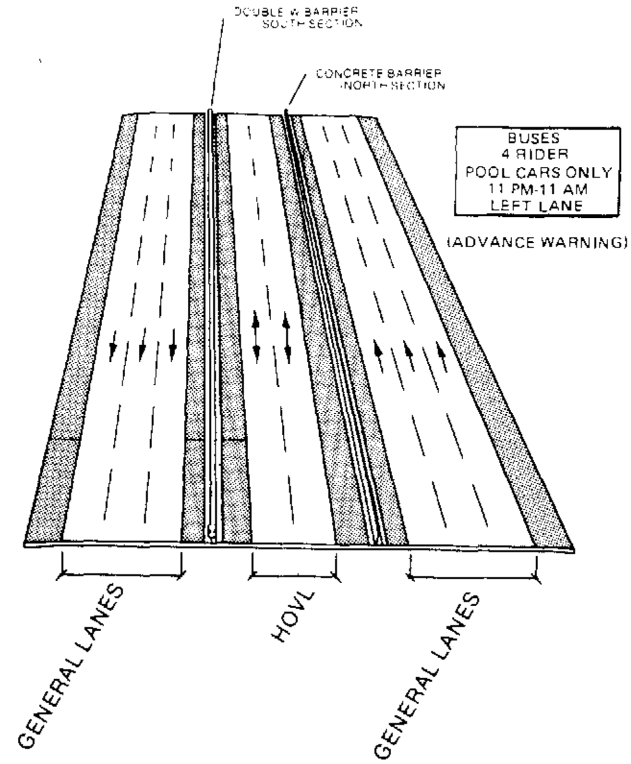
The separate HOV facility is a roadway or lane(s) that is physically separated from the other freeway lanes and is designated for exclusive use by HOV. Access to, and egress from, these facilities are generally possible at only limited locations.

One example of a separate HOV facility (I-395 Shirley Highway near Washington, D.C.) is depicted in Figure 8. A second project, the El Monte Busway on the San Bernardino Freeway in Los Angeles has two distinctly different cross sections. Over a 7-mile section, the HOV lane is separated from the general traffic lanes by a common shoulder, while the remaining 4.2 miles of the HOV priority roadway are off to the side and are completely separated from the general traffic lanes.

Operational and Accident Data

Peak period operational data, the time during which the impact of the facility is greatest, for the Shirley Highway and San Bernardino Freeway projects are shown in Table 12. Detailed accident data have been compiled for both the Shirley Highway and San Bernardino Busway projects as part of the Miller et al. work (8). These data are also presented in Table 12.

Separate HOV facilities generally operate with a high degree of safety. For example, transportation officials from the Virginia Department of Highways and Transportation, overseeing the Shirley Highway operation, do not consider the facility to have an accident problem.



HOVL—High-Occupancy Vehicle Lanes

Figure 8. Separate HOV Facility

SOURCE: Reference 8

On the San Bernardino Freeway (8, 30, 31), there were almost no accidents, violations, or enforcement problems on the busway during bus-only operations. With the addition of carpools, however, a safety problem arose in the section of the HOV lane consisting of the busway in the median of the freeway separated from the normal lanes by a buffer shoulder with flexible posts. Illegal weaving across the buffer shoulder in this area increased the accident rate to that of a typical freeway. In the unseparated access lanes, on either end of the busway, the illegal weaving resulted in accident rates double that of normal freeway operations. This was largely due to violators using the access lane to bypass congested traffic in the normal lanes. There was also an accident problem in the half-mile-long merging lane at the eastern end of the HOV lane. This has been attributed to congestion effects caused by a large number of carpool vehicles entering a freeway which is already at capacity.

TABLE 12 - Operating and Accident Characteristics of Selected Separate HOV Facilities on Freeways

VARIABLE	UNIT						
Project Location		I-395 (VA)	San Bernardino Freeway (CA)				
HOV Application		BUS/4 ppv Carpool (a)	Before HOV (b)		Bus-Only		Bus/3 ppv Carpool
Critical Peak Period	-	6-9:30AM	3-7PM		3-7PM		3-7PM
Length of HOV Lane	Miles	11.5			7.0		7.0
Total Peak Directional Lanes	Lanes	5	4		5		5
Number of HOV Lanes	Lanes	2			1		1
Volume - All Lanes	Vehicles	26,050	28,018		28,018		28,346
Volume - HOV Lanes	Vehicles	4,704			168		906
Volume - HOV Lanes (bus only)	Vehicles	545			168		164
HOV Lanes/Total Volume	%	18.1			0.6		3.2
Auto Occupancy - All Lanes	ppv	1.8	1.3		1.3		1.3
Auto Occupancy - HOV Lanes	ppv	4.6					3.1
Person Throughput - All Lanes	Persons	64,450	40,096		40,096		41,543
Person Throughput - HOV Lanes	Persons	38,263			5,240		7,780
Hov Lanes/Total Throughput	%	58.9			13.1		18.7
Speed - General Lanes	MPH	30.2	35.0		37.0		39.0
Speed - HOV Lanes	MPH	51.0			57.1		57.1
Travel Time - General Lanes	Minutes	22.8	12.0		11.4		10.8
Travel Time - HOV Lanes	Minutes	13.5			7.4		7.4
Violation Rate	%	2.5			0		9.1
			A	B	A	B	B
AM Peak Period							
Accidents	Number	87	30	156	54	79	19
Accident Rate	Acc/MVM	2.0	0.9	(c)	1.0	0.9	
Accident Rate	Acc/MPM	0.6	0.7	(c)	0.7	1.4	
PM Peak Period							
Accident	Number	43	34	90	100	70	13
Accident Rate	Acc/MVM	7.9	0.8	(c)	1.2	1.0	1.3
Accident Rate	Acc/MPM		0.6	(c)	0.8	0.7	0.9
Daily Accident Rate	Acc/MVM	2.3	1.4	(c)	1.2	1.5	1.0

ppv - Persons per vehicle
Acc/MVM - Accidents per million vehicle miles
Acc/MPM - Accidents per million person miles

A. Completely separated section
B. Partially separated section (with buffer lane)

(a) No before data available. Data shown, except for travel time and speed, are 1980 data provided by the Virginia Department Highways of Transportation (updated from reference 8).
(b) No explicit before data were available. Published reports and graphs indicate there was little change in volume or person trips between before HOV and bus-only stages, so that latter data is assumed to apply to both.
(c) Measured vehicle miles are not available

SOURCE: Reference 8

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CHAPTER 9 - ON-STREET PARKING

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INTRODUCTION

Conflicts on roadways between parked vehicles, those parking and unparking, and other road users result in a substantial number of accidents. A comprehensive review of research publications dealing with on-street (curb) parking was conducted to identify studies that relate traffic control and roadway elements to on-street parking safety. This chapter presents a synthesis of pertinent research in this subject area.

SUMMARY OF THE SAFETY PROBLEM

Collisions involving parked cars constitute a substantial proportion of all accidents. Early information, compiled by the National Safety Council in 1947, (1) showed that almost 10% of all accidents in urban areas and 5% of those in rural areas involved parked cars. In addition, 5% of the urban accidents involved cars leaving parked positions. Approximately 17% of the urban accidents and 10% of the rural accidents involved vehicles which were parked, entering or exiting a parked position, or stopped in traffic in connection with parking. Parking accidents in 10 large cities in 1940 ranged from 5% to 28% of all reported accidents. In Washington, D.C., in the late 1960's, 17% of the motor vehicle accidents involved parked cars (2). Similar values were tabulated in a 1972 nationwide accident study (3) with parked cars being involved in 13.1% of the accidents. This included 11.3% with cars parked, 0.2% with cars entering parking positions, and 1.6% in which cars were leaving parked positions. The overall parked car involvement was 17% in urban areas and 8% in rural areas (4).

While the urban proportion is significant, the severity rate of parking accidents is low. In the early data (1), 1.3% of urban and 2.5% of rural fatal accidents involved parked cars. In the 1972 study, 2.5% of the urban and 0.7% of the rural fatal accidents involved parked cars (3). For urban traffic accidents not involving pedestrians the ratio of fatal to total accidents is about 1 to 900 while the ratio of fatal parking accidents to all parking accidents is about 1 to 4,000.

Pedestrian safety involvement with parked cars is also important. Five percent of the pedestrians killed in cities and 6% of those killed in rural areas entered the roadway from behind parked cars (1). Nine percent of the pedestrians injured in each area came from behind parked vehicles.

The main reasons for introducing on-street parking controls are usually to improve capacity, traffic flow level-of-service, and/or to serve abutting properties. Safety has not been a prime objective of most parking control measures and has not been studied as intensively as other measures of traffic performance.

Humphreys et al.(5) noted that the safety aspects of parking practices have not been given the same attention as have the operational effects. He concluded that no widely accepted relationships have been established between parking configurations (diagonal, flat, angle, parallel) and parking density, traffic flow, pedestrian activity, and highway safety.

MEASURES OF EFFECTIVENESS AND EXPOSURE

On-street parking safety studies usually develop statistics based on the number of accidents in a year and severity in terms of human injury and property damage. In order to make valid comparisons it is required that the raw accident data be modified by the correct "exposure" at the study site. For example, the recent Humphreys (5) study used the number of accidents per year per mile of streets as one measure of effectiveness (MOE) to compare various parking controls and configurations. The mile of street is the linear spatial exposure element in the study. Another spatial measure would be the number of parking spaces and the MOE would be accidents per year per space. When parallel parking replaces angle parking, the exposure decreases with this MOE since there are fewer parking spaces after the change. Hence, one would expect fewer accidents based on this factor alone.

Another level of exposure would be to incorporate parking activity. The amount of parking and unparking as measured by turnover rate and/or the average occupancy of a set of spaces would incorporate this effect. Such an MOE would be accident/year/mile/space occupancy. Humphreys et al. (5) also used this measure.

The importance of traffic volume as an exposure element is found in virtually every traffic safety study. Its treatment is often complicated since the tendency is to use it directly as a measure of exposure and express safety experience as a rate. For example, parking accidents per million vehicle miles (acc/MVM) has been used in many of the studies reported. Unfortunately, the non-uniform behavior of this measure leads to problems in interpreting results as found by Humphreys et al. (5).

When the number of spaces changes such as when angle parking is changed to parallel parking, the demand for parking may change. It is then important to recognize that an areawide occupancy study, including off-street parking spaces would be necessary to capture the actual effect of such a change.

None of the studies reviewed for this synthesis was completely satisfactory with respect to exposure. None of the empirical studies in which angle parking was replaced with parallel parking reported on the change in accident experience on nearby streets where the displaced demand may have been satisfied. The same is true for cases where parking was prohibited.

Safety Study Analysis

Two main methods for studying the safety response to differences in curb parking configuration and control are the before-after study and the cross section analysis.

The simple before-after study compares accident data for a suitable period before a change is made with that recorded after the change. The change is viewed as being the sole source of the difference between the before and after periods. Including sections where no change has been made to capture citywide change determinants is useful and has occasionally been used in parking studies. The power of the before-after study lies in its holding constant all nonparking related elements. This is an extremely valuable feature when there are many factors contributing to safety experience.

The cross section analysis compares accident experience at several sites. Similar exposure is sought. When the sites appear to be identical, differences in parking control are assumed to be the sole source of safety differences. In studies where there is information on many different characteristics at each site, the opportunity exists to construct a multivariate cross section model which will predict site accident experience as a function of its characteristics.

Data from different cities will rarely be directly comparable because of wide variations in accident reporting requirements and practices. When considering only those accidents related to curb parking, criteria and judgment in elminating other accident types from the data to be studied is an important source of variation.

HUMAN BEHAVIOR

Parking safety is influenced by an extremely complex set of driver and pedestrian attitudinal and behavioral patterns. No research studies found examined these elements in depth. Traffic engineers have described conflict situations, information gathering difficulties, and vehicle control errors which are clearly causal in parking accident occurrence.

Observations of conflicts related to parking accidents show that:

1. Parking, parked, stopped, and backing vehicles are obstacles for moving traffic, both straight and turning.

2. Parking maneuvers often take place with inadequate warning to other traffic.

3. Parked vehicles reduce the sight distance of pedestrians and other traffic.

4. Persons leaving or entering parked vehicles create unexpected midblock conflicts.

Due to the many variations at parking sites, it may be expected that wide variations in accident experience exist at different locations.

Studies which have classified the actions involved in on-street accidents help scale the relative importance of these various accident conditions. Seburn (6) investigated the vehicle movements and parking positions reported for 2,100 parking accidents involving two or more vehicles. Table 1 shows the conflicts and the frequency of accidents involving vehicle actions.

TABLE 1 - Vehicle Action in Multiple Vehicle Parking Accidents

Moving Along the Street	
Straight Ahead	31%
Turning	6%
	37%
Parking and Unparking	
Parking	21%
Unparking	6%
	27%
Parked	
Curb Parked	34%
Double Parked	2%
	36%
Total	100%

SOURCE: Reference (6)

Table 2 relates parking involvement and accident types occurring at midblock on one-and two-way streets.

Operational Studies

Studies have shown that the presence of parallel parked vehicles affects moving traffic far from the curb. Also, the effect on signalized intersection capacity is great (7). Other studies have measured the time required to park and unpark for various configurations (8, 5). None of these studies related findings to accident experience.

TABLE 2 - Major Street Midblock Accidents by Type and Parking Involvement

Type of Parking Involvement	ACCIDENT TYPE (One-Way Street)								ACCIDENT TYPE (Two-Way Street)							
	Vehicle-Vehicle On-Street	Vehicle-Vehicle at Driveway	Vehicle-Fixed Object	Vehicle-Parked Car	Vehicle-Pedestrian	Vehicle-Bicycle	Miscellaneous	TOTAL	Vehicle-Vehicle On-Street	Vehicle-Vehicle at Driveway	Vehicle-Fixed Object	Vehicle-Parked Car	Vehicle-Pedestrian	Vehicle-Bicycle	Miscellaneous	TOTAL
No Parking Involvement	241	93	32	0	11	3	0	380	518	315	154	0	24	9	16	1036
Open Door	0	0	1	19	0	0	0	20	1	0	0	59	0	1	0	61
Parking	3	0	4	67	0	0	0	74	6	1	3	73	2	0	0	85
Sight Restricted	0	1	0	0	1	0	0	2	0	2	1	0	8	0	0	11
Stationary	3	2	0	264	0	0	3	272	11	5	1	357	2	0	0	376
Unparking	6	0	2	189	2	0	0	199	8	6	4	222	2	0	0	242
Subtotal Parking	12	3	7	539	3	0	3	567	26	14	9	711	14	1	0	775
TOTAL	253	96	39	539	14	3	3	947	544	329	163	711	38	10	16	1811
Proportion Parking Involved	4.7%	3.1%	17.9%	100%	21.4%	0%	100%	60%	4.7%	4.2%	5.5%	100%	36.8%	10%	0%	43%

SOURCE: Reference 5

GENERAL RELATIONSHIPS

In this section a number of general parking-safety relationships developed in a number of studies are brought together.

DIRECTIONAL CONTROL

Seburn's (6) results of parking accident safety on one-and two-way arterial streets are shown in Table 3. Note that, with regard to parking accidents, two-way streets are six times as hazardous as one-way streets. Humphreys et al.(5) did not make a direct comparison of accident rates. He found that 60% of the midblock accidents on one-way streets were parking related. Only 43% of these accidents on two-way streets were of that type (Table 2).

TABLE 3 - Parking Accident Rates by Directional Control on Major Streets

Directional Control	Accident Rate Acc/MVM
Two-Way	8.0 ± 1.7
One-Way	1.3 ± 0.5

Acc/MVM - Accidents Per Million Vehicle Miles

SOURCE: Reference (6)

STREET WIDTH

In 1964, Box (9) reported that street width and parking were important factors in minor street accidents in Skokie, Ill.

Seburn's (6) analysis of the effect of street width is shown in Table 4. Parking accident rates decrease with increasing street width. When traffic exposure is used as the MOE it appears that lateral freedom of operation on wider streets results in a decrease in accidents involving parking.

Humphreys et al. (5) found no correlation between street width and accident rates for any of the street types; -- major, collector, or local -- after correcting for parking configuration, land use and parking space utilization.

In 1968, Box (10) reported the results of a 5-year study of more than 10,000 accidents in Skokie, Ill. in which he explored parked car accidents by street types (Table 5). When these accidents were related to the mileage of streets in the community, the overall parking hazard along heavily-traveled routes was nearly eight times as great as on the local streets.

Humphreys et al. (5) related accident frequency, severity, parking involvement, and functional street classification. Table 6 presents this data for midblock accidents on two-way streets. Parking involvement varied with 39% on major streets, 86% on collector streets, and 69% on minor local streets. More than 50% of the PDO accidents and only about 25% of the injury accidents were related to parking. The injury percentage on major streets was much lower than on the minor streets.

TABLE 4 - Parking Accident Rate and Street Width

Street Width Ft.	Accident Rate Acc/MVM
Under 40	11.5
41 - 45	9.9
46 - 50	8.2
51 - 56	8.3
Over 56	4.3

Acc/MVM - Accidents Per Million Vehicle Miles

SOURCE: Reference (6)

TABLE 5 - Accident Type and Street Classification

	Parked Car Involvement	No Parking Involved	Total
Major Streets	1,174 (12%)	7,795 (88%)	8,969
Minor Streets	1,083 (43%)	1,427 (57%)	2,510
	2,257 (20%)	9,222 (80%)	11,479

SOURCE: Reference (10)

TABLE 6 - Severity, Street Classification, and Parking Involvement for Midblock Accidents on Two-Way Streets

Street Classification	Number of Accidents: Property Damage (PDO) Parking	Property Damage (PDO) Other	Injury Parking	Injury Other	Total
Local	72	27	10	9	118
Collector	32	2	4	4	42
Major	238	300	19	100	657
TOTAL	342	329	33	113	817

SOURCE: Reference (5)

Land Use

Table 7 demonstrates the relationship between land use and parking accidents as reported in the three studies cited (9,6,5). While the three studies are not directly comparable, each shows an increase in parking accidents associated with increased intensity of land use.

TABLE 7 - Land Use and Curb Parking Accidents

Land Use/ Development	Accident Rates: Acc/Mile/ Year	Accident Rates: Acc/MVM	Reference
Residential:			
Single Family	1.0		
Apartments	3.1		
Business	3.5		
Industrial	1.2		(9)
Downtown		1.6 ± 0.9	
Intermediate		0.9 ± 0.2	
Outlying		0.9 ± 0.4	(6)
Residential		1.5	
Apartment		5.4	
Office		8.4	
Retail		11.8	(5)

Acc/MVM - Accidents Per Million Vehicle Miles

Parking Utilization

Parking utilization was found by Humphreys et al. (5) to be a significant factor. Figure 1 shows that increases in parking space occupancy up to approximately 1.5 MAVH/M (million annual vehicle hours of parking per mile of street) are associated with increases in the parking accident rate. Humphreys concludes that the accident rate does not increase further above this level although no data supporting this conclusion are tabulated.

Traffic Flow

Traffic flow is an important and complex contributor to parking accident occurrence and must be considered in an effective analysis of parking accident experience. Table 8 shows Seburn's (6) analysis of parking accident experience on streets for two volume classes. Parking accident rates are less than 10% as great on heavier traveled streets as on those with ADT's from 5,000 to 10,000 vpd. Humphreys et al. (5) found that as volume increases the accident rate band sloped downward to about 5,000 ADT and then became constant.

TABLE 8 - Traffic Flow and Accident Rate

ADT	Accident Rate Acc/MVM
5-10,000	12.4 ± 1.8
>10,000	0.9 ± 0.2

Acc/MVM - Accidents Per Million Vehicle Miles

SOURCE: Reference (6)

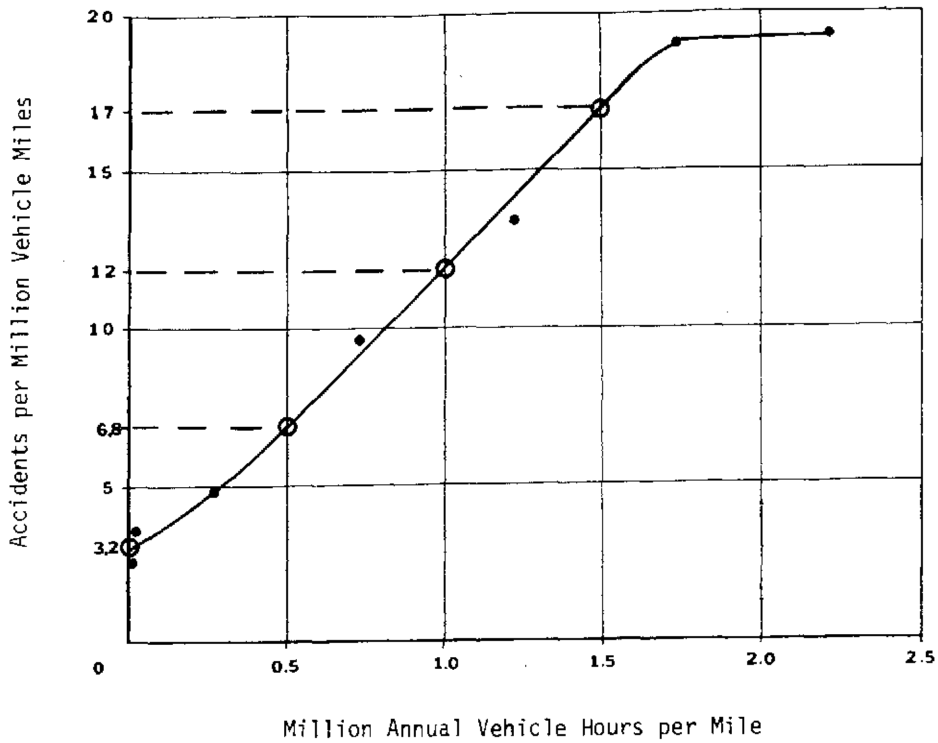

Figure 1. Accident Rates vs Utilization, Major Streets, All Land Uses

SOURCE: Reference 5

PARKING PROHIBITION AND REGULATION

In this section, consideration is given to parking prohibition and time-limit control. The removal of curb parking to expedite traffic movement is a well-established traffic engineering technique. The space released is used for an additional travel lane, to increase effective lane width for traffic, or to create a median area which can be used for the improved operation and safety of left turning vehicles.

Table 9 summarizes results from 12 empirical before-after studies of parking prohibition. It is clear that prohibiting parking, especially during peak hours, leads to a reduction in on-street accident experience. The range of this reduction is wide, reflecting different conditions in each of the study areas. (See Table 9 for citation of the reference numbers.) Several of the before-after studies summarized in Table 9 are discussed in the paragraphs that follow.

A 1947-8 Chicago Transit Authority (12) study of 4-hour peak period parking control revealed benefits far beyond expectations. Control was implemented on seven major arterials, primarily radials. Traffic volumes increased more than 15%. Average speeds went up by at least 25% for automobiles and an average of 8% for streetcars on two streets. Accidents for two streets for 1-year before were compared with those for 1-year after control was introduced. On one street there was a 34% accident reduction and on the other a 24% accident reduction during the 4-hour peak period.

Pak-Poy (13) described Australian experience over a 4-week interval with peak period parking prohibitions on the 5-mile radial Anzac Highway in Adelaide. The 32-foot-wide traveled way includes a 24 foot curbed median and minimal channelization at the six median openings per kilometer. One major intersection is signalized and bus flow is heavy, almost 1/min. Peak hourly flows ranged from 1,300 to 3,000 vph in the direction of major flow. In addition to operational improvements and a 29% reduction in peak period accidents, only 70 parked vehicles were displaced from their previous locations.

In 1965, the Michigan Highway Department (14) reported a 44% reduction in on-street accident frequency in Garden City during the after period of a 9-month before and after parking prohibition study. The most dramatic reductions were in midblock accidents most influenced by curb parking. Annual motorist savings exceeded $100,000 in reduced repair costs and medical expenses.

TABLE 9 - Summary of Before-After Studies Involving Parking Prohibition on Major Streets

Location	Report Year	Accident Reduction	Comments	Reference
Dallas, Texas	1946	22%		(11)
New Orleans, Louisiana	1947	90%		(1)
Chicago, Illinois	1949	24% 34%	Two streets, 4-hour peak	(12)
Adelaide, Australia	1963	29%	4-week trial period, reduction in peak hour accidents	(13)
Garden City, Michigan	1965	44%	PDO accidents decreased 38% Injury accidents decreased 65%	(14)
Dearborn, Michigan	1966	3.5% 19.1%	First year reduction Total 2-year reduction	(15)
Beverly Hills, California	1967	--	195 fewer PDO accidents in 1965 compared with 1961	(16)
Detroit (Suburb), Michigan	1968	24%	32% increase in traffic	(17)
Gateshead, United Kingdom	1968	20%		(18)
Yuma, Arizona	1969	41%		(19)
Knightsbridge, United Kingdom	1973	34%	Traffic increased 15%-20% in peak periods	(20)
Sheffield, United Kingdom	1973	12%		(20)

In 1966, DeRose (15) reported a 3-year study conducted on one side of an undivided street in Dearborn, Mich. The ADT was in excess of 45,000. For the 1-year before period the 60-foot-wide street operated with 10-foot lanes and 170 curb parking spaces. Parking and left turns were prohibited during peak hours. During the 2-year after period, the curb parking was removed. The street operated with 12-foot lanes. The center lane was used reversibly during peak hours and for left turns at other times. Peak traffic flow increased 10% and travel speed increased 20% following the change. Parking related accidents dropped from 59 to 4. Table 10 shows that since there was peak period prohibition of parking before the change the additional parking prohibition was primarily responsible for the accident reduction.

Hoffman et al. (17) reported the results of prohibiting parking completely on a 64-foot-wide major arterial street in a Detroit suburb. Several left turn prohibitions were removed at intersections after a center left turn lane was installed. A 2-year before-after study revealed almost a 32% increase in traffic flow to almost 30,000 vpd. The resulting accident reduction is shown in Table 11. Parked car accident involvement dropped from 48 before to 1 after. An 81% reduction in rear end accidents involving left turning vehicles (82 accidents) was also recorded.

Traffic engineering measures applied to U.S. Highway 80 in Yuma, Ariz., produced a significant reduction in accidents according to Crosette and Allen (19). In a 14-block study area containing two "no parking during school hours" sections, there were 100 accidents in 1963 (9.70 accidents/MVM, compared with the Arizona average of 3.73 on urban highways). In July 1964 all curb parking was removed, signals were synchronized and interconnected, and painted channelization was introduced. In 1966 there were 59 accidents, a 41% reduction from 1963. The accident rate dropped almost 50% to 4.85 accidents/MVM. The severity ratio (ratio of personal injury to total accidents) decreased from 47% to 41%. Crosette and Allen estimated that accident costs had been reduced by $46,000 a year.

TABLE 10 - Total Accidents - Peak and Off-Peak Periods

Number of Accidents

	PEAK PERIODS			OFF-PEAK
	6-9 A.M.	3-6 P.M.	TOTAL	
One Year Before	31	84	115	230
1st Year After	34	86	120	213
2nd Year After	30	80	110	169

SOURCE: Reference 15

TABLE 11 - Summary of the Effects of Parking Prohibition

	Severity		Location		
	Injury	Property Damage	Inter-section	Midblock	Total Accidents
Before	275	384	353	306	659
After	235	266	297	204	501
Accident Reduction	40	118	56	102	158

SOURCE: Reference 17

Cross Section Studies

Several cross section studies have compared accident experience at different locations with some having parking prohibited. Results of three studies are summarized in Table 12 and are described in the following paragraphs.

TABLE 12 - Effect of Parking Prohibition

Location or Study	Report Year	Accident Reduction	Comment	Reference
San Francisco	1966	32% 42%	Intersection Midblock	(21)
Maine	1972	10%	Compared with parallel parking	(22)
Humphreys et al.	1978	19% 73%	Midblock, low utilization Midblock, high utilization	(5)

A 1966 California study by Marconi (21) compared accident rates in a cross section study for a small number of sites in San Francisco. The results are shown in Table 13.

In 1971, Segal (22) reported on a statewide analysis of urban accidents in the State of Maine. Table 14 was developed from his report for more than 3,100 street sections recording more than 30 accidents. The average accident rates for the two most common configurations, no parking on either side and parallel parking on both sides, were only 10% different. No statistical analysis was given.

Humphreys et al. (5) found that the prohibition of curb parking along major streets with low utilization (approximately 0.5 MAVH/M) could be expected to reduce mid-block accident rates by up to 19%. At higher levels of utilization (1.0 MAVH/M) the reduction could be up to 73%. Prohibited parking was compared with parallel parking. This included marked skip spaces at locations with similar land use and very low levels of parking utilization. It was found that the parallel parking arrangement had an accident rate of 14.3 acc/MVM, more than 4 times as great as 3.4 acc/MVM when parking was prohibited

Parking Time Limit Control

Only two studies presented information on the safety response associated with time-limit controls.

Seburn's (6) cross section study found that sections with unrestricted parking had greater parking accident rates than streets with short time limited parking or with parking completely prohibited. Nonparking accident experience was highest where parking was prohibited and least where it was unrestricted (Table 15).

Green and Inwood (23) described the effect of metered parking on accidents in three areas of London in 1963. Personal injury accidents were studied for 4 years, commencing 1 year before the first meters installed. The accidents in these and neighboring areas were compared with those in all metropolitan boroughs and on certain main streets near the metered zone. Accidents were reduced in all three areas. In only one area was this reduction statistically significant. Fatal and serious accidents decreased 8% in the meter zone while increasing 40% nearby. Compared with rates on surrounding major streets, accidents in the study zones fell by 12% during meter hours and by 25% outside these hours.

TABLE 13 - Effect of Parking Prohibition at Intersections and Midblock Locations

	Accident Rates	
	Intersection (acc/MEV)	Midblock (acc/MVM)
Parking Prohibited	0.43	2.1
Parking Permitted	0.63	3.6

Acc/MEV - Accidents Per Million Entering Vehicles

Acc/MVM - Accidents Per Million Vehicle Miles

SOURCE: Reference 21

TABLE 14 - Parking Control and Accident Rate

Parking Configuration	No. of Street Sections	Average ADT	Average Acc Rate (Acc/MVM)
No Parking - Both Sides	1,500	7,300	3.4
No Parking One Side and Parallel - One Side	175	9,800	4.2
Parallel - Both Sides	1,472	7,000	3.8

Acc/MVM - Accidents Per Million Vehicle Miles

SOURCE: Reference 22

TABLE 15 - Parking and Nonparking Accidents by Type of Control

	Accident Rates	
Parking Control	Parking Accidents (Acc/MVM)	Other Accidents (Acc/MVM)
Unrestricted	1.16 ± .41	.87 ± .35
Time Limited	.87 ± .40	2.34 ± .65
Prohibited	.95 ± .67	4.28 ± 2.85

Acc/MVM - Accidents Per Million Vehicle Miles

SOURCE: Reference 6

PARKING CONFIGURATION

The geometric arrangement of parking spaces, whether parallel to or at an angle with the curb, has long been of great interest to traffic engineers. This section reports on safety findings in this area.

Within the parallel parking mode there are several ways of organizing the spaces which influence street and parking capacity. These include the uncontrolled (unmarked) location of parking along the curb, designated (marked) lengths for each vehicle, and designated positions (marked) for each vehicle with specific maneuvering space (marked) between two adjacent positions termed "paired parking."

Table 16 presents summary information for five before-after studies involving changes from angle to parallel parking. See Table 16 for reference numbers.

Table 17 is summarized from an eight city Utah Study (25). Street widths varied from 58 feet to 108 feet. Individual city effects ranged from no change to a 73% reduction in the total number of accidents and from a 5% to a 79% reduction in accident rate. The reduction in parking spaces was not taken into account. There was also a lesser reduction in nonparking related accidents following the change.

TABLE 16 - Summaries of Before-After Studies Involving a Parking Change from Angle to Parallel Parking

Study Location	Report Year	Accident Reduction	Comment on Results	Reference
Minnesota City	1947	41%	27 Accidents before 16 Accidents after	(1)
Wichita, Kansas	1950	63%	8 Accidents before 3 Accidents after	(24)
Utah	1966	28% 57%	Average for 8 studies (range C to 73%) Reduction for parking related accidents	(25)
Grand Rapids, Michigan	1967	19%		(2)
Kansas City, Missouri	1967	50%	Accidents/Block 5 before, 1 after	(2)

TABLE 17 - Effect of Angle vs. Parallel Parking

Parking Configuration	Numbers of Accidents: Total Accidents	Parking Related	Injuries & Fatalities	Overall Accident Rate Acc/MVM
Angle (Before)	466	93	64	15.3
Parallel (After)	336	40	38	10.9
Reduction	28%	57%	41%	29%

Acc/MVM - Accidents Per Million Vehicle Miles

SOURCE: Reference 25

Cross Section Studies

Cross section studies within the same city show similar effects when comparing angle with parallel parking. Table 18 summarizes 5 of these studies.

Smith (1) cited an Oakland, Calif., study of two six-block sections of street in the CBD with the same width. Buses used the street with parallel parking while no buses were on the street with angle parking. The flow on the latter was about 75% of the former.

TABLE 18 - Cross Section of Angle vs. Parallel Parking Studies in Same City

City/State	Report Year	Accident Reduction	Comment	Reference
Oakland, California	1947	50%	46 Accidents with angle parking 23 Accidents with parallel parking	(1)
Salem, Oregon	1948	53% 65%	All Accidents Parking Related Accidents	(26)
Mesa, Arizona	1960	71%		(27)
San Francisco, California	1966	58%	Total Accidents	(21)
Abilene, Texas	1975	59%		(28)

In 1948, Crandall (26) reported on a 5-year study of two similar blocks in Salem, Oreg., one with angle and the other with parallel parking. The streets were 59 feet wide in a business area with 7,500 ADT. Table 19 is adapted from this study.

TABLE 19. Angle vs. Parallel Parking Effects by Accident Type

Accident Type	Parking Configuration: Angle Accidents	Parallel Accidents	Accident Reduction
	(Number)	(Number)	
Parking Related	57	20	65%
Other	20	16	20%

SOURCE: Crandall (26)

The Arizona Highway Department (27) reported accident rates on two sections of a 100-foot-wide U.S. numbered route in the Mesa CBD with angle and parallel parking as shown in Table 20.

Data for the Abilene, Tex., Study (28) is shown on Table 20.

Marconi (21) reported studies made in San Francisco showing that for similar block lengths, signals, and other factors parking configuration affected the midblock accident rate. The accident rate for parallel parking was 2.96 acc/MVM and for angle parking was 7.12 acc/MVM.

Humphreys et al. (5) found that 22-1/2 degree parking appeared to be associated with accident rates higher than either parallel parking or 30 and 45 degree parking. Rates for parallel parking appear higher than 30 degree or 45 degree parking (Table 21). These results are explained by Humphreys et al. as they relate to the average ADT. One hindrance to straightforward interpretation of the data was that the parking types with higher accident rates had ADT values below 5,000. Also, those comparisons that might have been expected to be significantly different (such as parallel vs. angle) were not. Those that might have reasonably been expected to be similar (such as 22-1/2 degrees versus 30 degrees) were found to be significantly different. The differences were attributed to the low ADT values rather than the parking type.

Segal (22) compared angle and parallel parking for 1,523 urban sites in Maine as presented in Table 22. He found an 88% lower accident rate for parallel parking as compared to angle parking. Mixes of types on the different sides of the street at 46 sites resulted in intermediate values.

TABLE 20 - Angle vs. Parallel Parking

	Accident Rate (Acc/MVM)			
	Minimum	Maximum	Mean	Mean Acc. Reduction
Mesa, Arizona (27) Angle Parking	2.2	8.0	4.9	
Parallel Parking	0.0	1.8	1.4	71.4%
Abilene, Texas (28) Angle Parking			28.4	
Parallel Parking			11.6	59.4%

Acc/MVM - Accidents Per Million Vehicle Miles

SOURCE: References 27, 28

TABLE 21 - Accident Rates and Parking Angle (Major Streets)

	Parking Angle	Parking Utilization	Average ADT	Accident Rate Acc/MVM
ADT > 5,000	30^{0}	1.1	13,600	3.3
	45^{0}	2.4	16,400	7.6
	Parallel	1.2	10,500	10.0
ADT< 5,000	$22\text{-}1/2^{0}$	1.3	4,200	17.2
	Paired-Parallel	1.5	3,400	26.9

Acc/MVM - Accidents Per Million Vehicle Miles

SOURCE: Reference 5

TABLE 22 - Angle vs. Parallel Parking in Cities, State of Maine

Parking Configuration	No. of Sections	Average ADT	Average Acc. Rate (Acc/MVM)
Parallel - Both Sides	1,472	7,000	3.8
Parallel - One Side Mixed Parallel and Angle - One Side	10	5,600	11.6
Parallel - One Side Angle - One Side	36	10,300	15.0
Angle - Both Sides	5	5,100	31.5

Acc./MVM - Accidents Per Million Vehicle Miles

SOURCE: Reference 22

Central Business District Core Study

Kell and Johnson (29) evaluated several traffic engineering improvements in the CBD cores of Sunnyvale and Redwood City, Calif., cities with 1970 populations of 95,408 and 55,686, respectively. Combinations of signal timing changes, one-way streets, and turn prohibitions were introduced. Curb parking was prohibited on major streets. Angle parking replaced parallel parking on nearby side streets. A no-stopping towaway regulation from 7:00 a.m. to 6:00 p.m. on the south side of a main street eliminated parking conflicts and allowed re-striping of the street for wider lanes.

There was a pronounced reduction in midblock accidents on the Redwood City street where parking was prohibited. No midblock accident occurred during the test period. An average of five accidents occurred during corresponding time periods for the previous 2 years. There was no change in the midblock accident rate on the two streets where angle parking was introduced. This was in contrast to the performance in the remainder of the CBD core area where more midblock accidents were recorded.

Where one-way and unbalanced flow was introduced, there were no parking related accidents on the major street. The total number of midblock accidents was reduced although speeds occurred that sometimes contribute to increases in accident rates.

PEDESTRIAN SAFETY AND PARKING

In recent years there has been much interest in urban pedestrian safety. Parking changes to improve this aspect of the problem have been carefully considered. This section summarizes these studies.

Almost 980 pedestrian accidents that occurred in San Jose, Calif., during 1967-69 were analyzed by Walsh (30). Although this was only 3% of the total accident experience in the city, the severity was much greater with 7% of those injured and 25% of those killed being pedestrians. About 20% of all accidents involved pedestrians entering the travel lanes from behind parked cars. Driver vision was considered obscured by the parked vehicle in these cases. Forty-five percent of the injured pedestrians, less than five years of age, were involved in this type of accident. This group also accounted for about 15% of all pedestrian injuries. Twenty-nine percent of the 5 to 14-year-olds accident involvement was of this same type. Older persons, 78% of those injured, were rarely involved in this type of accident.

New York State pedestrian accident data (31) showed that 8% of the 786 killed and 19% of those injured were judged to have been coming from behind parked cars. Washington, D.C., data for the 1950's revealed that this action occurred in more than one-half of the fatal accidents involving school-age children (32).

Berger (33) studied the introduction of angle parking as an urban pedestrian accident countermeasure in two cities. The Miami, Fla., site was a one-way street, 34 feet wide with twelve 30-degree spaces on one side and no parking permitted on the other side. The original two lanes of traffic, plus two parking lanes, were reduced to one moving lane plus the parking lane. Abutting land use consisted of stores and apartments. The San Diego, Calif., site was a lightly used, two-way street, 30 feet wide with unrestricted two-side parallel parking. It was converted to one-lane one-way operation, with twenty-six 45-degree angle spaces on one side and no parking permitted on the other side. Land use was of the mixed single and multi-family type. Seventeen observations of pedestrians were recorded in Miami. Forty-five observations of pedestrians were recorded in San Diego.

Running into the first traffic lane did not seem to be affected by the parking configuration. In Miami, children ran into the roadway twice as frequently on the diagonal parking side as on the side with no parking. (This is not a significant difference.) The scanning of traffic by pedestrians significantly increased at both sites. The diagonally parked vehicles directed the pedestrians into the roadway at such an angle that looking in the direction of traffic was encouraged. There was a significant increase in the percent of pedestrians who aborted their crossing, hesitated in the traffic lane, and backed up in the traffic lane. In Miami, these results can be attributed to those pedestrians who entered the roadway from the side of the street opposite the angle parking spaces. Running in the roadway was significantly reduced in Miami. This reduction was noted for pedestrians entering the roadway from either side. Interviews with 15 residents in the San Diego experimental section resulted in one-third feeling that accidents would decrease with one-third being undecided. Two-thirds of the residents complained about the angle parking.

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SUBJECT INDEX