TRANSPORTATION RESEARCH RECORD 819

Operational Effects of Geometrics and Improvement Evaluations

TRANSPORTATION RESEARCH BOARD

COMMISSION ON SOCIOTECHNICAL SYSTEMS
NATIONAL RESEARCH COUNCIL

NATIONAL ACADEMY OF SCIENCES
WASHINGTON, D.C. 1981

TRANSPORTATION RESEARCH BOARD
National Research Council

ERRATA 1982-1985

Special Report 201

page 17, column 2, second paragraph, should read
"Tools also need to change as the nature of options changes significantly. Emerging policy options are not largely focused on network-expansion investments, whereas traditional models were developed long ago to deal with such options."

Special Report 200

page 3, column 1
Change the caption for the bottom figure to
"A new AM General trolley bus starts down the 18 percent grade on Queen Anne Avenue North in Seattle in October 1979 (photograph by J. P. Aurelius)".

Transportation Research Record 1040

page ii
Under "Library of Congress Cataloging-in-Publication Data," delete "Meeting (64th: 1985: Washington, D.C.)" and "ISBN 0361-1981"

Transportation Research Record 1020

page 7, Figure 1
The histogram should reflect that the rail mode is represented by the black bar and that the highway mode is represented by the white bar.

Transportation Research Record 1017

page 19, column 1, 7 lines above Table 1
Change "ranged from 1 in.^{2} to nearly 30 in.^{2} of runoff" to "ranged from 1 area inch to nearly 30 area inches of runoff"

page 22, column 1, last line
Change "1 to nearly 30 in.^{2}" to "1 to nearly 30 area inches"

page 22, column 2, first line
Change "13 in.^{2}" to "13 area inches"

Transportation Research Record 1011

page 12, Figure 4
Figure does not show right-of-way structure for O-Bahn. See discussion on page 11, column 1, paragraph 3.

Transportation Research Record 996

page 49
Insert the following note to Figure 2:
"The contour lines connect points of equal candlepower."

page 49
Insert the following note to Figure 3:

"The candlepower contours are superimposed on a 'headlight's-eye-view' of a road scene. The candlepower directed at any point in the scene is given by the particular candlepower contour light that overlays that point. For example, 1400 candlepower is directed at points on the pedestrian's upper torso. For points between contour lines, it is necessary to interpolate."

page 50
Insert the following note to Figure 3:

"Where

ρ = the azimuth angle from the driver's eye to a point P on the pavement;
θ = the elevation angle from the driver's eye to a point P on the pavement;
EZ = the driver's eye height above the pavement; and
DX, DY, DZ = the longitudinal, horizontal, and vertical distance between the headlamp and eye point.

Then

$EX = EZ/\text{Tan}\,\theta \qquad HZ = EX\text{-}DZ$
$H1^2 = EZ^2 + EX^2 \qquad HX = EX\text{-}DX$
$EY = H1\ \text{Tan}\,\rho \qquad HY = EY\text{-}DY$
$H2^2 = H1^2 + EY^2 \qquad H3^2 = HX^2 + HZ^2$
$\alpha = \text{Tan}^{-1}\,(HZ/HX),\ \beta = \text{Tan}^{-1}\,(HY/H3),\ H4^2 = H3^2 + HY^2$"

Transportation Research Record 972

page 30, column 2, 22 lines up from bottom
Reference number (5) should be deleted

page 31, column 2, 5 lines up from bottom
Reference number should be 5, not 4

page 34, column 2, 8 lines above References
Reference number (5) should be deleted

Transportation Research Record 971

page 31, reference 3
Change to read as follows:

Merkblatt fürLichtsignalanlagen an Landstrassen, Ausgabe 1972. Forschungsgesellschaft für das Strassenwes en, Köln, Federal Republic of Germany, 1972.

Transportation Research Record 965

page 34, column 1, Equation 1
Change equation to
$r_u = \gamma_w \cdot h/\gamma \cdot z$

where

γ_w = unit weight of water,
γ = moist unit weight of soil,
h = piezometric head, and
z = vertical thickness of slide.

Transportation Research Record 905

page 60, column 1, 9 lines up from bottom
Change "by Payne (6)" to " by us"

Transportation Research Record 819

page 47, Table 1

Replace with the following table.

Table 1. Summary of interactions between signal-timing parameters and MOEs.

Timing Method	Parameter	Total Delay	Stops	Fuel Consumption	Emissions		
					HC	CO	NO_x
Manual	Cycle length	⊕	⊕	⊕	⊕	⊕	⊕
	Speed of progression	+	⊕	+	+	+	+
	Priority policy	+	+	+	+	+	+
	Split method	+					
TRANSYT	Cycle length	⊕	⊕	⊕	⊕	⊕	⊕
	K-factor	+	⊕				
	Priority policy				+		

Note: + = main effect detected from TRANSYT output, and ○ = main effect detected from NETSIM output.

Transportation Research Record 869

page 54, authors' names

The second author's name should read "Edmond Chin-Ping Chang"

Transportation Research Record 847

page 50, Figure 3

Add the following numbers under each block in the last line of the flowchart:

R1, R2, R3, R4, D1, D2, D3, A1, A2

page 50, Figure 4

Make the following changes in the last line of the flowchart.

Change "R4" to "D1" and "Recognition" to "Decision"

Change "R5" to "D2" and "Recognition" to "Decision"

Change "R6" to "D3" and "Recognition" to "Decision"

Change "R7" to "R4"

Change "R8" to "D4" and "Recognition" to "Decision"

Change "R9" to "A1" and "Recognition" to "Action"

Change "R10" to "A2" and "Recognition" to "Action"

Transportation Research Record 840

page 25, column 1, line 5

Change "money" to "model"

Transportation Research Record 831

page ii, column 1

Change ISBN number to "ISBN 0-309-03308-X"

Transportation Research Circular 255

page 6, column 1, third paragraph

Change "Marquette University" to "Northern Michigan University"

NCHRP Synthesis of Highway Practice 87

page ii

Change ISBN number to 0-309-03305-5

NCHRP Synthesis of Highway Practice 84

page ii

Change ISBN number to 0-309-03273-3

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Survey of Single-Vehicle Fatal Rollover Crash Sites in New Mexico

JEROME W. HALL AND PAUL ZADOR

The results of a study of roadway and roadside characteristics at the sites of 151 fatal overturning crashes in New Mexico are discussed. Comparisons were made with data from nearby locations on the same roads and with data from a similar study of 214 sites in Georgia. The New Mexico crash sites were characterized by sharper curvature and curves to the left, steeper downgrades and embankments, and greater embankment depths than the nearby comparison sites. The Georgia sites exhibited significantly sharper curvature, flatter grades, more spot fixed objects, and steeper but shallower embankments than the New Mexico sites. Guardrail use was significantly higher in Georgia. In New Mexico, the roadsides at a majority of the sites of fatal overturning crashes do not satisfy current guardrail warrants, and it is recommended that these warrants be reexamined. The difference in the values of alignment characteristics between the two states suggests that priority schemes for selecting hazardous locations cannot currently rely on uniform, nationwide criteria.

In 1979, there were 8911 fatalities in single-vehicle rollover crashes; these accounted for 21 percent of nationwide vehicle occupant fatalities. Previous research (1) has shown that the problem is especially critical in western states. The 1979 U.S. Department of Transportation Fatal Accident Reporting System data indicate that more than 40 percent of occupant fatalities in Montana, New Mexico, and Wyoming involved single-vehicle rollover crashes whereas less than 15 percent of occupant fatalities in Illinois, New Jersey, and Pennsylvania involved this type of crash.

Despite the importance of rollover crashes, there is no indication that their highway-related aspects have been studied. The lack of previous study is probably attributable in part to traditional beliefs that hold that single-vehicle crashes are the fault of the driver rather than the roadway. As a consequence, engineers have remained complacent with respect to their responsibilities for this type of crash and have justified their inaction on the assumption that appropriate remedial action is beyond their control.

The research described in this report was designed to examine this questionable premise. Specifically, the research sought to evaluate the hypothesis that the roadway and roadside characteristics at the sites where rollover crashes occurred were more adverse than for the road system in general. Separate but similar studies were conducted in Georgia (see the paper by Wright and Zador in this Record) and New Mexico. This study describes the methodology and findings of the study of rollover crashes in New Mexico and compares the findings of the New Mexico and Georgia studies.

METHOD

The study discussed in this paper was designed to compare roadway and roadside characteristics at the sites of fatal rollover crashes in New Mexico with similar characteristics for a matched set of comparison sites. The field-study procedure was similar to that used in a previous study of fixed-object crashes (2), which selected a set of comparison sites located 1.6 km in advance of the crash site. The crash vehicle and driver would generally have passed the comparison site within 1-2 min of reaching the site of the fatal crash.

The locations studied were the sites of all fatal single-vehicle rollover crashes in New Mexico for the one-year period ending July 31, 1979. The study did not include eight fatal overturning crashes that involved motorcycles, six crashes in which a second vehicle was involved, or three dirt-road crash sites that could not be located. Studies were conducted at the sites of 151 fatal rollover crashes, which represented more than 25 percent of New Mexico's fatal crashes during the study period.

The sites were located in the field through reference to data provided in the reports of the investigating officers. Although these reports varied in quality and the sites were not studied until 4-8 weeks after the crashes, the damage associated with the crashes usually made it possible to identify study sites. When there was doubt concerning the crash site, assistance was obtained from the investigating officer.

A three-person field crew was used to make an engineering survey in the vicinity of each crash and comparison site. Measurements were made of curvature, superelevation, and gradient; roadside spot objects were enumerated; and elongated objects were measured. The alignment measurements were made by using techniques described in a previous report (3). Other characteristics of the sites, including road and shoulder widths, roadside slopes, pavement friction, speed limit, and number of intersections and driveways, were also recorded. At the crash sites, measurements were made of the lateral and longitudinal distances traveled off the roadway by the overturning vehicle.

RESULTS OF THE NEW MEXICO STUDY

One of the most obvious differences between the crash and comparison sites was found to be horizontal alignment. At both sites, 10 curvature measurements were made at approximately 30-m intervals, from 137 m before to 137 m beyond the site. Figure 1 shows the average curvature at 10 positions at both the crash and comparison sites. The average curvature at the crash site was significantly higher than at the comparison site for each position from 137 m before through 76 m beyond the site (unless otherwise noted, all statistical comparisons were performed by using t-tests with $\alpha = 0.05$). The average curvature of 1.7° for all crash sites was significantly higher than the average curvature of 0.7° at the comparison sites. In the area that was most critical to the approaching driver, from 137 m before through 15 m beyond the crash site, the average curvature of 1.9° was also significantly higher than that at the comparison site (0.7°).

The relatively low average values of curvature are misleading. Approximately 36 percent of the comparison sites, versus 54 percent of the crash sites, had a maximum curvature of more than 0.5°. Analysis of curvature at these nontangent locations found that the average curvature at the crash sites was 3.1°, significantly higher than the average value of 2.0° at the comparison sites. The difference at the nontangent locations was even more significant (3.5° versus 1.9°) in the area from 137 m through 15 m beyond the site.

Only 10 percent of the comparison sites had a maximum curvature of 6° or more, whereas at crash

Figure 1. Average degree of curvature for crash and comparison sites.

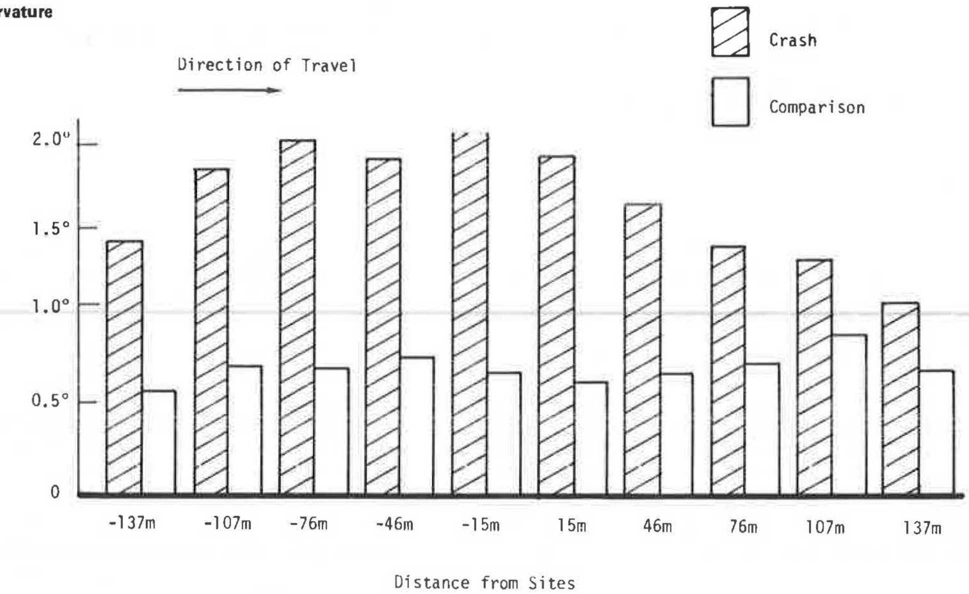
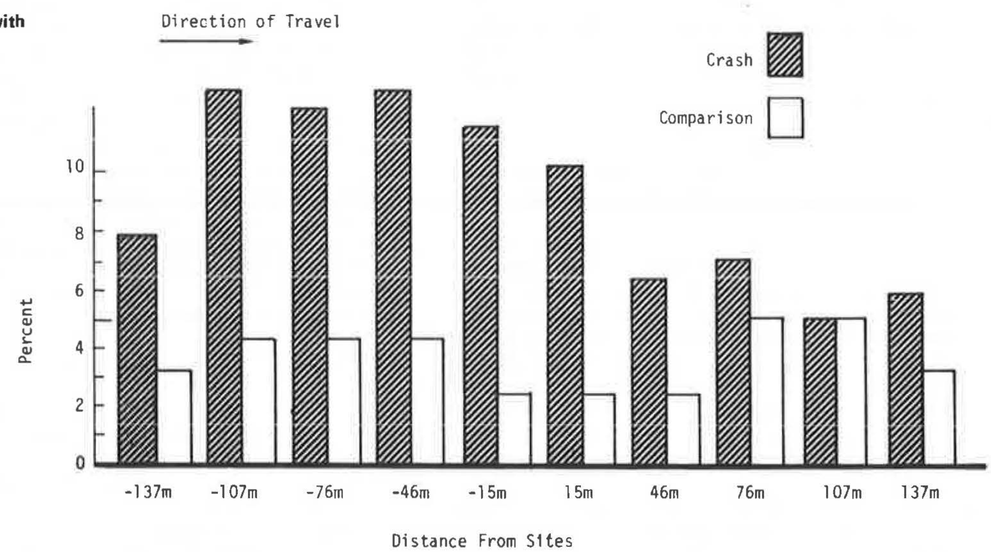


Figure 2. Percentage of sites with curvature greater than 6°.



sites the comparable figure was 21 percent. The difference in horizontal curvature between crash and comparison sites is also suggested in Figure 2, which shows the percentage of sites where curvature was greater than 6° at the 10 measurement positions. There is clearly a difference between the sites in the area immediately before the crash position.

Roadway curvature was also analyzed with respect to the direction of curvature. By using the sign convention of "+" for curves to the left and "-" for curves to the right, it was found that the average maximum curvature was +0.2° at the comparison sites and +1.4° at the crash sites. Although both average values indicate curves to the left, only the value for the crash site was significantly different from zero. The average mean curvature at the comparison sites (+0.03°) did not differ significantly from zero, whereas the corresponding value at the crash site (+0.79°) did. The significant overrepresentation of left-hand curves at crash sites is indicated by the data given below, which compare the direction and sharpness of curvature:

	Percentage of Sites	
Curvature	Crash	Comparison
>5° right	8	5
0.5°-5° right	9	17
Tangent	46	64
0.5-5° left	22	8
>5° left	15	6

An analysis of the pavement superelevation at crash and comparison sites yielded results consistent with those found for curvature. Due to the higher average curvature at crash sites, average and maximum superelevations at these locations were significantly higher than those at comparison sites.

In the vicinity of both the crash and comparison sites, 11 measurements of roadway gradient were taken at approximately 30-m intervals, from 152 m before to 152 m beyond the site. The analysis of average gradients found that they were significantly steeper at the crash sites (-0.92 percent) than at the comparison sites (-0.33 percent). Figure 3 shows the average gradient at each of the 11 measurement positions. For each position from 61 m

Figure 3. Average roadway gradient for crash and comparison sites.

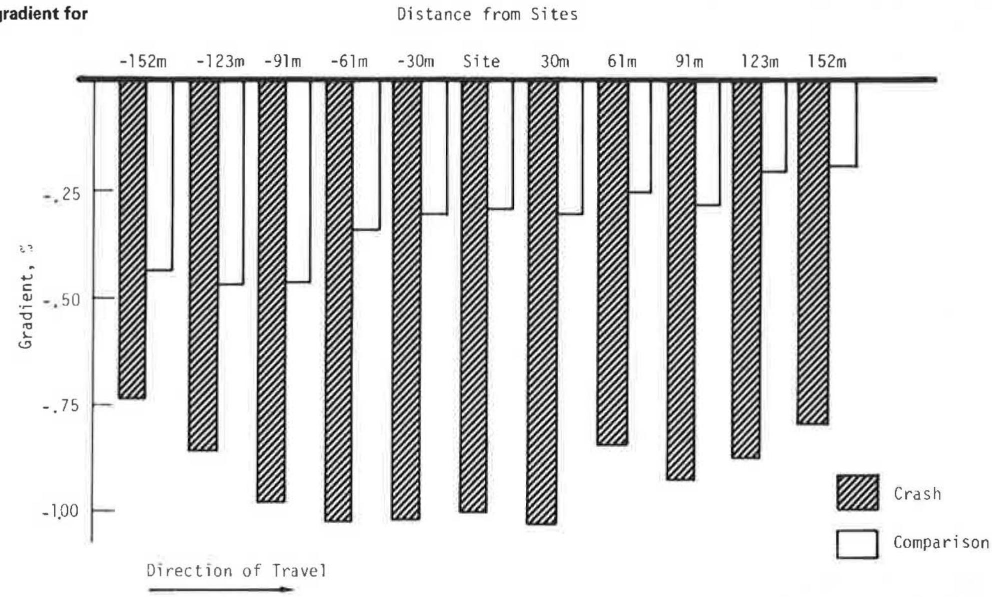


Table 1. Comparison of maximum curvature and minimum gradient at crash and comparison sites.

	Percentage of Sites					
	Gradient < -2 Percent		Gradient -0.55 to -2 Percent		Gradient > -0.55 Percent	
Curvature	Crash	Comparison	Crash	Comparison	Crash	Comparison
>5° right	4.6	4.0	2.0	0	1.3	0.7
0.5°-5° right	4.6	5.3	3.3	7.3	0.7	4.6
Tangent	9.3	11.2	21.9	24.5	15.2	27.8
0.5-5° left	8.6	2.6	4.6	0.7	9.3	2.7
>5° left	12.6	6.0	1.3	1.3	0.7	1.3

before through 152 m beyond the site, the downgrade at the crash sites was significantly steeper than at the comparison sites.

Logic suggests that the average roadway gradient should be zero, and it was therefore surprising that the gradients at the comparison sites were negative. However, 21 of the crash sites (14 percent) were on lengthy downgrades, where the grade was continuously negative for at least 1.6 km before the crash site. These cases, in which both the crash and comparison sites were on the same downgrade, influence the total results shown in Figure 3. When these cases are removed from the analysis, the average gradient at the remaining comparison sites was +0.01 percent, which differs significantly from the average crash-site gradient of -0.65 percent.

Curvature and gradient data show that roadway geometrics were significantly worse at the sites of fatal overturning crashes than at comparison sites. When the condition described by the combined effect of curvature and gradient is examined, the results again indicate that crash sites were characterized by poorer geometric conditions. Table 1 indicates that the crash sites exhibited a higher incidence of the combination of sharp curvature and downgrades than did the comparison sites. The safest condition in the table--tangent roadways on grades more positive than -0.55 percent--accounted for nearly 28 percent of the comparison sites versus only 15 percent of the crash sites. The most adverse condition in the table--curvature in excess of 5° and grades less than -2 percent--was found at 17 percent of the crash sites versus only 10 percent of the comparison sites. Table 1 also indicates the dominance of left-hand curves at crash sites. Analysis showed that the principal factors in distinguishing between crash and comparison sites were the degree and the direction of curvature.

A general observation from the field studies was that a comparatively small object was the most probable immediate cause of overturning. These objects included curbs, edge drop-offs, ditches, and soft soil. However, since a fixed-object collision is one alternative to overturning for a vehicle that has left the roadway, a survey was made of fixed objects within 9 m of the side of the roadway on which the vehicle overturned. Separate surveys were conducted for 161 m before and beyond the crash and comparison sites. Spot fixed objects were counted, and the lengths of elongated fixed objects were measured. Banks and embankments were included if their slopes exceeded 4:1. The results of the fixed-object surveys before and beyond the sites are given in Tables 2 and 3.

There were approximately five spot fixed objects in the 0.16-km area before the crash and comparison sites and an equal number in the area beyond the sites. No significant difference was found between the number of spot fixed objects at the two types of sites. The comparatively low number of spot fixed objects at both sites reflects the generally clear nature of New Mexico roadsides and the fact that 90 percent of the crash sites were in rural areas.

The principal type of elongated fixed object at both crash and comparison sites was the embankment, which accounted for 55 percent of the length of elongated objects at crash sites and 45 percent of the corresponding length at comparison sites. Significant differences in the length of embankments were found between the two types of sites in the

Table 2. Average number of spot fixed objects 161 m before and beyond crash and comparison sites by distance from the roadway.

Type of Fixed Object	Number of Objects							
	Crash Sites				Comparison Sites			
	0-3 m	3-6 m	6-9 m	Total	0-3 m	3-6 m	6-9 m	Total
Before Site								
Luminaire poles	0	0	–[a]	–[a]	–[a]	–[a]	0	–[a]
Utility poles	–[a]	0.1	0.1	0.2	0.1	0.1	0.1	0.3
Traffic signs	0.1	0.1	–[a]	0.2	0.1	0.1	–[a]	0.2
Trees	0.1	0.6	1.5	2.2	0.1	0.4	1.2	1.7
Other	0.8	0.9	0.7	2.4	0.3	1.5	0.9	2.7
Total	1.0	1.7	2.3	5.0	0.6	2.1	2.2	4.9
Beyond Site								
Luminaire poles	0	–[a]	–[a]	–[a]	–[a]	–[a]	0	–[a]
Utility poles	–[a]	–[a]	0.1	0.1	–[a]	0.1	0.1	0.2
Traffic signs	0.2	0.1	–[a]	0.3	0.2	0.1	–[a]	0.3
Trees	0.1	1.2	1.0	2.3	0.2	0.7	0.9	1.8
Other	0.7	0.4	0.4	1.5	0.4	1.4	1.0	2.8
Total	1.0	1.7	1.5	4.2	0.8	2.3	2.0	5.1

[a]Less than 0.05 but greater than zero.

Table 3. Average length of elongated fixed objects 161 m before and beyond crash and comparison sites by distance from the roadway.

Type of Fixed Object	Length of Objects (m)							
	Crash Sites				Comparison Sites			
	0-3 m	3-6 m	6-9 m	Total	0-3 m	3-6 m	6-9 m	Total
Before Site								
Banks	2.2	12.0	13.9	28.1	0.9	6.5	12.8	20.2
Curbs	6.0	0.3	0.2	6.5	9.2	0.8	1.0	11.0
Ditches	11.5	19.1	15.1	45.7	18.6	21.3	17.6	57.5
Embankments	43.8	43.3	32.6[a]	119.7	37.0	41.2	20.4	98.6
Guardrail	0.7	0.3	0.2	1.2	1.9	0.1	0	2.0
Median barriers	0.6	0.9	1.1	2.6	0	0	0	0
Other	5.0	6.3	11.3	22.6	3.6	7.6	13.0	24.2
Beyond Site								
Banks	2.5	10.5	12.3	25.3	3.6	10.8	15.0	29.4
Curbs	5.9	0.8	0.5	7.2	6.8	0.7	0.9	8.4
Ditches	13.8	15.8	15.7	45.3	21.5	18.4	17.6	57.5
Embankments	50.5[a]	52.8[a]	36.0[a]	139.3[a]	35.2	36.9	20.3	92.4
Guardrail	2.4	0.3	0.6	3.3	1.9	0	0	1.9
Median barriers	0	0	0	0	0	0.1	–[b]	0.1
Other	7.9[a]	6.9	11.7	26.5	2.2	6.4	13.5	22.1

[a]Significantly higher than comparison site at $\alpha = 0.05$.
[b]Less than 0.05 but greater than zero.

0.16-km area beyond the sites. For the crash sites, guardrails were more common in the area beyond and less common in the area before the crash sites, although neither difference was significant. The category "other" for elongated fixed objects (e.g., bridge rails) within 3 m of the roadway in the 0.16-km section beyond the site had a significantly greater length at crash sites.

The findings related to embankments led to a more detailed study of embankment characteristics at one-third of the study sites. The sites for detailed study were chosen on the basis of their alignment characteristics. The set consisted of all rural sites on paved roads where the average curvature exceeded 2.5° or the average gradient was less than -2 percent at either the crash or comparison site. Cross-sectional measurements, including data sufficient to calculate shoulder width and slope, front and back slope, and embankment length and depth, were made at crash and comparison sites and 30 m before and beyond these sites (see Figure 4). Analyses of these data, given in Table 4, indicate that at crash sites the front slope and the depth of the embankment or ditch were significantly greater than at comparison sites. At a point 30 m beyond the crash site, the front slope and embankment length and depth were all significantly greater than for the corresponding location at the comparison site. For the comparatively few locations that had a back slope, the mean value was significantly higher at the crash site than at the comparison site.

Although the front slopes were significantly steeper at crash sites than at comparison sites, only 18 percent of crash sites had slopes in excess of 3:1. This slope is part of the normally accepted criteria for the installation of guardrails (4), under the hypothesis that impact with a guardrail would be more severe to vehicle occupants than the consequences of driving down a relatively flat slope.

Other parameters that were measured at the crash and comparison sites were the number of lanes, intersections, and driveways; pavement and shoulder widths; pavement friction; and posted speed limits. There were no significant differences in these characteristics between the crash and comparison sites.

Information from the reports of investigating officers and measurements at the sites were used to determine other characteristics of the crashes. Based on officers' sketches of the crash sites, it was determined that 24 percent of the vehicles actually overturned on the opposite side of the road from where they initially departed. Figure 5 shows the method of departure involved in these crashes. The longitudinal and lateral distances traveled off the roadway were also evaluated. Longitudinal distances varied from 0 to 141 m, and the mean value was 24 m. The 85th percentile longitudinal distance was 50 m. Lateral distances ranged from 0 to 91 m and averaged 5.3 m. It was found that 85 percent of the vehicles overturned within 8.2 m of the roadway, a value that is comparable to the often-quoted "9-m clear roadside".

The principal vehicle types involved in fatal overturning crashes were passenger cars (50 percent) and pickup trucks (37 percent). The involvement of pickup trucks is unusually high, since they account for only 18-20 percent of vehicle registrations and kilometers of travel in New Mexico.

COMPARISON OF NEW MEXICO AND GEORGIA DATA

During the same time that this study was being conducted in New Mexico, an identical study was performed in Georgia (see the paper by Wright and Zador in this Record). The Georgia study investigated the sites of 214 fatal overturning crashes, approximately 17 percent of the fatal crashes in that state during the 12-month study period. By using t-tests, comparisons were made between the data from the two states.

Table 5 summarizes the average values of the alignment characteristics at the New Mexico and Georgia sites. With respect to the crash sites, the Georgia data had significantly higher values of maximum and average curvature, maximum superelevation, and maximum gradient. Average crash-site gradients in both states were negative, but the New Mexico gradients were significantly steeper. There were also some significant differences with respect to the comparison sites, where the maximum values of

Figure 4. Locations of cross-sectional measurements.

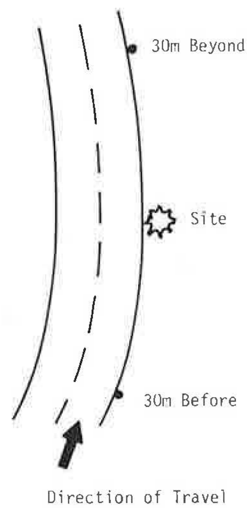


Table 4. Average cross-sectional characteristics at crash and comparison sites with embankments.

Characteristic	30 m Before	Site	30 m Beyond	Mean[a]
Shoulder width (m)				
Crash	2.5	2.1	1.9	2.2
Comparison	2.3	2.2	2.2	2.2
Shoulder slope (%)				
Crash	4.8	5.2	5.4	5.1
Comparison	5.2	5.2	5.4	5.3
Front slope (%)				
Crash	16.1	22.7[b]	24.9[b]	21.0
Comparison	15.1	14.8	17.6	16.5
Back slope (%)				
Crash	48.4	46.0	48.9	47.5[b]
Comparison	27.3	29.4	26.9	25.7
Embankment depth (m)				
Crash	1.1	1.5[b]	1.7[b]	1.4[b]
Comparison	0.9	0.9	0.9	0.9
Embankment length (m)				
Crash	12.6	12.7	12.9[b]	12.4
Comparison	10.6	11.4	10.2	10.8

[a]Average of three measurements at each site.
[b]Significantly higher than comparison site at $\alpha = 0.05$.

Figure 5. Vehicle departures from the roadway in overturning crashes: percentage of departures versus alignment at crash site.

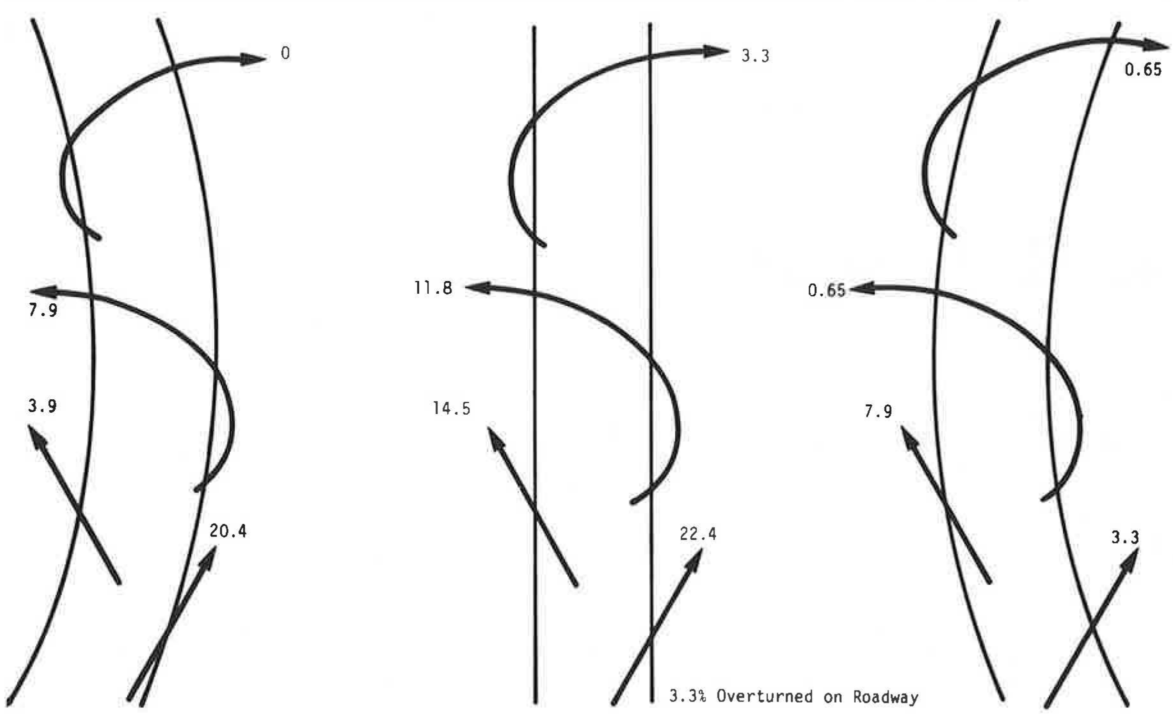


Table 5. Average alignment characteristics for New Mexico and Georgia sites.

Characteristic	Crash Sites		Comparison Sites	
	New Mexico	Georgia	New Mexico	Georgia
Curvature (°)				
Maximum	3.7	6.3[a]	1.9	3.2[a]
Minimum	0.2	0.2	0.1	0.1
Avg	1.7	2.3[a]	0.7	1.1
Critical-area avg[b]	1.9	2.9[a]	0.7	1.0
Superelevation (%)				
Maximum	4.1	5.0[a]	3.2	4.4[a]
Minimum	0.2	0.1	0.5	0.6
Avg	2.3	2.7	1.9	2.6[a]
Gradient (%)				
Maximum	0.4	1.9[a]	0.8	1.8[a]
Minimum	-2.2	-2.0	-1.5	-2.2[a]
Avg	-0.9	-0.2[a]	-0.3	-0.3
Critical-area avg[c]	-1.0	-0.3[a]	-0.4	-0.4

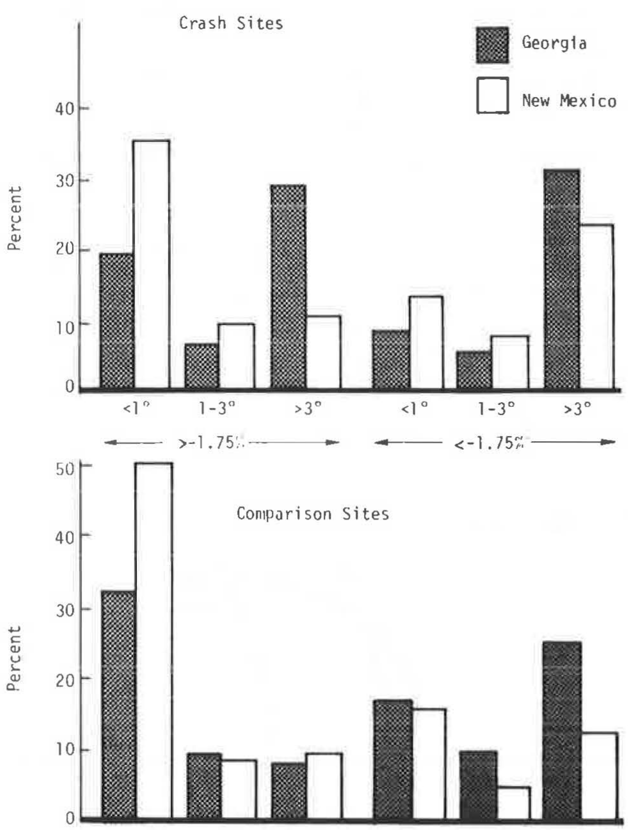
[a]Significant difference between New Mexico and Georgia at $\alpha = 0.05$.
[b]Average curvature from 137 m before site to 15 m beyond site.
[c]Average gradient from 122 m before site to 30 m beyond site.

Table 6. Georgia/New Mexico ratio of average number of spot fixed objects by distance from the roadway.

Type of Fixed Object	Crash Sites 0-3 m	Crash Sites 3-6 m	Crash Sites 6-9 m	Comparison Sites 0-3 m	Comparison Sites 3-6 m	Comparison Sites 6-9 m
Before Site						
Utility poles	7.4	2.4	4.0[a]	3.1	3.2[a]	2.5[a]
Traffic signs	5.8[a]	2.4[a]	3.8[a]	3.8[a]	1.8	5.6[a]
Trees	1.8	5.9[a]	4.9[a]	2.5	5.8[a]	3.6[a]
All spot objects	1.8	2.8	3.4	2.4	1.5	2.2
Beyond Site						
Utility poles	1.7	3.3[a]	3.4[a]	4.0	2.8[a]	5.6[a]
Traffic signs	3.0[a]	5.5[a]	2.3	2.5[a]	3.0[a]	4.7[a]
Trees	5.1	1.2	4.5[a]	1.4	2.3	3.9[a]
All spot objects	1.8	2.6	3.4	1.6	1.1	2.2

[a]Average number of objects significantly higher in Georgia at $\alpha = 0.05$.

Figure 6. Comparison of maximum curvature and minimum gradient at Georgia and New Mexico sites.



curvature, superelevation, and gradient were all higher in Georgia.

Only one of the Georgia crash sites, versus 21 of the New Mexico crash sites, was on a lengthy downgrade that extended for at least 1.6 km in the direction of the comparison site. When data for these sites were removed from the analysis, the gradient results for the two states stayed virtually identical. The average New Mexico curvature and superelevation values were not significantly influenced by the deletion of these data, whereas all of the gradient characteristics (maximum, minimum, and average) increased by approximately 0.3 percent. The significant differences identified in Table 5 were still present when comparisons were made between New Mexico and Georgia sites that were not on lengthy downgrades.

Six conditions of combined horizontal and vertical alignment were used to compare the data from Georgia and New Mexico. The results are shown in Figure 6. Chi-square testing showed that the condition classification, for both crash and comparison sites, was not independent of the state. The predominant characteristic that differentiated the two states was the extreme values of curvature. The New Mexico sites were overrepresented with respect to low curvature and were underrepresented with respect to high values of horizontal curvature. The states were quite similar for sites with curvature between 1° and 3°.

A general comparison of spot fixed objects along the roadside indicated that Georgia had 2.8 times as many objects at crash sites and 1.8 times as many objects at comparison sites. On the other hand, the length of continuous fixed objects was approximately 10 percent greater in New Mexico.

The average number of spot objects was compared--by object type, site type, distance from the pavement, and location (before or beyond the site)--between New Mexico and Georgia. Table 6 gives the Georgia/New Mexico ratio of average number of spot fixed objects. The categories of luminaire poles and other fixed objects are not included in the table because there was no significant difference in the two categories. The most substantial difference between the two states was for the category of trees, which constituted 49 and 68 percent of the crash-site spot fixed objects in New Mexico and Georgia, respectively.

A different pattern was found when a comparison was made between New Mexico and Georgia data for the length of continuous fixed objects. At both crash and comparison sites, Georgia had significantly more guardrails and ditches and significantly fewer embankments. The length ratios are given in Table 7. The differences were primarily attributable to the differences between the states rather than to the distinction between crash and comparison sites.

A detailed comparison of the roadside data from a portion of the New Mexico and Georgia sites revealed significant differences between the two states at both crash and comparison sites. When the sites were classified with respect to their roadside characteristics, it was found that 79 percent of the New Mexico crash sites had embankments versus only 52 percent of the Georgia crash sites. Values for specific cross-sectional features are compared in Table 8. The average percentages for front slopes

Table 7. Georgia/New Mexico ratio of average number of elongated fixed objects by distance from the roadway.

Type of Fixed Object	Crash Sites 0-3 m	Crash Sites 3-6 m	Crash Sites 6-9 m	Comparison Sites 0-3 m	Comparison Sites 3-6 m	Comparison Sites 6-9 m
Before Site						
Bank	0.76	1.5	0.80	1.0	1.7	1.1
Ditch	1.0	2.2[a]	0.86	0.89	2.0[a]	0.80
Embankment	0.40[b]	1.2	0.36[b]	0.24[b]	0.92	0.68
Guardrail	6.4[a]	10.8[a]	1.2	1.4	36.1[a]	–[c]
All continuous objects	0.60	1.5	0.61	0.59	1.3	0.76
Beyond Site						
Bank	0.05[b]	1.1	0.79	0.68	1.4	1.2
Ditch	0.65	3.0[a]	0.75	0.61	2.3[a]	0.87
Embankment	0.31[b]	0.83	0.39[b]	0.29[b]	1.1	0.73
Guardrail	1.7	8.8[a]	0.41	1.2	–[a,c]	–[c]
All continuous objects	0.48	1.3	0.62	0.53	1.52	0.82

[a]Average length in Georgia significantly greater than in New Mexico at $\alpha = 0.05$.
[b]Average length in New Mexico significantly greater than in Georgia at $\alpha = 0.05$.
[c]Average guardrail length in New Mexico = 0.00 m.

Table 8. Average cross-sectional characteristics of Georgia and New Mexico sites with embankments.

Characteristic	Georgia Sites Crash	Georgia Sites Comparison	New Mexico Sites Crash	New Mexico Sites Comparison
Shoulder width (m)	1.9	1.7	2.2	2.2[a]
Shoulder slope (%)	4.8	6.3	5.1	5.8
Front slope (%)	33.4[b]	26.6[b]	21.0	16.5
Back slope (%)	36.1	31.9	39.4	25.4
Embankment depth (m)	0.6	0.5	1.4[a]	0.9[a]
Embankment length (m)	7.2	7.3	12.4[a]	10.8[a]

[a]Average value in New Mexico significantly higher at $\alpha = 0.05$.
[b]Average value in Georgia significantly higher at $\alpha = 0.05$.

Table 9. Other general characteristics of Georgia and New Mexico sites.

Characteristic	Georgia Sites Crash	Georgia Sites Comparison	New Mexico Sites Crash	New Mexico Sites Comparison
Number of lanes	2.3	2.5	2.7[a]	2.7[a]
Pavement width (m)	7.0	7.4	9.7[a]	10.0[a]
Number of intersections	0.4[b]	0.4[b]	0.2	0.2
Number of driveways	1.3[b]	1.5[b]	0.5	0.7
Longitudinal distance (m)	33.8[b]		24.0	
Lateral distance (m)	5.7		5.3	
Downgrade distance (m)	0.16		0.64[a]	

[a]Average value in New Mexico significantly higher at $\alpha = 0.05$.
[b]Average value in Georgia significantly higher at $\alpha = 0.05$.

at both crash and comparison sites were significantly higher in Georgia. However, embankment length and depth were significantly greater in New Mexico. Average shoulder widths for sites that had shoulders were higher in New Mexico, although the difference was significant only at the comparison sites. Data from the two states were used to plot values for front slope versus embankment depth. Whereas the New Mexico data showed a consistently lower limit for the slope of 10.5 percent per meter of depth, there was no discernable relation between these parameters for the Georgia data.

Other characteristics of the crash and comparison sites are summarized in Table 9. The number of lanes was significantly higher in New Mexico at both crash and comparison sites. The number of intersections and driveways was significantly higher at both types of sites in Georgia. The average longitudinal distance traveled off the roadway by overturning vehicles was significantly greater in Georgia, although there was no significant difference in the lateral distance.

DISCUSSION OF RESULTS

The roadway and roadside data at the sites of fatal overturning crashes in New Mexico clearly show that the conditions at these sites were more adverse than at a systematically chosen set of comparison sites. The most dramatic difference was with respect to roadway curvature. Although it is not possible to specify an exact value of curvature that separates safe and hazardous conditions, values of maximum curvature in excess of 5° occurred at crash sites at twice the expected rate. Curves to the left were more frequent and sharper at crash sites. Roadway gradients at the sites of fatal overturning crashes were shown to be significantly steeper downgrades than at comparison sites. The difference was especially apparent for downgrades of less than -2 percent, which were 40 percent more common at crash sites. Although curvature was more significant than gradient in distinguishing between crash and comparison sites, left curves on steep downgrades were twice as frequent at the crash sites.

These alignment characteristics can serve as preliminary screening criteria for the determination of roadway locations that need correction. They appear to be the principal roadway factors that contribute to a vehicle running off the road. However, roadside features were found to influence the probability of overturning for a run-off-the-road vehicle. Although front-slope values, which averaged approximately 4:1 in the area immediately downstream from the crash sites, might not be judged critical by current engineering standards, the evidence clearly indicates that vehicles that departed the traveled way had serious difficulty in traversing such a slope. Current standards for guardrail use do not specify the use of guardrails on embankments that are less than 1.2 m in height, despite the fact that more than half of the fatal overturning crashes occurred where embankment heights were less than this value; other data (5) also indicate that approximately 60 percent of the run-off-the-road crashes involve low embankments and shallow ditches. Analysis of the New Mexico data suggests that, according to current standards (4), guardrails are warranted at less than 15 percent of the sites of fatal overturning crashes. This finding attacks the merits of guardrail warrants, especially in view of the relative severity of overturning and guardrail crashes. Although the current guardrail standards do not assume that slopes of 3:1 or embankments less than 1.2 m high are traversible, they do assume that the occupants of a vehicle that is under control will experience less injury in negotiating a side slope or an embankment than in colliding with a guardrail. The theoretical analyses (6) and field studies (7) on which this assumption is based should be reexamined.

Fatal overturning crashes accounted for a lower percentage of all crashes in Georgia than in New Mexico. The more extensive use of guardrails in Georgia is one factor that partly explains this difference. It is recognized that there may be other traffic and demographic factors that contribute to the variation in frequency of overturning crashes between the states. However, the Georgia data show significantly more adverse horizontal alignment conditions than those found in New Mexico. On the other hand, downgrades were significantly more common at New Mexico sites.

The roadsides in Georgia had significantly more spot fixed objects than those in New Mexico. This fact is supported by other data (8,9) that show that 29 percent of Georgia's fatal accidents involve fixed objects whereas the comparable figure in New Mexico is less than 11 percent. The fatal-accident data indicate that, for vehicles that have left the roadway, crashes in Georgia are more likely to involve fixed objects whereas those in New Mexico are much more likely to involve overturning. The difference is attributable not only to the number of spot fixed objects but also to the extent and height of embankments.

Although the findings of this study offer some guidance for the selection of hazardous locations in New Mexico, the significant differences found between Georgia and New Mexico suggest that other roadway, traffic, and environmental factors need to be considered in the development of a priority scheme for nationwide application. A project is under way to coalesce the results of these studies into a model for establishing priorities for improving locations where there is a potential for overturning crashes. There may have to be different criteria among the states for assessing the level of hazards.

ACKNOWLEDGMENT

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REFERENCES

1. J.W. Hall. Characteristics of Crashes in Which a Vehicle Overturns. TRB, Transportation Research Record 757, 1980, pp. 41-45.
2. P.H. Wright and L.S. Robertson. Amelioration of Roadside Obstacle Crashes. Transportation Engineering Journal, Proc., ASCE, Vol. 105, No. TE6, Nov. 1979, pp. 609-622.
3. P.H. Wright and L.S. Robertson. Priorities for Roadside Hazard Modification. Insurance Institute for Highway Safety, Washington, DC, March 1976.
4. Guide for Selecting, Locating, and Designing Traffic Barriers. AASHTO, Washington, DC, 1977.
5. K. Perchonok and others. Hazardous Effects of Highway Features and Roadside Objects. FHWA, Rept. FHWA-RD-78-202, Sept. 1978.
6. H.E. Ross, Jr., and others. Warrants for Guardrails on Embankments. HRB, Highway Research Record 460, 1973, pp. 85-96.
7. J.D. Glennon and T.N. Tamburri. Objective Criteria for Guardrail Installation. HRB, Highway Research Record 174, 1967, pp. 184-206.
8. Fatal Accident Reporting System. National Highway Traffic Safety Administration, U.S. Department of Transportation, Annual Rept., 1978.
9. R.L. Lee. Fixed-Object Fatal Accidents. FHWA, Jan. 1980.

Study of Fatal Rollover Crashes in Georgia

PAUL H. WRIGHT AND PAUL ZADOR

Engineering surveys were performed at 214 locations in Georgia where single-vehicle fatal rollover crashes occurred over a one-year study period. Similar surveys were made at comparison locations 1.6 km (1 mile) upstream from the crash locations. The most prominent roadway feature associated with fatal rollover crashes in Georgia was horizontal curvature, particularly along left curves. It was found that fatal rollover crash locations can be discriminated from comparison locations by curvature greater than 6°, the same value suggested in the fixed-object studies. Steep gradients were also found to be strongly and significantly associated with rollover crash locations. The pattern of distribution of longitudinal slopes observed in earlier studies of fixed-object crashes, in which negative slopes tended to occur upstream and positive slopes downstream, was also apparent at rollover crash locations. Rollover sites were characterized by significantly larger changes in lateral slope at the shoulder edge than were found at comparison sites. The rollover sites were also more likely than the comparison sites to have embankments along the roadside but less likely to have trees and certain other spot fixed objects. Similarly, the rollover crash sites had longer embankments, banks, and ditches than were found at fixed-object crash sites. On the other hand, more trees, poles, and signs were found at the fixed-object crash sites than at the rollover crash sites.

Vehicle rollover is one of the leading causes of death in single-vehicle crashes. According to an estimate obtained from the U.S. Department of Transportation Fatal Accident Reporting System (FARS), in 1978 and 1979, 46 percent of the passenger cars in fatal single-vehicle crashes rolled over. Little research has been performed on possible contributions of the roadway to the occurrence and severity of such crashes.

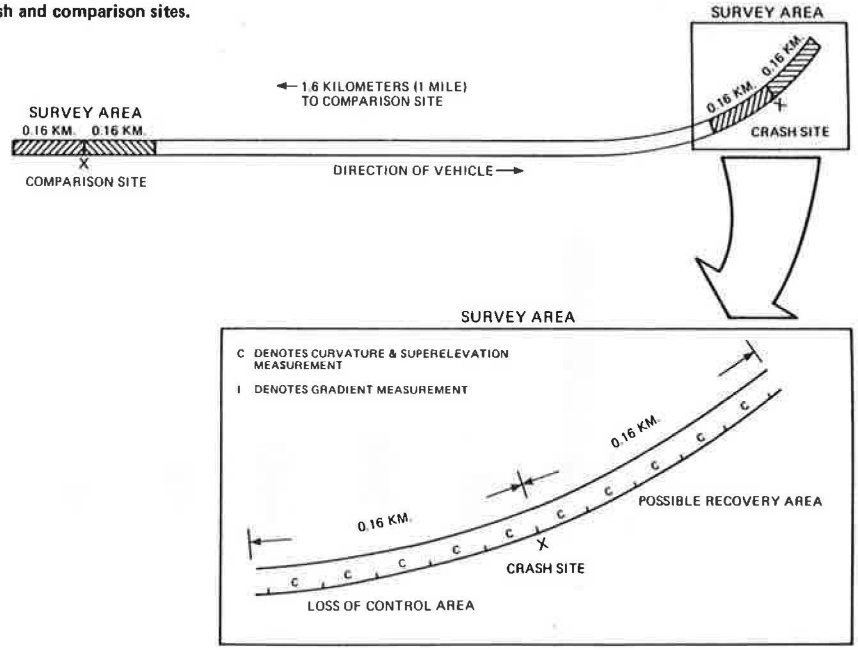
The objective of the study described in this paper was to identify distinctive roadway characteristics at locations in Georgia where fatal rollover crashes occurred and to develop guidelines for the reduction or elimination of such crashes by modifying roadway and/or roadside features. A companion study, described in the paper by Hall and Zador in this Record, was undertaken in New Mexico.

The study described here is the third in a series relating single-vehicle crashes in Georgia to roadway and/or roadside characteristics. The first two studies (1,2) involved crashes of vehicles into fixed objects. One project focused on 300 fatal fixed-object crashes in 108 counties in Georgia during a 14-month period ending in April 1975 (1). The second project was a study of a general population of fixed-object crashes, including 7 fatal, 112 nonfatal injury, and 181 property-damage-only crashes, in a three-county area in north Georgia during a five-month period in 1977 and 1978 (2). These two studies, and the one described here, were based on surveys of geometric design features and an inventory of roadside obstacles at both crash and noncrash sites.

BACKGROUND

FARS provided general statistics on the circumstances and conditions associated with fatal rollover crashes. These statistics revealed that, for fatal single-vehicle rollover crashes throughout the United States in 1978, 43.5 percent occurred along roadways with curved alignment, 34.3 percent oc-

Figure 1. Hypothetical crash and comparison sites.



curred along roadways with gradient, 87.5 percent occurred along two-lane roadways, 86.1 percent occurred where the roadway surface was reported to be dry, and 9.5 percent occurred where inclement weather or adverse atmospheric conditions were identified.

METHOD

This study was designed to compare roadway characteristics at two groups of sites: sites where one or more vehicle occupants died in a rollover crash and sites 1.6 km (1 mile) away that the vehicle was likely to have passed prior to reaching the site of the fatal crash. Differences between the two groups of sites can be used to identify roadway and/or roadside features where fatal rollover crashes are more likely to occur. Virtually all of the locations of fatal single-vehicle rollover crashes that occurred in Georgia during a 12-month period ending in July 1979 were included in this study.

The study area included a variety of land uses (rural, suburban, and urban), roadway types, and topography. Police reports of fatal rollover crashes were routinely mailed to the research team by the Georgia State Patrol. A total of 223 crashes were identified, but 9 were eliminated because of difficulties in locating or collecting data at the sites.

Engineering surveys were made, usually by three-person teams, at 214 fatal crash locations and at 214 comparison locations. The surveys were confined to a 0.3-km (0.2-mile) section at each of the locations. The measurements were referenced to the point at which the rollover of the vehicle commenced. A point along the roadway edge immediately adjacent to the reference rollover point was identified as the "crash site". As Figure 1 shows, a point 1.6 km upstream (i.e., away from the crash site, in the direction from which the vehicle traveled) was designated as the "comparison site". In locating comparison sites, turn choices at T- or Y-intersections were made randomly (by flip of a coin).

Measurements of curvature and superelevation were made beginning 15 m (50 ft) from the crash and comparison sites and at 30-m (100-ft) intervals for 137 m (450 ft) both upstream and downstream from these sites. The gradient was measured every 30 m for 152 m (500 ft) both upstream and downstream from the sites.

A 30-m cloth tape was used for measuring distances. Horizontal curvatures were measured by the middle ordinate method. The curve measurements were usually taken on the edge of the roadway. The middle ordinates were converted to degrees of curvature of the centerline of the roadway. Superelevation and gradients were measured at the center of the side of the road used by the driver in approaching the crash location. Those measurements were made with a specially designed instrument consisting of a 1.2-m (4-ft) carpenter's level with an adjusted calibrated leg. On Interstate highways, curvature, superelevation, and gradient data were taken from plan and profile sheets.

At a subsample of 48 locations, side slopes and other elements of the cross section were carefully measured with a cloth tape, hand level, and level rod. This subsample was chosen to include a pair of crash and comparison sites for which either or both locations had sharp curvature and steep negative gradient. The subsample included all cases for which the curvature exceeded 6° and the gradient was negative and steeper than 2 percent at both the crash and comparison locations. The subsample also included half of the cases where these criteria were satisfied at either the crash or comparison location and all of the remaining cases where the curvature exceeded 4° and the gradient was negative and greater than 1 percent at both locations.

Inventories were taken of various types of fixed objects in 3-m (10-ft) segments of a 9-m (30 ft) border for 161 m (0.1 mile) in each direction from the crash and comparison sites. In addition, type of road, number of lanes, and widths of pavement and shoulder were recorded.

Pavement skid resistance was measured at approximately half of the crash and comparison sites by pulling a 32-kg (71-lb) lead block, mounted on small rubber shoes, along the roadway and measuring the resistance by means of a spring scale.

The data-collection procedures used in this study were essentially the same as those used in the

Figure 2. Distribution of maximum road curvature at sites of fatal rollover crashes and at comparison sites.

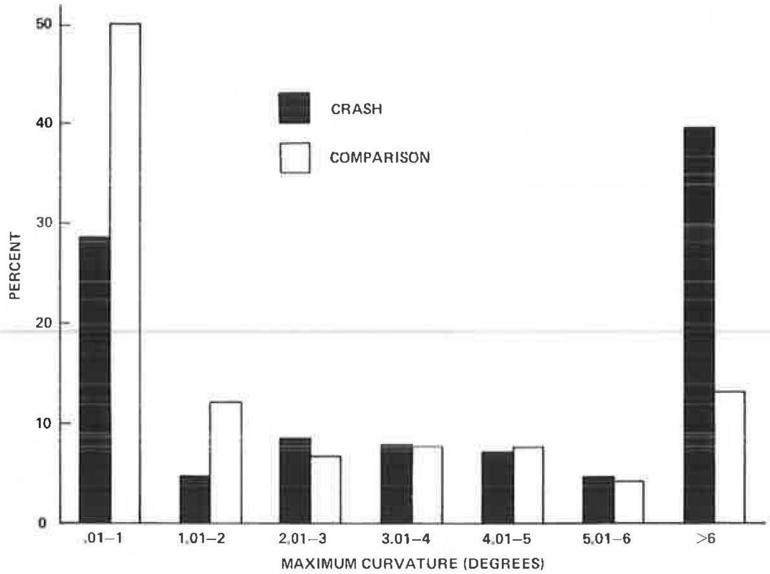
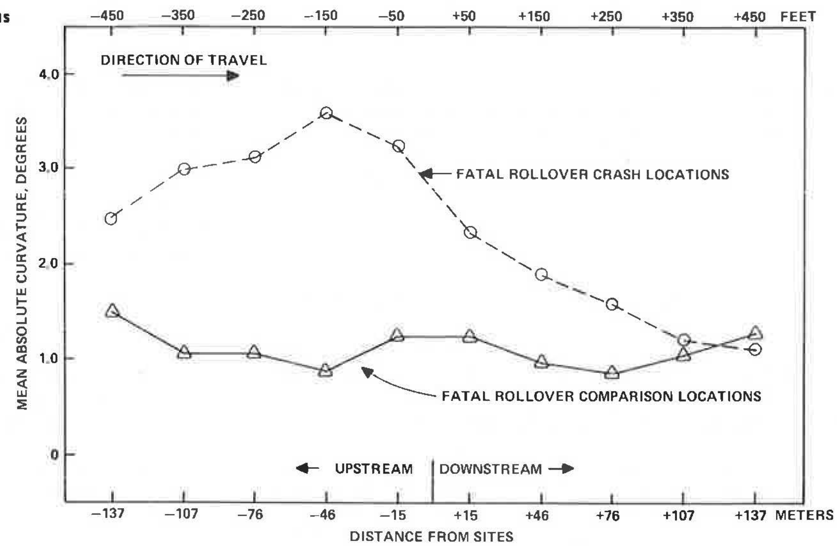


Figure 3. Mean degree of curvature observed at various section positions at crash and comparison sites.



earlier studies of single-vehicle collisions with roadside obstacles (1,2).

RESULTS

Curvature

The largest difference between the crash and comparison sites was in road curvature. Approximately 40 percent of the crash sites had a maximum curvature greater than 6° whereas only 13 percent of the comparison sites had a maximum curvature greater than 6° (see Figure 2). At half of the comparison sites, but at only 28 percent of the crash sites, the roadway was straight or had negligible curvature (degree of curve $\leq 1°$). The difference in distribution of curvature between the crash and comparison locations shown in Figure 2 could not commonly occur from chance fluctuations in sampling ($X^2 = 218.5$, df = 6, $p < 0.001$).

The curvature usually occurred near the crash site or upstream. The largest differences in curvature occurred in the area from 107 m (350 ft) upstream to 15 m (50 ft) downstream from the sites. The maximum curvature tended to occur at a point located 46 m (150 ft) upstream from the crash site, as Figure 3 shows. This is reasonable, since horizontal curvature places heavier demands on drivers and increases the likelihood of a driver losing control of a vehicle.

The pattern of distribution of mean curvatures with station location was similar to that found in the earlier study of fatal fixed-object crashes (see Figure 4). The mean curvature values for the fixed-object crash locations were generally higher than for the rollover crash locations, and Student's t-test indicated that the differences were significant at the 5 percent level for four locations: 46 m upstream and 76, 107, and 137 m downstream (150 ft upstream and 250, 350, and 450 ft downstream).

Table 1 gives a distribution of the fatal rollover and fixed-object crashes by general type of alignment and direction of vehicle departure from the roadway. The distribution shows a marked tendency for vehicles in rollover crashes to leave the roadway along left-turning curves and, among these curves, vehicles leaving the roadway on the outside (or right side) are overrepresented. Among crashes in which the vehicle left a straight road section on the left side, there were more off-the-

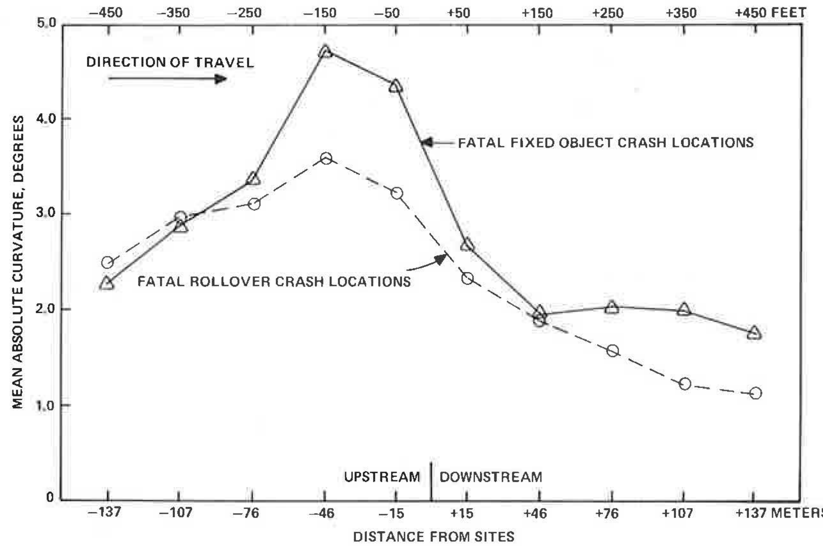


Figure 4. Mean degree of curvature observed at various section positions at sites of fixed-object crashes and rollover crashes.

Table 1. Distribution of fatal crashes by type of alignment and direction of vehicle departure from roadway.

Roadway Alignment	Side of Road on Which Vehicle Crashed	Percentage of Crashes Observed: Fixed-Object Study	Percentage of Crashes Observed: Rollover Study
Straight	Left	11.2	16.9
	Right	15.7	15.5
Curve to right	Left	20.2	8.9
	Right	7.3	7.0
Curve to left	Left	15.3	15.0
	Right	30.3	23.5
Not specified	On road	-	13.2

road rollovers than fixed-object crashes. For vehicles that crashed along the left side of right-turning curves and the right side of left-turning curves, a greater percentage of fixed-object crashes than rollover crashes was found.

Lateral Slope

The mean lateral slopes of the traveled lanes are shown in Figure 5 for each position at the crash and comparison locations. The data shown represent both superelevation values (for curved roadways) and crown values (for straight roadways). The slightly higher mean values noted in the upstream area reflect the superelevation commonly provided for the curves that tend to occur in the areas approaching the crash sites; these differences were statistically significant ($p < 0.085$). The lateral slopes tended to be greater at the locations of fixed-object crashes than at the locations of rollover crashes, but in only two instances--at 107 and 137 m (350 and 450 ft) downstream--were the differences significant.

Gradient

Figure 6 shows the pattern of variation of mean gradients for crash and comparison locations. The apparent differences in mean gradients were tested for each of the 11 positions by using t-tests. None of the differences was found to be significant at the 5 percent level.

The finding of steeper downhill slopes at comparison sites than at crash sites prompted further analysis of these data. Table 2 gives the percentages of rollover crash sites that have various combinations of average curvature and gradient in a 91-m (300-ft) section immediately upstream from these sites. The comparable percentage distribution for the opposite sides of these road sections is also given. The opposite-side percentages were obtained by reversing "left" and "right" for curvature and "uphill" and "downhill" for gradient. For each curvature range, there were more downhill crashes than crashes on the opposite side of the road. Since a crash could have taken place on either side of the roadway, these results show that crashes were more common on downhill than on uphill road segments with the same curvature.

Differences in the gradients at locations of fatal fixed-object and rollover crashes were not significantly different. The patterns of distribution of gradients at the two classes of locations were remarkably similar; there were more negative slopes upstream of the sites and positive slopes downstream (see Figure 7).

Roadside

Measurements of eight key lateral dimensions or slopes along the roadside were made at 48 locations selected from the original set of 214 (50 locations were selected, but field survey teams were unable to perform surveys at 2 of them). In the vicinity of each crash and comparison site, the following measurements of the cross-sectional dimensions and slopes were made at stations 30 m (100 ft) upstream and downstream: shoulder width, shoulder slope, inside slope, back slope, depth of ditch, lateral distance from edge of shoulder to bottom of embankment, extent of drop-off at the pavement edge, and height of curb.

Twenty-four t-tests were made to compare each of the eight variables at each position in the crash vicinity with the corresponding variable and position at the comparison location. Mean values of these slopes and dimensions are given in Table 3. On the basis of two-tailed t-tests, significant differences ($p < 0.10$) were noted for five of the tests:

1. The height of curb 30 m upstream was higher at the comparison location than at the crash location.

2. The shoulder slope at the comparison site was steeper than at the crash site.
3. The inside slope at the crash site was steeper than at the comparison site.
4. The shoulder slope 30 m downstream was steeper at the comparison location than at the crash location.
5. The inside slope 30 m downstream was steeper at the crash location than at the comparison location.

Of special interest in these findings about the roadside is the change in lateral slope at the edge of the shoulder. At the crash site, the mean change in lateral slope was 32.9 percent (37.5 - 4.6). At the comparison site, the mean change in slope was

Figure 5. Mean lateral slope observed at various section positions.

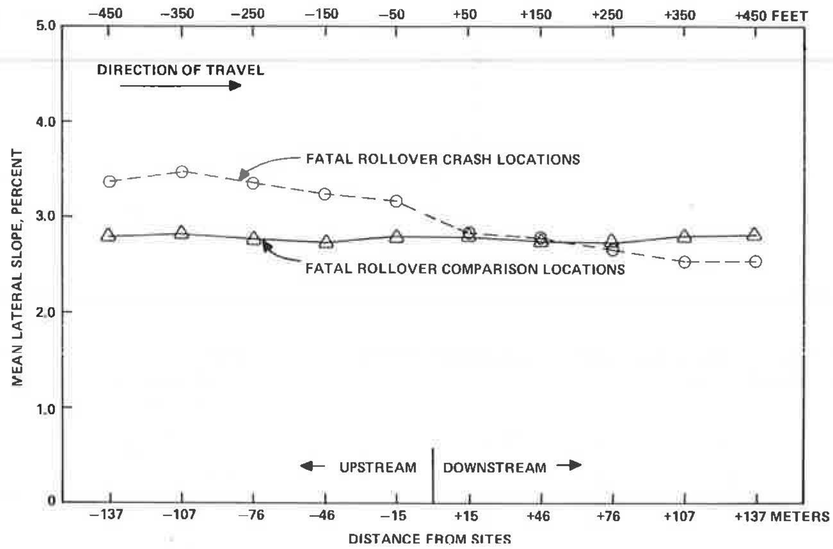


Figure 6. Mean gradient observed at various section positions at sites of rollover crashes and at comparison sites.

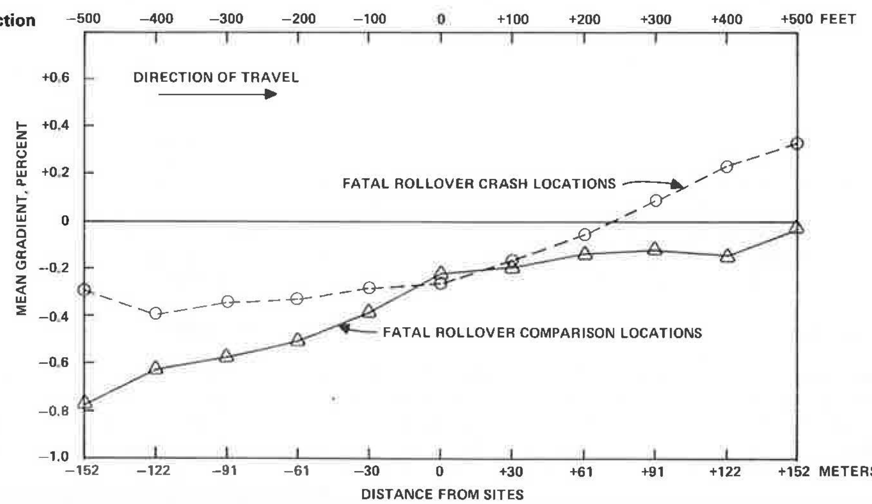


Table 2. Comparison of crash sites and opposite sides of road for various combinations of gradient and curvature.

	Upgrade (>+1.0%)			Nearly Level (≤+1.0% to ≥-1.0%)			Downgrade (<-1.0%)		
Curvature	Crash (%)	Opposite (%)	Ratio	Crash (%)	Opposite (%)	Ratio	Crash (%)	Opposite (%)	Ratio
Sharp right (≤-3.01°)	3.7	10.7	0.35	5.1	7.5	0.69	3.3	8.9	0.37
Gradual right (<-3.00° to <-0.1°)	2.8	6.1	0.46	2.8	7.9	0.35	3.3	2.8	1.17
Nearly tangent (≥-0.1° to ≤+0.1°)	7.9	10.7	0.74	16.4	16.4	1.00	10.7	7.9	1.35
Gradual left (>0.1° to ≤+3.00°)	2.8	3.3	0.86	7.9	2.8	2.83	6.1	2.8	2.17
Sharp left (≥+3.01°)	8.9	3.3	2.71	7.5	5.1	1.45	10.7	3.7	2.88

Figure 7. Mean gradient observed at various section positions at sites of fixed-object crashes and rollover crashes.

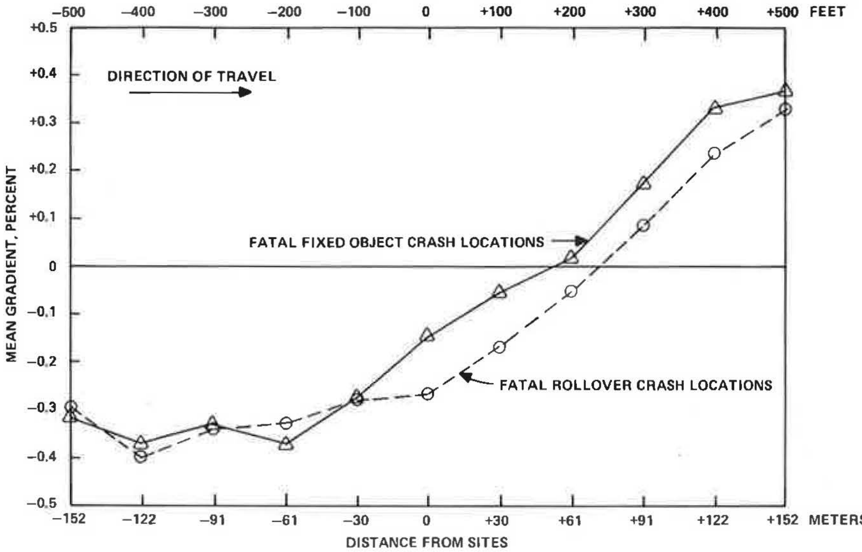


Table 3. Mean dimensions of roadside cross section at various locations.

Variable	30 m Upstream	At Site	30 m Downstream
Shoulder width (m)			
Crash	1.9	1.9	1.9
Comparison	1.7	1.8	1.7
Shoulder slope (%)			
Crash	5.2	4.6[a]	4.1[a]
Comparison	5.5	6.9[a]	6.6[a]
Inside slope (%)			
Crash	30.7	37.5[a]	38.9[a]
Comparison	28.2	28.9[a]	29.2[a]
Back slope (%)			
Crash	26.3	21.7	17.8
Comparison	21.5	13.5	20.1
Ditch depth (m)			
Crash	0.37	0.37	0.38
Comparison	0.36	0.36	0.35
Lateral embankment length (m)			
Crash	3.8	3.2	4.4
Comparison	3.3	3.9	3.6
Curb height (cm)			
Crash	0.70[a]	1.16	1.24
Comparison	2.48[a]	3.10	1.58
Drop-off at shoulder (cm)			
Crash	3.92	4.57	3.16
Comparison	2.95	3.31	4.17

Note: 1 m = 3.28 ft.
[a]Significantly different ($p < 0.10$, two-tailed).

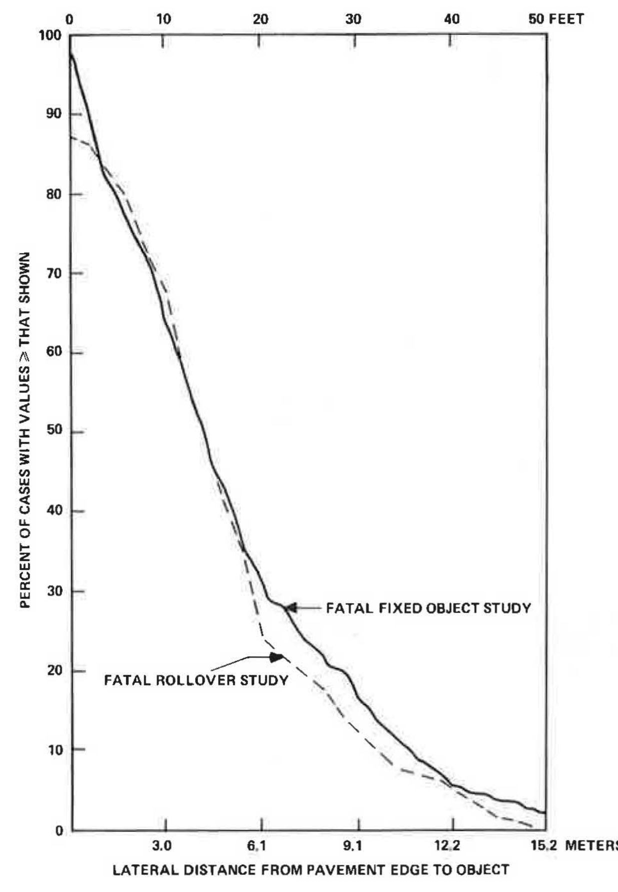
only 22.0 percent (38.9 - 6.9). Similar results were obtained in comparing the mean changes in slopes 30 m downstream.

As Figure 8 shows, about 90 percent of rollover crashes were precipitated at points within 9.1 m (30 ft) of the pavement edge. The distribution of lateral displacement of such points was similar to that for lateral distances to objectives struck in the fixed-object study. The average angle of departure was 9.6°, a value that compares favorably with encroachment angles reported by other researchers (3,4).

Roadside Objects

Tables 4 and 5 give the average numbers of "spot" obstacles and the lengths of elongated obstacles in 0.16-km (0.1-mile) sections upstream and downstream from rollover sites (crash and comparison) as well as at sites of fixed-object crashes. Hazard densities at the rollover crash sites were compared with

Figure 8. Distributions of lateral distance to crash point for studies of fatal fixed-object crashes and rollover crashes.



densities at both the rollover comparison sites and the fixed-object crash sites. The t-tests used showed that 8 among the 72 former and 25 among the 72 latter differences were statistically significant ($p < 0.10$); these differences are indicated in Table 4. The relatively few and small differences between single-vehicle crash and comparison sites in regard to hazard densities confirm the field observation that the placement and frequency of roadside

Table 4. Average number of spot potential hazards 161 m upstream and downstream of crash and comparison sites by distance from pavement.

Hazard	Rollover Crash Sites 0-3 m	Rollover Crash Sites 3-6 m	Rollover Crash Sites 6-9 m	Rollover Crash Sites Total	Rollover Comparison Sites 0-3 m	Rollover Comparison Sites 3-6 m	Rollover Comparison Sites 6-9 m	Rollover Comparison Sites Total	Fixed-Object Crash Sites 0-3 m	Fixed-Object Crash Sites 3-6 m	Fixed-Object Crash Sites 6-9 m	Fixed-Object Crash Sites Total
Upstream												
Trees	0.2	3.3	7.3	10.8	0.4	2.4	4.1	6.9	0.7[a]	2.7	3.9[a]	7.3
Utility poles	0.2	0.2	0.3	0.7	0.2	0.3	0.2	0.7	0.6[a]	0.4[a]	0.3	1.3
Traffic-signal posts	0.5	0.1	0.2	0.8	1.9	0.2	0.1	2.2	0.7	0.2[a]	0.1	1.0
Street luminary poles	--[b]	--[b]	--[b]	--	--[b]	--[b]	--[b]	--	0.1	--[b]	--[b]	0.1
Other narrow objects	0.8	0.7	0.3	1.8	0.5[a]	1.2	0.3	2.0	1.3	2.0	1.7	5.0
Total	1.7	4.3	8.1	14.1	3.0	4.1	4.7	11.8	3.4	5.3	6.0	14.7
Downstream												
Trees	0.6	1.5	4.5	6.6	0.3	1.7	3.7	5.7	1.0[a]	3.1[a]	4.9[a]	9.0
Utility poles	0.1	0.1	0.3	0.5	0.2	0.2	0.3	0.7	0.6[a]	0.4[a]	0.2	1.2
Traffic-signal posts	0.5	0.3	0.1	0.9	0.4	0.2	0.1	0.7	0.6	0.2	0.1	0.9
Street luminary poles	--[b]	--[b]	--[b]	--	--[b]	--[b]	--[b]	--	--[b]	--[b]	--[b]	--
Other narrow objects	0.6	2.4	0.4	3.4	0.5	0.5	0.2	1.2	1.4	1.7	1.5	4.6
Total	1.8	4.3	5.3	11.4	1.4	2.6	4.3	8.3	3.6	5.4	6.7	15.7

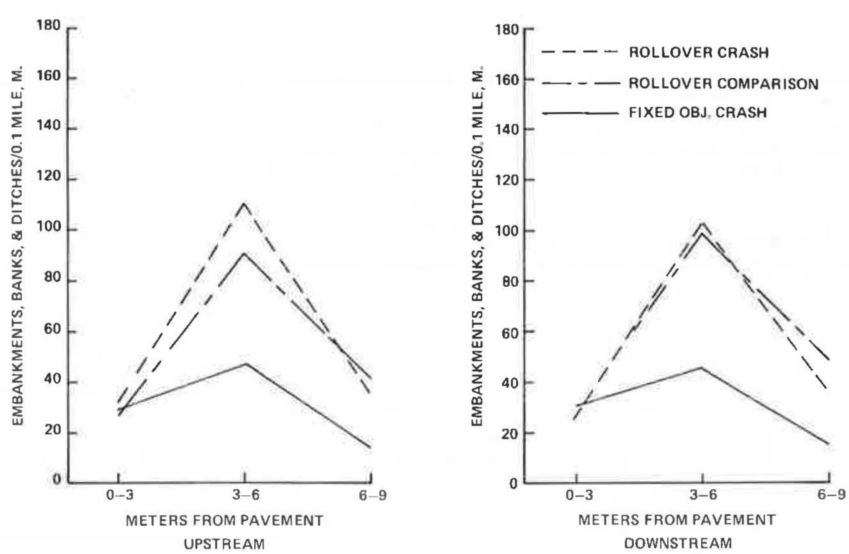
Note: 1 m = 3.28 ft. [a]<0.05 but not 0.00. [b]Significantly different from rollover crash site data ($p < 0.10$).

Table 5. Average number of elongated potential hazards 161 m upstream and downstream of crash and comparison sites by distance from pavement.

Hazard	Rollover Crash Sites 0-3 m	Rollover Crash Sites 3-6 m	Rollover Crash Sites 6-9 m	Rollover Crash Sites Total	Rollover Comparison Sites 0-3 m	Rollover Comparison Sites 3-6 m	Rollover Comparison Sites 6-9 m	Rollover Comparison Sites Total	Fixed-Object Crash Sites 0-3 m	Fixed-Object Crash Sites 3-6 m	Fixed-Object Crash Sites 6-9 m	Fixed-Object Crash Sites Total
Upstream												
Curbs	4.3	2.8	0.1	7.2	8.4	2.5	0.1	11.0	9.3[a]	1.7	0.6	11.6
Embankments	17.6	50.4	11.7	79.7	8.4[a]	37.9[a]	13.8	60.1	11.1[a]	19.2[a]	4.9	35.2
Banks and cuts	1.7	17.7	11.2	30.6	1.0	10.9[a]	13.9	25.8	4.6[a]	10.0[a]	4.6[a]	19.2
Ditches	11.5	42.1	12.9	66.5	16.5	42.5	14.1	73.1	13.0	18.3[a]	4.4[a]	35.7
Guardrails	4.3	3.7	0.2	8.2	2.7	3.0	0.2	5.9	3.3	3.5	0.4	7.2
Other	2.6	4.3	8.2	15.1	3.6	4.8	7.3	15.7	--	--	--	--
Total	42.0	121.0	44.3	207.3	40.6	101.6	49.4	191.6	41.3	52.7	14.9	108.9
Downstream												
Curbs	6.2	1.3	0.7	8.2	7.2	3.4	0.0	10.6	9.4	1.9	0.1	11.4
Embankments	15.9	43.9	14.0	73.8	10.4[a]	40.6	14.8	65.8	9.9[a]	18.7[a]	5.2[a]	33.8
Banks and cuts	0.1	11.7	9.7	21.5	2.5[a]	15.1	18.2[a]	35.8	5.0[a]	11.4	6.0[a]	22.4
Ditches	9.0	47.3	11.7	68.0	13.1	42.9	15.4	71.4	15.5[a]	15.7[a]	3.8[a]	35.0
Guardrails	4.0	2.9	0.2	7.1	2.2	3.2	1.3	6.7	5.0	3.1	a	8.1
Other	4.9	4.9	10.5	20.3	1.6	6.1	6.0	13.7	--	--	--	--
Total	40.1	112.0	46.8	198.9	37.0	111.3	55.7	204.0	44.8	50.8	15.1	110.7

Note: 1 m = 3.28 ft. [a]Significantly different from rollover crash site data ($p < 0.10$. [b]< 0.05 but not 0.00.

Figure 9. Average lengths of embankments, banks, and ditches combined in 161-m sections upstream and downstream from sites.



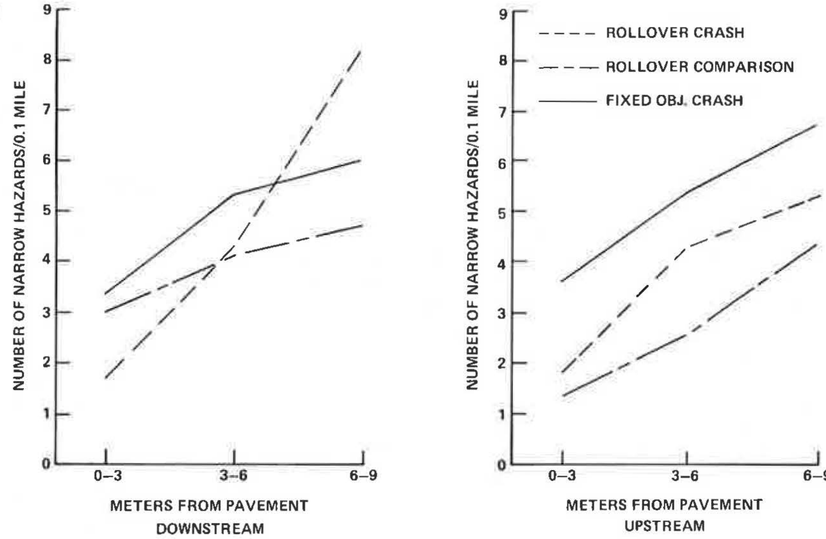


Figure 10. Average number of spot fixed objects combined in 161-m sections upstream and downstream from sites.

hazards vary relatively little along highways.

Figure 9 shows the average lengths of embankments, banks, and ditches combined in the 161-m (530-ft) sections upstream and downstream from the sites. Sharp peaks are noted within 3-6 m (10-20 ft) from the pavement edge for both the rollover crash and comparison sites; fewer of these hazards were noted at the fixed-object crash locations. The presence of the peak at the comparison location suggests that correlations in these values may exist between the rollover crash and comparison locations, as noted above. If this is the case, the role of these hazards is underestimated by the comparison of the hazards at the two locations.

Figure 10 shows the average counts of spot fixed objects combined in the 161-m sections upstream and downstream from the sites. There were twice as many spot fixed objects per section within 3 m (10 ft) of the pavement edge at the fixed-object crash sites as at the rollover crash sites. On the other hand, elongated hazards, notably embankments and ditches, were found to be nearly twice as long at the rollover crash sites as at the fixed-object crash sites.

Differences in the densities of street lights and traffic signs at rollover and fixed-object crash locations were not found to be significant. Similarly, the average lengths of guardrails, curbs, and median barriers were not significantly different.

The pavement widths at the rollover crash locations were significantly narrower ($p < 0.01$) than at the fixed-object sites, but the shoulders were significantly wider ($p < 0.001$) at the rollover sites. A greater density of driveways was found at the fixed-object sites. Differences in the number of pavement lanes and the number of intersections per section were not significant.

Approximate measures of pavement skid resistance made at 130 crash sites and 115 comparison sites were compared and found not to be significantly different ($p = 0.32$).

The roadway at each survey site was functionally classified by the field research team. A broad distribution of the roadways at the crash locations, along with a similar breakdown for all Georgia roads in rural and urban areas, is given below:

Roadway Class	Georgia Roads (%)	Crash Sites (%)
Freeway and principal arterial	5.3	31.0
Minor arterial	7.7	31.5
Collector	23.2	15.5
Local	63.8	22.0

The data suggest that there was an overrepresentation of principal and minor arterial roadways in the crash population and an underrepresentation of local roads. This phenomenon, which was also noted in the case of fixed-object crash studies (1,2), reflects the heavier traffic flows on nonlocal roads. As expected, the distribution of functional roadway classes at comparison locations was almost identical to that at crash locations.

SUMMARY AND CONCLUSIONS

Engineering surveys were performed at 214 locations in Georgia where single-vehicle fatal rollover crashes occurred over a study period of one year. Similar surveys were made at comparison locations 1.6 km upstream from the crash locations. The field survey procedures were similar to those used in two earlier studies of fixed-object crashes (1,2). It was found that single-vehicle fatal rollover crashes are more likely to occur

1. Along nonlocal (especially principal and minor arterial) roads than along local roads,
2. Along curved sections turning to the left than along straight sections or right curves,
3. Along downhill slopes than along level or uphill sections,
4. Along the outside of curves (especially left-turning curves) than along the inside, and/or
5. In the area downstream from a curve than in the area upstream.

The most prominent roadway feature associated with fatal rollover crashes in Georgia was horizontal curvature. The results indicate that locations of fatal rollover crashes can be discriminated from comparison locations by curvature greater than 6°, the same value suggested in the fixed-object studies.

Steep gradients were also found to be strongly and significantly associated with rollover crash locations. The pattern of distribution of longitudinal slopes observed in the fixed-object crash studies, in which negative slopes tended to occur upstream and positive slopes downstream, was also apparent at rollover crash locations.

Sites of rollover crashes were characterized by significantly larger changes in lateral slope at the shoulder edge than were found at comparison sites. The crash sites were also more likely to have embankments along the roadside than the comparison sites but less likely to have trees and certain other spot fixed objects.

In addition, the rollover crash sites had longer embankments, banks, and ditches than were found at fixed-object crash sites. On the other hand, more trees, poles, and signs were found at the fixed-object sites than at the rollover crash sites.

These findings may be summarized in a scenario that fits many of the rollover crashes investigated: The vehicle enters a left curve going downhill at or above a critical speed, the driver loses control of the vehicle, and the vehicle overturns near or beyond the end of the curve where the downslope flattens out.

ASSESSMENT AND RECOMMENDATIONS

Differences in rollover crash rates are explicable in part by the design features of the roadway, the configuration of the roadway surfaces, and the type and density of roadside obstacles. Undesirable geometric design features, especially excessive left-turning curves and downslopes, can increase the demands on the driver-vehicle system and contribute to loss of vehicle control and possible encroachment onto the roadside.

Once a driver has lost control of a vehicle, the outcome is determined, to a large degree, by the roadway environment: the dimensions and slopes of the cross section, the nature and density of roadside obstacles, and the configuration of the roadside surface.

Researchers are seeking to further refine road-improvement priorities for both rollover and fixed-object crashes and to account for regional differences in crash rates attributable to such factors as population, topography, and climate. Pending the completion of such work, the roadside hazard modification scheme (1,2) based on horizontal curvature and gradient should be suitable for identifying and establishing priorities for the correction of locations that have a potential for rollover crashes, in Georgia as well as in other states that have similar topography, demography, and climate.

The modifications undertaken at a specific location depend on several factors: number and type of hazards, width of right-of-way, cooperation of utility companies, and costs of alternative means of modification. In some instances, it may be possible to reduce or eliminate curvature and gradient as well as to modify the roadside. In other cases, only resloping of the roadside and removal or screening of hazardous obstacles would be appropriate. Where roadside encroachments are likely to occur, it is important for the roadside to be free of not only fixed-object hazards but also ditches, steep embankments, and other features that would increase the likelihood of vehicle rollovers.

ACKNOWLEDGMENT

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REFERENCES

1. P.H. Wright and L.S. Robertson. Priorities for Roadside Hazard Modification. Insurance Institute for Highway Safety, Washington, DC, March 1976.
2. P.H. Wright and L.S. Robertson. Amelioration of Roadside Obstacle Crashes. Transportation Engineering Journal, Proc., ASCE, Vol. 105, No. TE6, Nov. 1979, pp. 609-622.
3. J.W. Hutchinson and T.W. Kennedy. Safety Considerations in Median Design. HRB, Highway Research Record 162, 1967, pp. 1-29.
4. C.P. Brinkman and K. Perchonok. Hazardous Effects of Highway Features and Roadside Objects: Highlights. Public Roads, Vol. 43, No. 1, June 1979.

Discussion

John C. Glennon

I would first like to commend the authors of both of the preceding papers--Hall, Zador, and Wright--on their dedication and very worthwhile efforts. I believe these two studies provide some dramatic insights concerning highway safety. I use the word insight because these studies have really just scratched the surface of a more universal safety problem--the association between roadside design and highway curves. I also use the word insight as a caution against drawing any very specific conclusions from a limited sample of a recognizably small portion of the total accident population.

The most significant conclusion of these studies, and perhaps the only firm one, is that fatal overturning crashes (10 percent of all fatal accidents) are highly associated with highway curves. This conclusion seems allied to some conclusions of past research and more particularly to preliminary results of an ongoing Federal Highway Administration research project, "Effectiveness of Design Criteria for Geometric Elements". Some preliminary data from this ongoing research, in which I am a consultant to Jack E. Leisch and Associates, indicate the following:

1. Curves show an overrepresentation of roadside accidents.
2. Left curves (as seen by the colliding driver) are overrepresented in curve accidents.
3. Roadside design may be the factor that is most related to the safety of highway curves.
4. Roadsides tend to be more hazardous on curves.

Perhaps even without these two studies, we should have expected to find a predominance of overturning accidents on curves. In hindsight, I can think of three reasons for this phenomenon:

1. The proportion of run-off-the-road accidents is two to three times higher on curves than on tangents.

2. Overturning tends to be related to the side skidding and vehicle rotation that are common to curve accidents.

3. The dynamics of overturning are enhanced by the usually greater cross-slope breaks at both the edge of the pavement and the edge of the shoulder on curves.

To suggest the development of realistic prediction models or design criteria from the results of these two studies may be overoptimistic. However, the results do suggest some possible new orientations. For example, the 4:1 side slope that is commonly regarded as minimally acceptable, based on full-scale tests and simulations performed on tangent sections, may in fact be unacceptable on highway curves. In addition, the reexamination of guardrail warrants suggested by Hall and Zador may have some merit. It must be remembered, however, that their study only considers fatal overturning crashes, which constitute a small portion of all roadside accidents. Decisions on guardrail placement must, of course, consider the net effect on all roadside encroachments.

On another, more minor matter, the reader should be cautioned about basing any overt conclusions on the comparisons between Georgia and New Mexico data. The differences documented in these papers probably reflect little more than the basic differences in the two state's practices, terrains, and relative traffic exposures to various highway design configurations.

Although the fact is not new, these studies strongly reemphasize the basic safety problem of highway curves. As everyone knows, curves cannot be eliminated and flattening them is usually too expensive (and, except for extremely sharp curves, may only be marginally effective). If major improvements are to be made in safety on highway curves, therefore, these studies and the ongoing research in which I am participating seem to suggest that we look toward minimizing the consequences of run-off-the-road accidents. All indications are that, if there is to be a major emphasis in general roadside safety improvement efforts, it ought to be directed toward highway curves.

Authors' Closure

We would like to thank Glennon for his comments on these two papers. We believe that there is more importance to the results of these studies than that cited by Glennon. A study of all fatal overturning crashes in two states for a one-year period may be a limited sample, but national data clearly indicate that these types of crashes are responsible for a significant portion of highway fatalities. Furthermore, these two studies are the most recent of a series of studies of off-road crashes undertaken by the research group using common methodology. Combined, these projects have involved nearly 1000 on-site engineering surveys at crash locations plus an equal number at comparison locations.

Our data do not support Glennon's statement that roadsides are more hazardous on curves. The disproportionate share of crashes that occur at these locations seems to be more closely related to roadway alignment than to roadside design. As the paper by Wright and Zador states, undesirable geometric design features can increase the demands on the driver-vehicle system and contribute to loss of vehicle control and possible roadside encroachment. Once a driver has lost control of a vehicle, the type and severity of a crash are largely determined by the roadside environment: the dimensions and slopes of the cross section, the nature and density of roadside obstacles, and the configuration of the roadside surface.

There are several techniques the engineer can apply to reduce the frequency and severity of rollover crashes. These techniques, which include improved signing and delineation, roadway realignment, roadside barriers, and flatter side slopes, are not guaranteed to eliminate either roadside encroachments or fatal rollover crashes. We recognize, of course, that vehicles can depart from tangent roadways and overturn on very flat side slopes and that guardrail impacts can result in fatalities. We believe our data support the finding that, although it is impossible to eliminate fatal rollover crashes, the engineer can take action at a limited and identifiable number of locations to reduce the frequency of fatal roadside crashes.

Abridgment

Evaluation of Driveway-Related Accidents in Texas

RAMEY O. ROGNESS AND STEPHEN H. RICHARDS

The results of an extensive study of driveway-related accidents that occurred in Texas between 1975 and 1977 are presented. The study was conducted as part of a larger study to determine the extent and nature of driveway operational and safety problems on Texas streets and highways. The state of Texas computerized master accident file was the primary source of data for the evaluation. The findings of the study indicate that driveway-related accidents constitute a significant portion of the state's total traffic-accident experience. In fact, 16 percent of all traffic accidents in Texas during the three study years were driveway related. This percentage and the overall accident characteristics are consistent with results of previous research. The study results also indirectly suggest that better design and operation of driveways could reduce the number of driveway-related accidents and thus improve traffic safety.

An evaluation of safety and operational problems experienced at urban driveways in Texas was recently conducted (1). Improved guidelines for urban driveway location, design, and operation were developed based on the findings of this evaluation (2). As part of the research, an extensive study of driveway-related accidents that occurred in Texas between 1975 and 1977 was conducted.

The study primarily evaluated driveway-related accidents on city streets and county roads ("off-system" facilities) in Texas. A limited comparative study of driveway-related accidents on state-maintained highways was also performed. The study evaluated the driveway safety problem in terms of the number of accidents, severity, characteristics, and, to some extent, causative factors.

STUDY RESULTS

The accident study revealed that driveway-related

Table 1. Severity of driveway-related accidents on off-system roadways.

Accident Severity	Driveway-Related Accidents		All Accidents	
	Number	Percent	Number	Percent
Fatal	35	0.1	960	0.3
Injury	4 635	10.6	53 420	19.4
Property damage only	38 930	89.3	221 765	80.3
Total	43 600		276 145	

Note: Data are based on 1975-1977 Texas accident records.

accidents are relatively common in Texas and constitute a significant portion of the state's total accident problem. It suggested that driveway accidents in Texas are influenced by driveway design features, vehicle operating characteristics, and uncontrolled traffic movements at driveways.

Accident Frequency

During the three-year period from 1975 to 1977, there were 130 868 driveway-related accidents reported on city streets and county roads in Texas, or approximately 43 600 accidents/year. This number represents more than 16 percent of all off-system accidents (one out of six off-system accidents was driveway-related.)

Approximately 95 percent of these off-system driveway-related accidents reportedly occurred in urban areas. The remaining 5 percent occurred on rural county roads. In comparison, 92 percent of all off-system accidents reportedly occurred on city streets, and 8 percent occurred on county roads during the study period. Thus, the off-system driveway safety problem is concentrated, in terms of accident numbers, in urban areas.

Accident Severity

The severity of driveway-related accidents that occurred on city streets and county roads in Texas is summarized in Table 1. The data in the table indicate that about 35 fatal, 4635 injury, and 38 930 property-damage-only (PDO) accidents occurred each year during the three-year study period. These accidents resulted in an average of 36 deaths and more than 6300 injuries/year.

Table 1 also presents data that indicate that the severity of driveway-related accidents, on the average, was less than that for all off-system accidents. There were 8 fatalities per 10 000 driveway-related accidents during the study years. In contrast, 34 of every 10 000 off-system accidents resulted in a fatality.

In addition, injury rates associated with off-system driveway-related accidents were lower than injury rates for total off-system accidents. Approximately 11 percent of off-system driveway-related accidents resulted in at least one injured person, whereas 20 percent of the total accidents resulted in an injury. By using a test of proportions, these data indicate a statistically significant difference in driveway-related accident severity compared with total accident severity.

Vehicle Involvement

Vehicle involvement in driveway-related accidents is summarized below:

Type of Vehicle Involvement	Driveway-Related Accidents (%)	All Accidents (%)
Two moving vehicles	77.8	68.7
Vehicle and parked car	18.5	14.5
Vehicle and fixed object	2.3	12.0
Vehicle and pedestrian	0.3	1.4
Other	1.1	3.4

More than 96 percent of the accidents involved more than one vehicle. Almost 78 percent of these multiple-vehicle accidents involved two or more moving vehicles, and 18 percent involved a vehicle colliding with a parked car. In comparison, only 84 percent of all accidents on the off-system facilities involved more than one vehicle. This difference (96 versus 84 percent) is statistically significant but not surprising, since one might expect that one-car accidents would be less common at driveways.

A summary of the types of vehicles involved in driveway-related accidents is given below:

Type of Vehicle Involved	Driveway-Related Accidents (%)	All Accidents (%)
Passenger car	78	77
Truck	18	19
Motorcycle	3	1
Other	1	3

The data indicate that vehicle involvement in driveway-related accidents was similar to vehicle involvement in all types of accidents with a few important exceptions.

The study revealed that motorcycle involvment in driveway accidents was significantly different than in accidents as a whole. Only 3 percent of the accidents at driveways involved a motorcycle, yet these accidents accounted for one-third of the fatalities that resulted from driveway-related accidents in Texas between 1975 and 1977. In addition, the characteristics of motorcycle accidents at driveways were found to be somewhat different from those of most other types of driveway-related accidents. A disproportionately large percentage of driveway-related motorcycle accidents involved a motorcycle going out of control after the driver hit the curbing at a driveway.

The study also revealed that truck accidents at driveways are different from other types of driveway accidents. A disproportionately high percentage of driveway-related truck accidents involved a rear-end collision. A disproportionately low percentage of truck accidents, on the other hand, were accidents of the one-car, loss-of-control type. Truck involvement in angle-backing accidents was relatively high.

Trucks were also involved in a disproportionately high percentage of accidents in which a parked car was hit. The maneuvering requirements of trucks at driveways probably explain this finding. Approximately 39 percent of fatal driveway-related accidents involved a truck. It is notable that 72 percent of all fatal accidents at off-system driveways in Texas during the three-year study period involved a truck or a motorcycle.

City Size

An evaluation was conducted to determine whether the characteristics of driveway-related accidents varied with the size of the town or city in which the accidents occurred. This evaluation revealed that in smaller cities and towns the percentage of total driveway-related accidents that involved a backing vehicle was higher. For example, more than 25 percent of the driveway-related accidents in towns with a population of less than 2500 were angle-backing accidents. In cities with a population of more than 250 000, however, only 12 percent of driveway-related accidents were angle-backing accidents.

Several other differences were found in the characteristics of driveway-related accidents in large cities versus those in small towns. For the most part, however, the differences were a by-product of the greater occurrence of backing accidents in smaller towns and cities.

The increased percentage of driveway-related backing accidents in smaller cities and towns was not totally explained in this study. However, two factors that may explain this trend were suggested by an extensive statewide inventory of driveways and a review of driveway regulatory practices in Texas (1). First, many cities with populations of less than 20 000 do not regulate driveway design or operation. It is possible that the unregulated (and, consequently, inadequate) driveways and parking facilities in these towns contribute to the higher rate of driveway-related backing accidents. Second, in many smaller cities and towns, a higher percentage of the driveways serve single-family dwelling units and other low-traffic-volume generators. More backing maneuvers may occur at these driveways compared with other types of driveways.

Driveway Accidents on State-Maintained Highways

A limited study of driveway-related accidents that occurred during 1977 on state-maintained highways in Texas was conducted to determine whether there were similarities in accident characteristics between on-system and off-system facilities. This evaluation revealed that 22 754 driveway-related accidents were reported as occurring on on-system facilities in Texas in 1977. This number represents approximately 10 percent of all on-system accidents. In comparison, more than 16 percent of all off-system accidents in Texas were driveway-related. Thus, driveways appear to be a less critical safety problem on on-system facilities, at least in terms of accident numbers. This might be anticipated since there are fewer driveways along the rural highways and urban freeways that make up much of the on-system highway network in Texas.

The evaluation also revealed, however, that on-system driveway-related accidents were generally more severe than off-system driveway-related accidents. This finding may be explained by the generally higher speeds on state highways.

Motorcycles were involved in a disproportionately high percentage of on-system driveway-related accidents. Truck involvement, however, was not significantly different from truck involvement in all types of off-system driveway-related accidents.

SUMMARY AND DISCUSSION OF RESULTS

The finding that 16 percent of all accidents on city streets and county roads in Texas are driveway-related is consistent with the findings of previous studies by Box (3,4) and McGuirk (5). These respective researchers found that 12 and 14 percent of all traffic accidents occur at driveways.

By combining the off-system and on-system accident data, it was revealed that more than 93 percent of all driveway-related accidents in Texas occur in urban areas. Approximately two-thirds of all driveway-related accidents studied involved a vehicle exiting a driveway. Less than one-third involved a vehicle that was attempting to enter a driveway. Backing accidents at driveways were very common and were more common in smaller towns than in large cities.

Motorcycle and truck accidents appear to pose the greatest safety problem. Almost 72 percent of all fatal accidents involved a truck or a motorcycle. These vehicles must therefore be considered in the design and operation of driveways.

The problem of motorcycle accidents at driveways may be related to poor visibility. Vehicle drivers simply may not always see motorcycles at driveways. In addition, a high percentage of driveway-related motorcycle accidents involve a motorcycle going out of control after striking a raised curbing at a driveway. Improved driveway geometry and delineation (painting) of driveway curbs and islands could prevent some of these accidents. The problem of truck accidents at driveways, on the other hand, is apparently related to the maneuvering requirements of trucks and their large size.

The study findings suggest that improved design and operation of driveways could reduce the number of driveway-related accidents. The conflicting movements at multiple adjacent driveways could be reduced by limiting the number of driveways and by designing driveways so that potential conflict areas can be easily recognized by street traffic. Many accidents may be prevented by installing left-turn bays and/or right-turn deceleration lanes at major driveways. In addition, adequate throat width and curb radii should be provided at driveways to allow smoother turning movements. Other accidents may be prevented by designing and operating parking areas so that drivers are not forced to back through driveways.

ACKNOWLEDGMENT

The research documented in this abridgment was sponsored by the Traffic Safety Section of the Texas State Department of Highways and Public Transportation as part of a study entitled "Guidelines for Driveway Design and Operation". Bobby G. Lay of the Traffic Safety Section is acknowledged for his guidance and assistance in all phases of this research study. Special acknowledgment is extended to Roger W. McNees of the Texas Transportation Institute for assisting us in obtaining and analyzing the accident data.

REFERENCES

1. S.H. Richards. Guidelines for Driveway Design and Operation: Volume 2--Technical Report. Texas Transportation Institute, Texas A&M Univ., College Station, Res. Rept. 5183-2, April 1980.
2. S.H. Richards, R.H. Eckols, and C.L. Dudek. Guidelines for Driveway Design and Operation: Volume 3--Guidelines for Urban Driveway Regulation. Texas Transportation Institute, Texas A&M Univ., College Station, Res. Rept. 5183-3, Oct. 1980.
3. P.C. Box. Access Control and Accident Reduction. Municipal Signal Engineer, May-June 1965.
4. P.C. Box. Accident and Volume Studies. Public Safety Systems, May-June 1969.
5. W.W. McGuirk. Evaluation of Factors Influencing Driveway Accidents. Purdue Univ., West Lafayette, IN, Joint Highway Research Project, Rept. 59P, 1973.

Abridgment

Analysis of Exiting Vehicle Paths at Undivided Two-Way Driveways

STEPHEN H. RICHARDS

The results of preliminary field studies conducted at six commercial driveways to evaluate exiting vehicle paths at undivided two-way driveways are presented. Vehicle position data for several hundred exit maneuvers were collected at each driveway and analyzed. Due to the preliminary nature of the studies, no attempt was made to gather data on the volume of street traffic or information about vehicles entering the driveways. The studies suggest that encroachment by exiting traffic over the midpoint of undivided two-way driveways is fairly common, especially if the driveways have no centerline markings. They also indicate that driveway width, the type of driveway maneuver, and the presence of centerline markings greatly influence the paths taken by motorists exiting a driveway. Based on the study results, the use of centerline markings is recommended at high-volume driveways on arterial streets.

At undivided two-way driveways, the design throat width must be shared (not necessarily equally) by entering and exiting traffic. In other words, traffic waiting to exit occupies part of the driveway. Entering traffic must either use the remaining portion of the driveway or stop in the street and wait until the exiting traffic clears. Therefore, it is important to consider the "positioning" characteristics of exiting traffic in developing driveway design and control requirements. Preliminary studies (1) conducted at six commercial driveways in Bryan and College Station, Texas, were intended to

1. Determine the extent to which traffic exiting undivided two-way driveways encroaches into the portion of the driveway intended for entering traffic,
2. Investigate factors (e.g., driveway throat width and type of exiting maneuver) that may influence the positioning of exiting vehicles, and
3. Evaluate the effectiveness of centerline markings in the driveway throat in discouraging lane encroachment by exiting vehicles.

Data on exiting vehicle position were collected at each driveway by an observer as vehicles passed over a series of inconspicuous tape reference markers on the pavement. These data were obtained only when there were no entering vehicles at the driveway. Due to the limited nature of the studies, no attempt was made to gather data on the volume of street traffic or information about vehicles entering the study driveways.

In the first part of the studies, data were collected at six driveways that had no centerline markings. Two driveways in each of three width categories (narrow, intermediate, and wide) were evaluated. Both of the narrow driveways were 25 ft wide. The intermediate driveways were between 30 and 35 ft wide. The wide driveways were slightly more than 50 ft wide. In the second part of the studies, a before-and-after evaluation of a centerline marking treatment was conducted at one of the narrow driveways.

STUDY RESULTS

The studies conducted should be considered preliminary, since they evaluated exiting maneuvers at a limited number of driveways in only one geographic region. Additional field studies are needed to fully validate the results.

The results suggest that encroachment by exiting traffic over the midpoint of undivided two-way driveways is fairly common, especially if there are no centerline markings present. The percentage of encroaching traffic is greatly influenced by driveway width and the type of exit maneuver (right or left turn). The results also indicate that centerline markings at an undivided two-way driveway may significantly reduce the frequency and extent of exiting vehicle encroachments.

Undivided Two-Way Driveways with No Centerline Markings

In the first part of the studies, exiting drivers were observed encroaching over the driveway midpoint at all six driveways. These encroaching vehicles hampered or blocked entry maneuvers into the driveways in some instances.

Encroachments by right-turn exiting vehicles appeared to decrease as driveway width increased, as shown in Figure 1. On the average, 25 percent of the right-turn traffic encroached at the two narrow driveways, 12 percent at the intermediate-width driveways, and almost no traffic at the wide driveways. Encroachments by left-turn exiting vehicles showed a much different trend, as shown in Figure 2. Left-turning drivers encroached more frequently at the intermediate-width driveways and less often at the narrow and wide driveways.

Figure 3 shows the range of paths used by exiting drivers by driveway width. From Figure 3, the paths selected by left-turn and right-turn traffic were similar at the narrow driveways. At the intermediate-width and wide driveways, left-turn traffic tended to use the middle of the driveway. Right-turn traffic tended to use the right side but remained a comfortable distance from the right curb line.

Undivided Two-Way Driveways with Centerline Markings

In the second part of the studies, a before-and-

Figure 1. Effects of driveway width on encroachment by right-turn exiting vehicles.

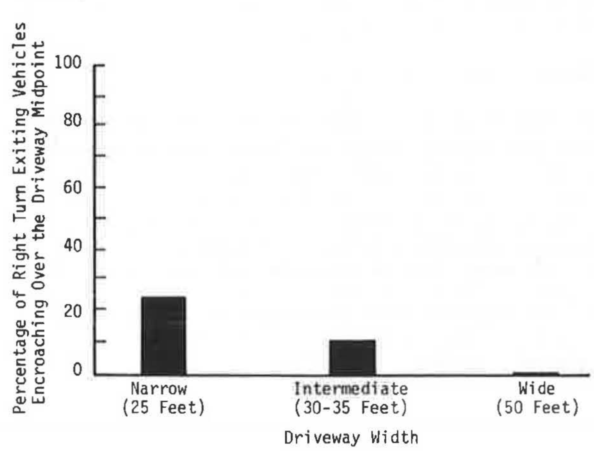


Figure 2. Effects of driveway width on encroachment by left-turn exiting vehicles.

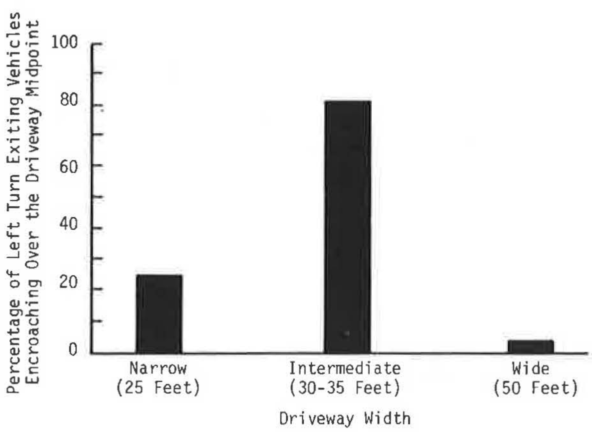


Figure 3. Paths of exiting vehicles.

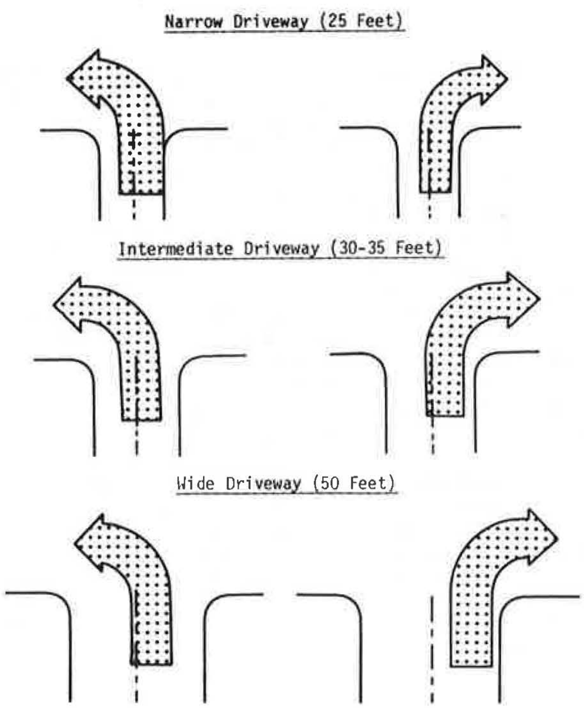


Figure 4. Study driveway (top) with and (bottom) without centerline markings.

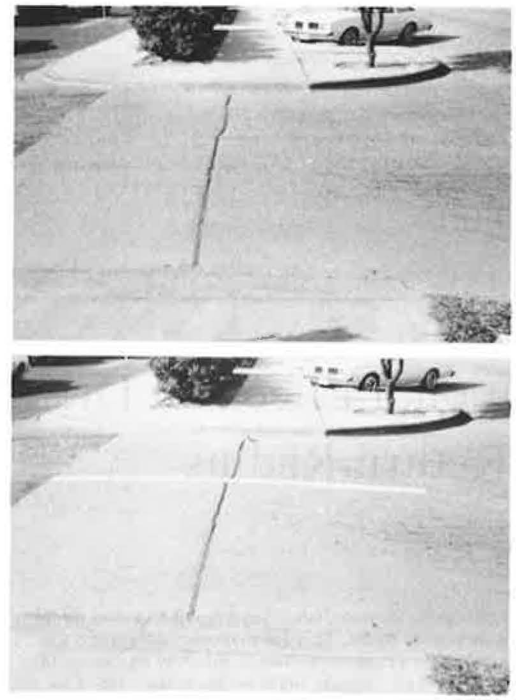
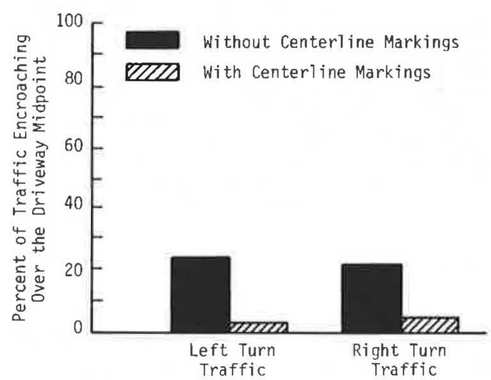


Figure 5. Effects of centerline markings on encroachment by right- and left-turn traffic.



after evaluation (with and without centerline markings) was conducted at one of the narrow driveways. Figure 4 shows the study driveway before and after the centerline markings were installed. The driveway had a throat width of 25 ft and a curb return radius of 15 ft.

Figure 5 shows the results of the before-and-after evaluation. The figure shows that the solid yellow centerline reduced the percentage of both left- and right-turn traffic encroaching over the driveway midpoint. Before the centerline markings were installed, about 23 percent of left-turn and 20 percent of right-turn exiting traffic encroached. After the centerline was installed, only 3 percent of the left-turn and 5 percent of the right-turn exiting traffic encroached.

The centerline markings also reduced the maximum encroachment distance, particularly for left-turn exiting traffic. The maximum encroachment by a left-turn exiting vehicle when no centerline markings were in place was 7 ft over the midpoint. When a centerline was present, the maximum encroachment was only 2 ft over the midpoint.

RECOMMENDATIONS

Based on the study results, centerline markings similar to those evaluated are recommended for undivided two-way driveways where conflicts between entering and exiting traffic result from exiting vehicles using too much of the driveway width. These markings can reduce the number and extent of midpoint encroachments by exiting traffic. The centerline markings can be placed on the driveway midpoint or offset to provide additional width for entering or exiting traffic as needed. The effectiveness of an offset centerline has not been evaluated, however.

Centerlines may be most appropriate at high-volume commercial driveways on arterial streets. At these driveways, the probability of dual use (simultaneous entry and exit) is high and, when a conflict occurs, it may have a very negative effect on traffic operations on the arterial street.

ACKNOWLEDGMENT

This abridgment presents results of a research study sponsored by the Traffic Safety Section of the Texas State Department of Highways and Public Transportation, entitled "Guidelines for Driveway Design and Operation". Bobby G. Lay of the Traffic Safety Section is acknowledged for his guidance and assistance in all phases of the research study.

REFERENCE

1. S.H. Richards. Guidelines for Driveway Design and Operation: Volume 2--Technical Report. Texas Transportation Institute, Texas A&M Univ., College Station, Rept. 5183-2, 1980.

Operational Effects of Driveway Width and Curb Return Radius

STEPHEN H. RICHARDS

Existing driveway design standards include independent design controls for throat width and curb return radius. They fail to recognize that these two driveway features may have an aggregate effect on driveway operation. In addition, current standards for driveway width and curb return radius are based primarily on vehicle turning capabilities and do not consider how drivers respond, in terms of speed and path, to various driveway designs. The results of proving-ground studies conducted to evaluate the effects of driveway width, curb return radius, and offset taper approach treatments on the speed and path of drivers entering and leaving driveways are presented. A total of 59 nonprofessional drivers participated in the studies. These motorists, driving an instrumented study vehicle, collectively performed more than 1400 driveway entry and exit maneuvers. Speed and path data were collected for each maneuver and were analyzed to determine the relative performance of 19 driveway design conditions. The studies revealed that current standards for driveway width and radius result in driveway designs that encourage very slow entry speeds and, in many cases, undesirable vehicle paths. Recommendations are presented, based on the study results, for driveway width and radii requirements. The studies also found that offset taper approach treatments do not have a significant effect on entry paths or speeds at driveways.

A primary objective of driveway regulation is to establish design controls for the physical features of driveways. Experience indicates that these design controls are needed to promote efficient traffic operation and safety (1,2). However, many of the design controls contained in existing state and local regulations are based more on intuition than on engineering evaluation. The actual effects of these controls on traffic operations and safety are not fully known and, because there is no documented evidence supporting them, it is sometimes difficult to justify or defend their use.

There is a particular need to determine how drivers respond (in terms of path and speed) to driveway throat width and curb return radius. Existing design controls for width and curb return radius are based primarily on vehicle turning capabilities and do not consider driver performance characteristics. In addition, existing regulations present independent design controls for these two driveway features. They do not recognize that width and curb return radius may have a combined effect on vehicle speed and path at driveways (3,4).

STUDY DESCRIPTION

A series of proving-ground studies was developed to evaluate driver response to various driveway features in terms of speed and path. The objectives of each study were as follows:

1. Study 1--Determine the effects of throat width and curb return radius (as individual design features and in combination) on the speed and path of drivers turning right into driveways,
2. Study 2--Determine the effects of exiting vehicle position on the speed and path of drivers turning right into driveways,
3. Study 3--Evaluate the effects of offset taper approach treatments on the speed and path of drivers turning right into driveways,
4. Study 4--Evaluate the effects of curb return radius on the speed and acceleration of drivers turning right out of driveways, and
5. Study 5--Evaluate the effects of unequal entry and exit curb return radii on the speed and path of drivers turning right into driveways.

In all five studies, a group of "off-the-street" motorists drove an instrumented study vehicle through a specially constructed driveway test track. The speed and path of these drivers as they entered or exited the various driveways under study were recorded. A comparative evaluation of the different driveways was then made based on the speed and path data.

Test Track

The studies were conducted at the Texas A&M University Proving Ground facility in Bryan, Texas. This facility is located at an abandoned U.S. Air Force base, and the unused airport runways provide an ideal environment for controlled driving studies.

A driveway test track, approximately 2000 ft long, was constructed on one of the runways. The study driveways were constructed by using canvas fire hoses, which were painted yellow and stuffed with wood shavings. The fire hoses provided a three-dimensional visual target and physical barrier very similar to concrete curbing and were flexible enough to use on both curved and tangent sections. In addition, the pliable hoses created no safety hazard and were easily repaired when damaged by a vehicle impact. Since the hoses were portable and did not scar the pavement, the test-track layout could be changed quickly and effectively in order to evaluate a new set of driveways.

The two test-track layouts used for the studies are shown in Figure 1. The first layout had eight

Figure 1. Test-track layouts: (top) first layout and (bottom) second layout.

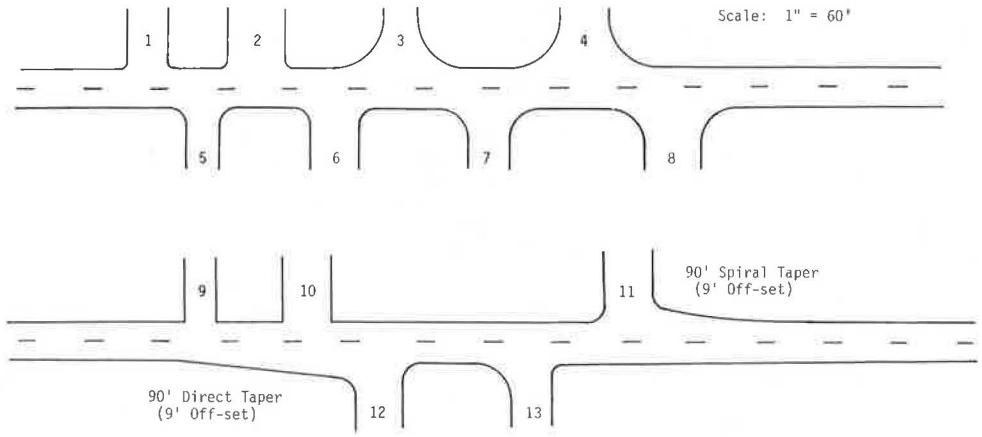


Table 1. Measurements of driveways evaluated in study 1.

Test-Track Layout	Driveway	Width (ft)	Curb Return Radius (ft)	Spacing Between Driveways (ft)
First	1	25	5	35
	2	35	5	60
	3	20	30	85
	4	30	30	
	5	20	10	55
	6	30	10	65
	7	25	20	80
	8	35	20	
Second	9	20	0	40
	10	30	0	165
	11	30	10	
	12	30	10	65
	13	25	20 (entry), 5 (exit)	

driveways, and the second layout had five. Temporary centerline and stopline markings (not shown in the figure) were installed at certain driveways during some of the studies. The measurements of the driveways in both layouts are given in Table 1.

It should be emphasized that driving conditions and driver expectancies at the test track were somewhat different from those that would exist under normal driving situations. For example, there was none of the vehicle or pedestrian traffic that, under normal conditions, could influence driver behavior. In addition, the entire runway was level and there were no approach grades or changes in elevation at the driveway entrances. For these reasons, extreme caution must be used in relating the speed and path data for a test driveway to an actual driveway.

Test Vehicle

A 1977 Chevrolet Impala with a 305 V-8 engine was used as the test vehicle in all studies. This vehicle was selected as representative of a standard-sized automobile. It had an overall length of 212 in, a total width of 76 in, and a wheel base of 116 in. The vehicle was equipped with power steering and brakes and weighed approximately 3800 lb.

A Labro Track Test fifth wheel was mounted on the rear bumper of the test vehicle and positioned so that it tracked behind the left wheels. A tachometer on the fifth wheel transmitted continuous speed data to a two-channel Brush 222 strip-chart recorder mounted in the vehicle. The recorder was remote controlled from the front seat.

Test Subjects

A total of 59 paid test subjects from the Bryan-College Station, Texas, area participated in the studies. Only licensed drivers with normal driving experience and skills were selected. Most subjects participated in several of the studies, and approximately 30 drivers participated in each study.

On the average, the study sample was younger and better educated than the national driver population and included a disproportionately high percentage of female drivers. However, since the studies involved determining the "relative" performance of various driveway designs, these sampling biases probably had minimal effect on the study results (relative performance was determined by comparing a group of drivers' responses to one design with the same group's responses to another design).

Study Administration

Because a limited number of driveways could be constructed on the test track at one time, the studies were administered in three phases:

1. During phase 1, most of study 1 was conducted. The first test-track layout (Figure 1) was used, and 31 of the 59 subject drivers participated.
2. During phase 2, studies 2 and 4 were administered to 29 of the 59 participating drivers. The first test-track layout was again used. An unmanned vehicle was positioned in the two driveways used for study 2, and stopline and centerline markings were installed at the four driveways used for study 4.
3. In phase 3, studies 3 and 5 and the remainder of study 1 were administered. The second test-track layout (Figure 1) was used and 26 of the 59 subject drivers participated.

Only one driver at a time was allowed on the test track and, once a driver entered the test track, he or she drove until completing an entire study phase. Approximately 45 min of driving time was required to complete each phase. Several subjects participated in more than one phase; however, there was a two- or three-week time period between each phase.

Study Procedure

On arriving at the study headquarters, the subjects were briefed on the nature of the studies and were driven through the test track by the study administrator, who explained the study procedures and demonstrated the required maneuvers. Minimum instruction was given on how to use the driveways.

Every subject was told to use test-track driveways as he or she would normally use driveways in daily driving. Each subject was also encouraged to make comments about any of the driveways being evaluated in the studies.

The subject was then allowed to drive the test vehicle. Each subject made a few practice runs through the test track to become familiar with the study procedures, the track layout, and vehicle handling characteristics. After the subject felt comfortable with the procedures and the vehicle, he or she began the required study maneuvers. The study administrator rode with each subject throughout the studies to give instructions and operate the strip-chart recorder. The subjects performed the various study maneuvers (right-turn entry or exit) three times at each driveway. Each subject progressed through the series of maneuvers required for a given study phase in a random manner. This prevented the driver learning process from affecting the overall study results.

Right-Turn Entry Maneuvers

Studies 1, 2, 3, and 5 involved a right-turn entry maneuver. In these studies, subjects accelerated to 30 miles/h, drove to a particular driveway (identified by traffic cones placed out of the travel lanes on either side of the driveway), and made a "comfortable" right turn into the driveway. In entering the driveway, the subjects could use any speed and path they believed appropriate. After entering, they drove approximately 50 ft into the driveway throat and stopped.

Right-Turn Exit Maneuvers

Study 4 involved a right-turn exit maneuver. Stopline and centerline markings were installed at the four driveways used in the study. The stopline markings provided a common starting point for all right-turn exit maneuvers, and the centerline markings were positioned to provide a 15-ft exit lane at all driveways. To make the required exit maneuver, the subjects, after stopping on the stopline, accelerated and turned right out of the driveway into the right travel lane. They continued accelerating at a comfortable rate until reaching a speed of 30 miles/h or more.

Data Collection

The speed of the test vehicle was continuously monitored during all five studies by the fifth wheel, a tachometer, and a strip-chart recorder assembly. The strip-chart recorder provided a continuous plot of vehicle speed versus time. An event recorder connected to the strip-chart recorder enabled the study administrator, who operated the equipment, to identify the point in time (and spot speed) at which the test vehicle cleared the travel lanes. The event recorder made it possible to relate the time-speed plot to the position of the vehicle at the driveway.

Vehicle position data were recorded for all right-turn entry maneuvers (studies 1, 2, 3, and 5). These data were collected by two ground observers, who manually recorded the position of the test vehicle's right front wheel as it passed over several sets of tape reference markers on the pavement. Wheel position relative to driveway geometry was measured to the nearest foot by using the tape marker system.

RESULTS

Study 1

The first study determined the effects of throat width and curb return radius on the path and speed of drivers turning right into driveways. The 10 driveway designs evaluated are given below:

Throat Width (ft)	Curb Return Radius (ft): None	5	10	20	30
20	X		X		X
25		X		X	
30	X		X		X
35		X		X	

Figure 2 shows the mean path of the test vehicle's right front tire (and paths representing one and two standard deviations from the mean path) observed at each of the study 1 driveways. For the designs studied, the average driver tended to move toward the test-track centerline to make the required right-turn entry maneuvers. Most drivers, however, did not encroach over the test-track centerline before entering a particular driveway.

Figure 2 also shows that vehicle paths tended to parallel the entry curbline at driveways that had a curb return radius of 20 ft or more and to diverge from the entry curbline at driveways that had a radius of 10 ft or less. In the latter cases, drivers made wide turns into the driveway throat to compensate for the small radius. Once the drivers entered the driveways, they turned toward the entry curbline to reposition their vehicles on the proper (entry) side of the driveways.

Driveway width did not significantly influence vehicle path at driveways that had a curb return radius of 20 ft or more (Figure 2). Motorists generally used the entry side of driveways to perform the right-turn entry maneuvers. At driveways that had a radius of 10 ft or less, however, drivers tended to make a wider turn at the wider driveways. They encroached into the exit side of the driveway to compensate for the small curb return radius.

Figure 3 summarizes the speed data collected at the 10 driveways in study 1. The figure shows speed profiles for right-turn entry traffic at driveways 4 and 9. Driveway 4 (Figure 1) was 30 ft wide and had a 30-ft curb return radius. Driveway 9 (Figure 1) was 20 ft wide and had no curb return. The speed profiles for the other eight driveways in study 1 fall within the boundaries established by the speed profiles for these two driveways.

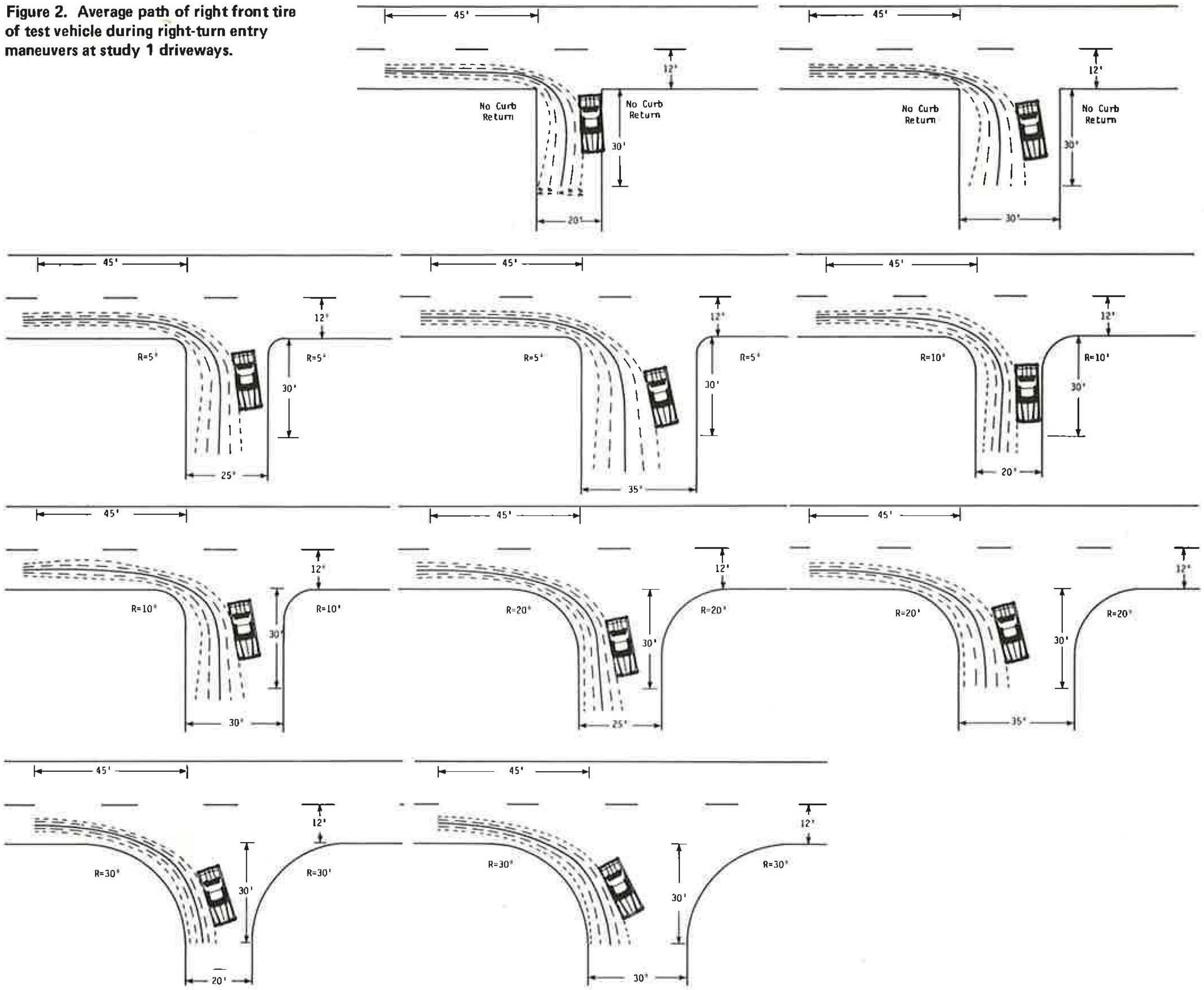
The results show that the speed profiles for all of the study 1 driveways have the same basic shape and almost overlap. This indicates that, in the absence of an exiting vehicle, the average speed of right-turn entry vehicles is essentially the same for driveways that have throat widths ranging between 20 and 35 ft regardless of curb radius. Drivers began slowing down (from 30 miles/h) approximately 250 ft upstream of the driveways. They began to decelerate more rapidly about 150 ft upstream of the driveways and continued decelerating in the driveways.

Drivers slowed down considerably to enter all of the driveways. The average entry speed (the speed when the test vehicle cleared the test-track travel lane) at driveway 4 was 13 miles/h, and the average entry speed at driveway 9 was 9 miles/h. It is important to note that the range of speeds observed at all of the study 1 driveways was relatively small. There was only a 4-mile/h difference in the average entry speed observed at the "fastest" and "slowest" driveways (driveways 4 and 9).

Study 2

The objective of the second study was to evaluate the effects of an exiting vehicle on the speed and path of drivers turning right into driveways. The

Figure 2. Average path of right front tire of test vehicle during right-turn entry maneuvers at study 1 driveways.



study also provided additional data for study 1. Figure 4 shows the six driveway situations evaluated. The various situations were created by changing the position of an exiting vehicle at two different driveways. As Figure 4 shows, the exiting vehicle was positioned in the test driveways so that the available entry widths were 10, 15, and 20 ft; the two driveways had curb return radii of 5 and 20 ft, respectively.

Figure 5 summarizes the study 2 vehicle-path data. At the driveways that had a 5-ft curb return radius, drivers tended to use as much of the available throat width as possible to complete the right-turn entry maneuvers. At the driveways that had a 20-ft curb return radius, drivers remained on the entry side of the driveway and their paths paralleled the entry curbline.

Drivers experienced extreme difficulty in turning right into the two driveways that had an available width of only 10 ft. As shown in Figure 5, many drivers ran over the curbing. Some drivers even refused to enter those driveways, saying that, if they encountered a similar situation while driving, they would stop in the travel lane and wait for the exiting vehicle to leave the driveway.

Figure 6 shows speed profiles for the six driveway situations evaluated. From the figure, average entry speeds ranged from 5 miles/h (10-ft width and 5-ft curb return radius) to 11 miles/h (20-ft width and curb return radius).

The speed profiles for the study 2 driveways are similar in shape and magnitude to those for study 1, except for the most restricted situation, in which the available width was only 10 ft and the curb return radius was 5 ft. At this driveway, drivers began rapid deceleration approximately 300 ft upstream of the driveway and reached their slowest speed (4-mile/h average speed) while still in the travel lane. Several drivers actually stopped in the travel lane before attempting to enter the severely restricted driveway opening.

Study 3

The third study evaluated the effects of offset taper approach treatments on the speed and path of drivers turning right into driveways. Two taper treatments were studied: direct and spiral. Each treatment, shown in Figure 7, had a 90-ft taper section that produced a 9-ft curbline offset at the driveway. The treatments were used at driveways that had 30-ft throat widths and 10-ft curb return radii.

Figure 8 shows vehicle paths for the two drive-

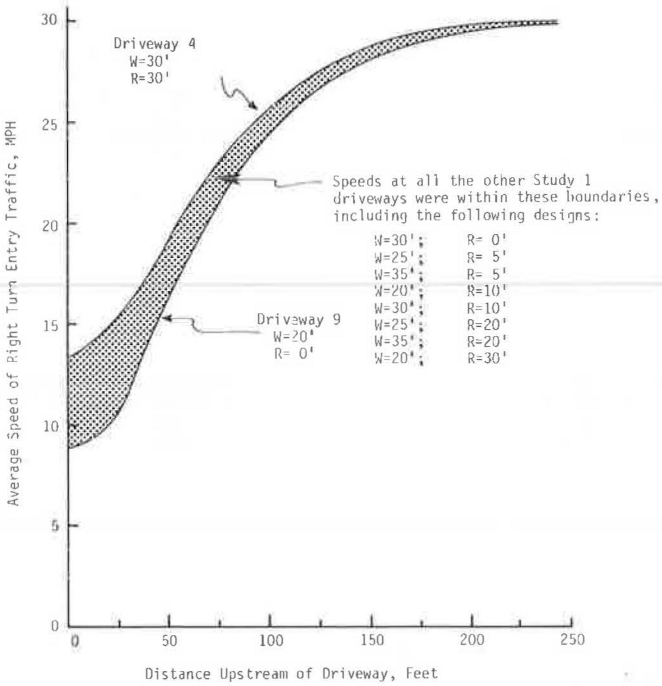


Figure 3. Summary of vehicle speed data for study 1.

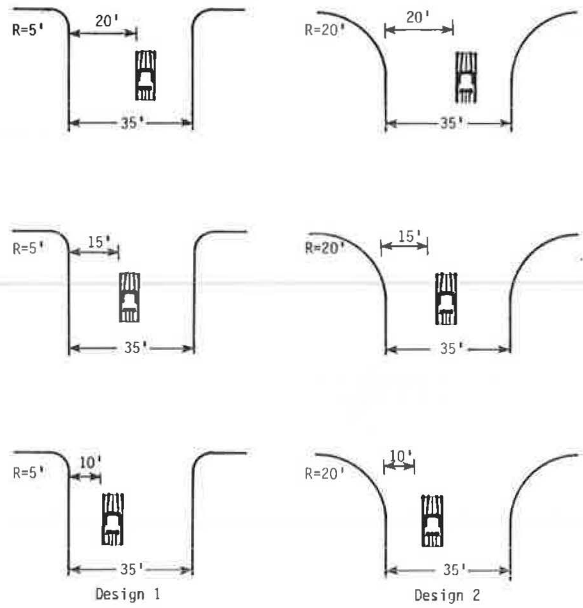
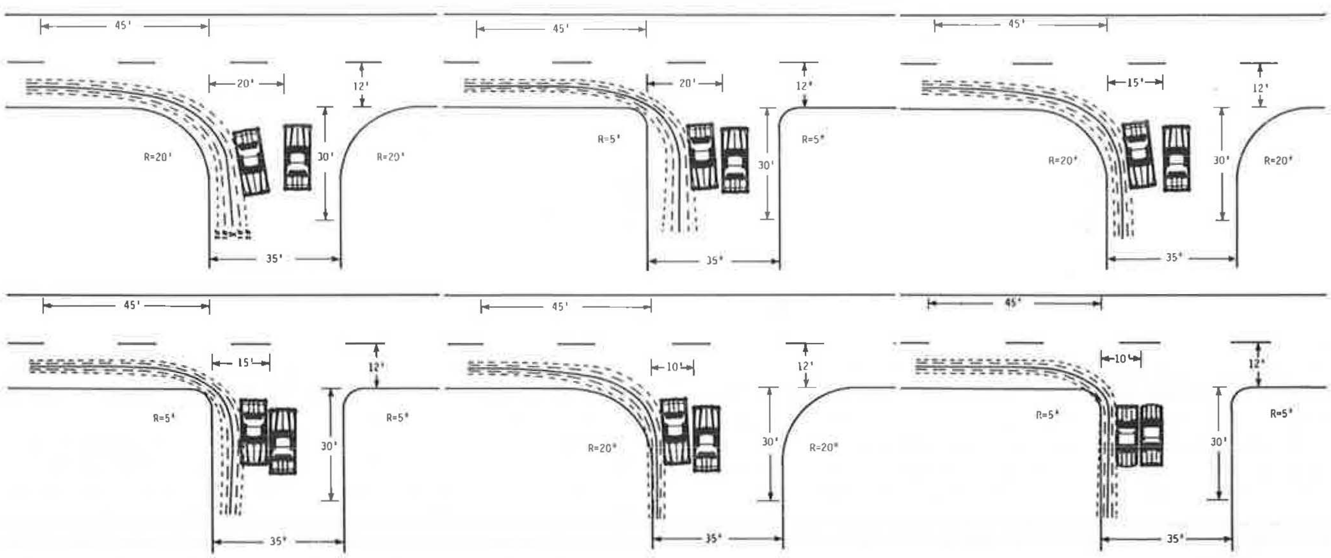


Figure 4. Driveway situations evaluated in study 2.

Figure 5. Average path of right front tire of test vehicle during right-turn entry maneuvers at study 2 driveways.



ways and reveals that, at both driveways, vehicle paths paralleled the tapered entry curbline. This trend was more apparent at the driveway that had the spiral taper approach treatment. Drivers also tended to use the entry side of the driveway. Encroachment into the exiting portion of the driveways was less than the encroachment observed at driveway 6 in study 1. (Driveway 6 was identical to the study 3 driveways except that it had no taper approach treatment.)

Figure 9 shows the speed profiles for the two approach treatments evaluated in study 3 and for the similar study 1 driveway. The speed profiles indicate that the offset taper approach treatments evaluated offered no advantages in terms of increased approach and entry speeds. In addition, there was no significant difference between the direct and spiral taper designs in terms of speed.

Study 4

Study 4 was designed to evaluate the effects of curb return radius on the speed and acceleration of drivers turning right out of driveways. Existing curb return radii of 5, 10, 20, and 30 ft were evaluated.

Figure 10 shows speed profiles for right-turn exiting traffic at the study 4 driveways. The profiles indicate that exit curb return radius had very little effect on the speed and acceleration of traffic turning right out of the driveways studied. For example, the average driver accelerated to a

Figure 6. Summary of vehicle speed data for study 2.

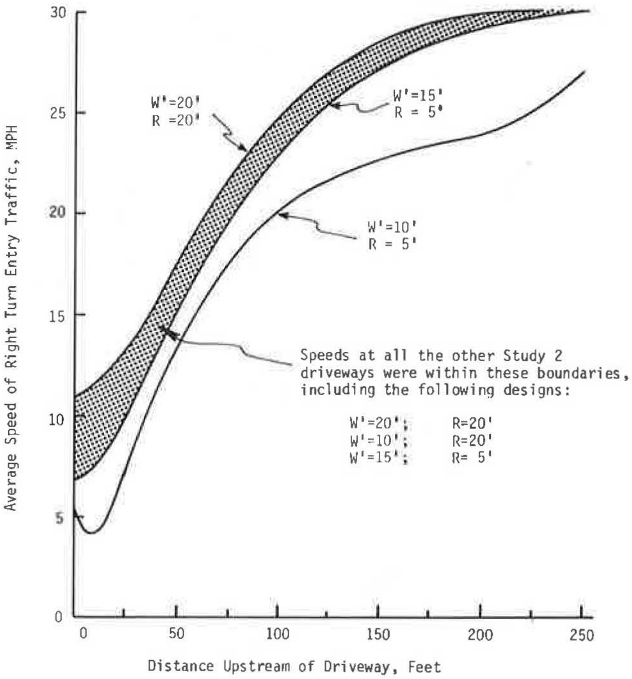


Figure 7. Offset taper approach treatments evaluated in study 3.

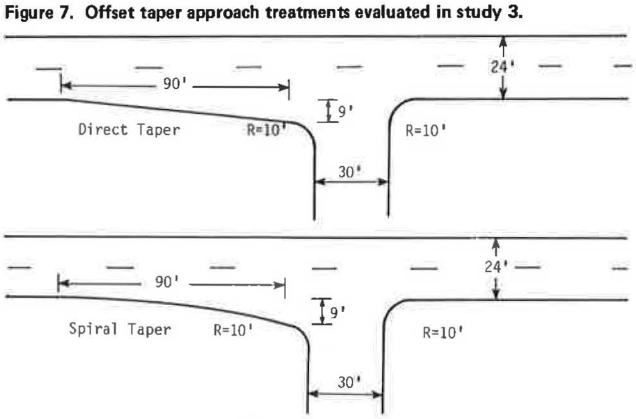


Figure 8. Average path of right front tire of test vehicle during right-turn entry maneuvers at study 3 driveways.

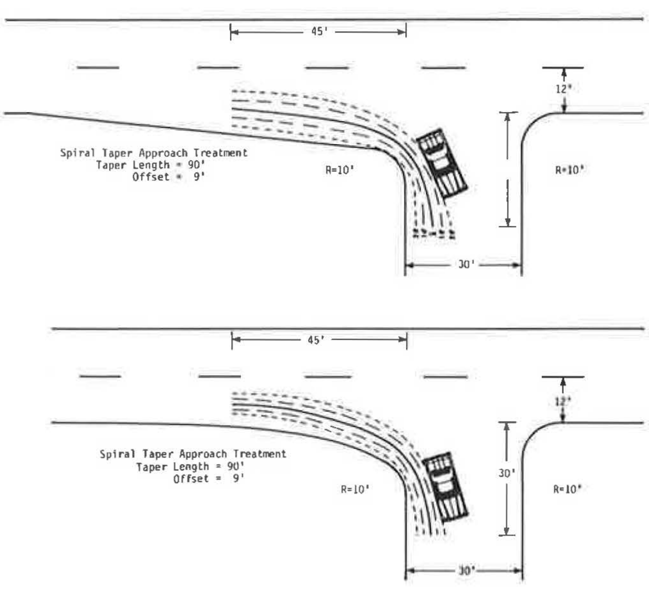


Figure 9. Effects of direct and spiral taper approach treatments on speed of right-turn entry traffic.

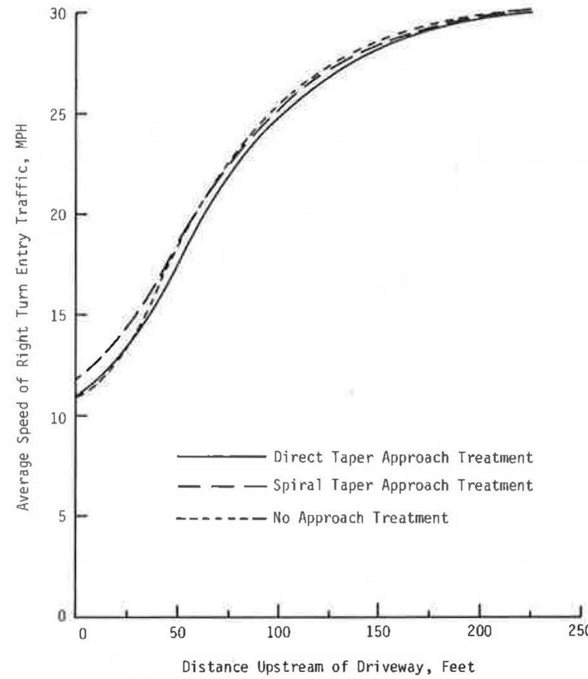


Figure 10. Influence of curb return radius on speed of right-turn exit traffic.

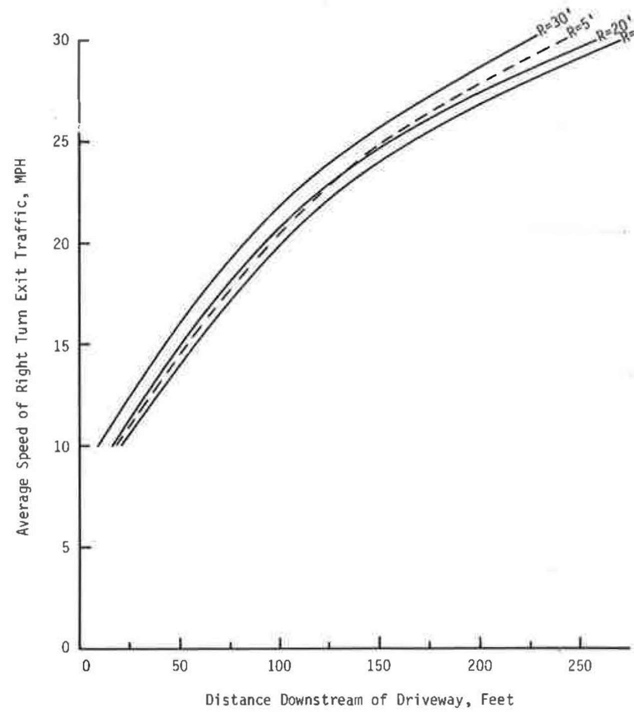
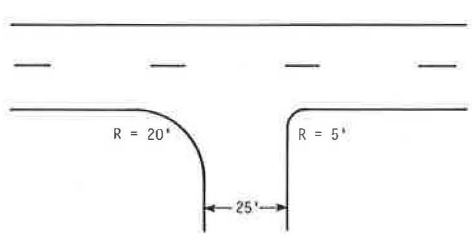


Figure 11. Driveway evaluated in study 5 (unequal entry and exit curb return radii).



speed of 20 miles/h in the first 100 ft after exiting from the driveway that had a 5-ft exit curb return radius and accelerated to 22 miles/h in the first 100 ft after exiting from the driveway that had a 30-ft exit curb return radius. In all likelihood, this relatively small difference (20 versus 22 miles/h) is not significant in terms of safety or traffic operations.

The effects of curb return radius on the positioning of right-turn exiting traffic were not evaluated in the proving-ground studies. However, if the exit curb return radius is small (e.g., less than 10 ft), it is reasonable to assume that right-turn exiting traffic would encroach more into the entry portion of the driveway and exit speed may be reduced in some cases.

Study 5

Study 5 was designed to determine whether unequal entry and exit curb return radii at a driveway affect the speed and path of right-turn entry vehicles. Figure 11 shows the single driveway tested in this study.

Vehicle-path data collected at the study 5 driveway are shown in Figure 12. The 25-ft driveway had an entry curb return radius of 20 ft and exit curb return radius of 5 ft. Also shown are the path data for the study 1 driveway (driveway 7), which was identical except that it had equal (20-ft) entry and exit curb return radii. The data in the figure reveal that there was little or no difference in the mean vehicle path at the study 5 driveway compared with the study 1 driveway. Therefore, the use of unequal radii had no apparent effect on vehicle path.

The speed profiles for right-turn entry traffic at the study 5 and study 1 driveways are shown in Figure 13. From the figure, approach speeds at the study 5 driveway (which had unequal curb return radii) were slightly lower than those at the study 1 driveway (which had equal curb return radii). This difference may suggest that right-turn entry traffic, on seeing a sharp exit curb return, slowed down more in anticipation of a difficult entry maneuver.

SUMMARY

Proving-ground studies were conducted at a driveway test track to evaluate the effects of driveway width, curb return radius, and offset taper approach treatments on the speed and path of driveway users. The 59 drivers who participated in the studies collectively performed more than 1400 driveway entry and exit maneuvers in an instrumented study vehicle. Speed and path data were collected for each maneuver, and these data were analyzed to determine the relative performance of 19 driveway design conditions. The results of the proving-ground studies are summarized below:

1. Driveway width and curb return radius had a combined effect on the speed of right-turn entry traffic at the test-track driveways. The relation among width, curb return radius, and speed of right-turn entry traffic is summarized in Figure 14. The figure shows that average entry speed decreased as available width and/or curb return radius decreased.

2. At driveways that had a curb return radius of 20 ft or more, the paths of vehicles turning right into the driveways tended to parallel the entry curbline and drivers tended to remain on the entry side, regardless of driveway width.

3. At driveways that had a curb return radius of 10 ft or less, drivers tended to make a wide turn, using all of the available throat width to compensate for the relatively small curb return radius. Once in these driveways, drivers immediately steered back toward the entry curbline to reposition the test vehicle on the proper (entry) side of the driveway.

4. The presence of an exiting vehicle in a driveway had a greater effect on the speed and path of right-turn entry traffic than could be explained by the reduction in available width that resulted from the position of the exiting vehicle. For example, the added effect of an exiting vehicle on entry speed can be seen by comparing the speed curves shown in Figure 14 for the driveways in studies 1

Figure 12. Average path of right front tire of test vehicle during right-turn entry maneuvers at (top) study 5 driveway and (bottom) driveway 7 (study 1).

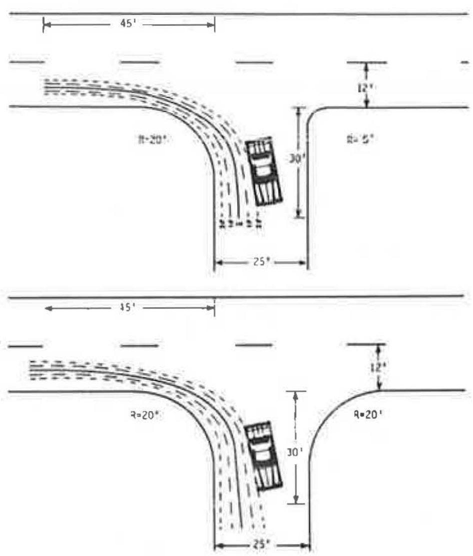


Figure 13. Effects of unequal curb return radii on speed of right-turn entry traffic.

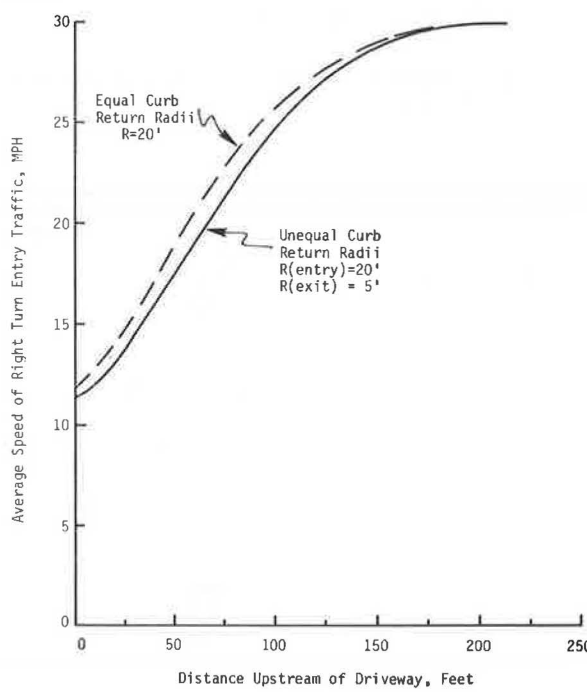
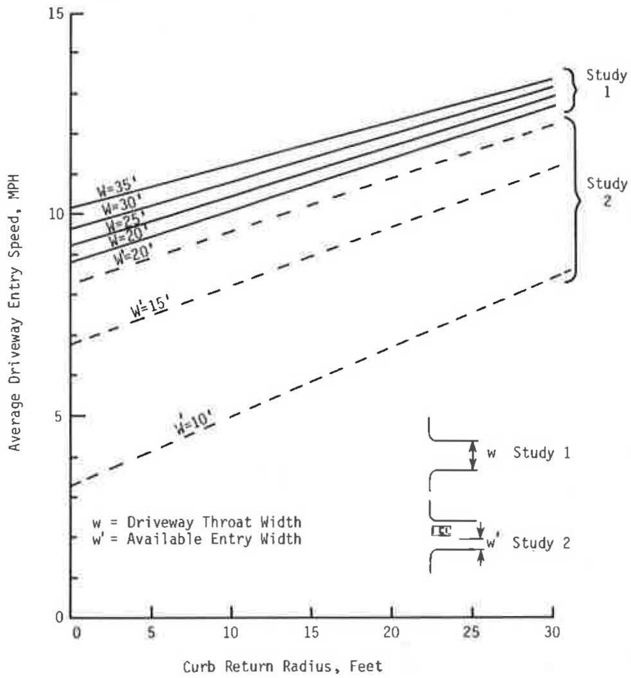


Figure 14. Influence of driveway width and curb return radius on driveway entry speed.



and 2, which both had an available width of 20 ft. Entry speed at the study 2 driveway (where an exiting vehicle was present) was slightly lower than that at the study 1 driveway (where no exiting vehicle was present).

5. The two offset taper approach treatments evaluated in the studies--direct and spiral--offered no advantages in terms of approach or entry speeds. However, in comparison with an identical driveway that had no approach treatment, these treatments did slightly reduce encroachment by right-turn entry traffic into the exit side of the driveways.

6. Under the test-track conditions, exit curb return radius had very little influence on the exit speed and acceleration of right-turn exiting traffic. This finding indicates that, at some driveways, the use of unequal curb return radii (e.g., large entry radius and small exit radius) may be acceptable.

7. The use of a relatively large entry curb return radius (20 ft) in combination with a small exit curb return radius (5 ft) was evaluated with respect to its effect on the speed and path of right-turn entry traffic. Although this use of unequal radii had no significant effect on vehicle path, approach and entry speeds were slightly lower compared with speeds at a driveway that had equal radii. Apparently, many drivers, seeing the small exit curb return radius, slowed down more in anticipation of a difficult entry maneuver.

DISCUSSION AND RECOMMENDATIONS

As mentioned earlier, driving conditions and driver expectancies at the driveway test track were different from those that exist under normal driving conditions. There was no vehicle or pedestrian traffic present, and there were no approach grades or elevation changes at the driveway entrances. Only one type of vehicle was evaluated. For these reasons, the speeds and paths observed at the test-track driveways may be different from those that would be observed at similar operational driveways, and direct application of the speed and path findings may not be appropriate. The findings can be used, however, to compare the relative performance of various driveway designs and, based on the study results, some general recommendations on driveway design can be made:

1. Current standards (5) for driveway width and curb return radius result in driveway designs that may encourage very slow entry speeds (less than 15 miles/h) and, in some cases, undesirable paths. Improved standards are needed, especially for high-volume driveways and driveways on high-speed arterials. In particular, the studies support the need for deceleration lanes at these driveways because the normal curb return radii and widths now used in urban areas severely limit entry speeds.

2. Driveway width and curb return radius work in combination to affect entry speed and path. Standards should be developed that recognize the relation among these features. Current design standards treat these as independent design features.

3. The entry curb return radius (for right-turn entry traffic) at two-way driveways should be at 20 ft to encourage desirable entry paths (for a standard-sized automobile). Smaller entry radii will force drivers to encroach into the exit side of the driveway.

4. Relatively small exit curb return radii (as small as 5 ft) may be used at driveways without significantly affecting the speed or acceleration of exiting traffic. If the radius is too small, however, it will probably negatively affect the path of right-turn exiting traffic.

5. Drivers of standard-sized automobiles should have at least 15 ft of "open" driveway to turn into; otherwise, they must slow down excessively to make the difficult turn maneuver. This suggests that a two-way driveway width of 30 ft would be desirable at high-volume driveways and at driveways on arterial streets (a 15-ft exit lane and a 15-ft entry lane). A narrower driveway may be desirable if the path of exiting vehicles can be controlled by using a centerline or median (1).

6. Offset taper approach treatments, similar to those evaluated, do not significantly increase entry speeds at driveways and have only a minor positive effect on entry path. Their widespread use is not supported by the study findings.

ACKNOWLEDGMENT

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REFERENCES

1. S.H. Richards. Guidelines for Driveway Design and Operation: Volume 2--Technical Report. Texas Transportation Institute, Texas A&M Univ., College Station, Res. Rept. 5183-2, 1980.
2. V.G. Stover, W.G. Adkins, and J.C. Goodnight. Guidelines for Medial and Marginal Access Control on Major Roadways. NCHRP, Rept. 93, 1970.
3. J.C. Glennon and others. Technical Guidelines for the Control of Direct Access to Arterial

Highways. FHWA, Rept. FHWA-RD-76-87, 1976.
4. R.H. Eckols and S.H. Richards. Guidelines for Driveway Design and Operation: Volume 1--Annotated Bibliography. Texas Transportation Institute, Texas A&M Univ., College Station, Rept. 5183-1, 1980.
5. Guidelines for Driveway Design and Location. ITE, Arlington, VA, 1975.

Effects of Paved Shoulders on Accident Rates for Rural Texas Highways

DANIEL S. TURNER, DANIEL B. FAMBRO, AND RAMEY O. ROGNESS

The shoulder is one of the most extensively studied roadway elements; however, its effectiveness in reducing accidents has been the subject of much debate. A study is described in which accident rates and characteristics were compared for three different types of rural Texas highways: two-lane roadways without paved shoulders, two-lane roadways with full-width paved shoulders, and four-lane undivided roadways without paved shoulders. Approximately 30 roadways of each type were selected for the study. A rigorous screening procedure was developed to ensure that each site was a "typical" Texas roadway. A detailed three-year accident history was obtained for each site. More than 1250 km of highway and 16 000 accidents were included in the study data base. For each roadway type, accident rates increased as traffic volume increased. Two-lane highways without paved shoulders had the highest accident rates and were the most sensitive to changes in traffic volume. Two-lane highways with paved shoulders had the lowest accident rates until the traffic volume reached 7500 vehicles/day. At that point, four-lane undivided highways without paved shoulders were safer. Based on the study findings, it was concluded that full-width paved shoulders are effective in reducing the accident rate on rural highways. It also appears that the presence of full-width paved shoulders may reduce the number of rural intersection accidents.

Today's highway engineers are faced with the dual dilemma of inflationary construction costs and reduced revenues, which necessitates the careful selection of new projects based on the optimum use of existing funds. The Texas State Department of Highways and Public Transportation (TSDHPT) has tried several innovative techniques in order to maximize the use of fiscal resources. One such technique has been to provide additional capacity at minimum expense by converting two-lane roadways with full-width paved shoulders to four-lane roadways without shoulders. This treatment, which has become known as the "poor-boy" highway, entails resurfacing and restriping or simply restriping the existing pavement. Expenses for earthwork, drainage, intersections, and structures are minimized. A poor-boy highway is typically undivided, has no shoulder, and has a paved travel surface from 44 to 48 ft wide.

A poor-boy is certainly an inexpensive alternative to upgrading a two-lane highway to a four-lane divided highway. However, the poor-boy concept is not a standard treatment, and there is currently a limited amount of such mileage in the state. Figure 1 compares the amount of four-lane roadway without shoulders with the amount of two-lane roadways in Texas. As the figure shows, Texas has 69 750 km (43 350 miles) of two-lane highways without shoulders. They are characterized by small traffic volumes. In fact, 95 percent of these roads carry fewer than 2000 vehicles/day. There is also 21 000 km (13 050 miles) of two-lane highways with full-width paved shoulders. Most of these roadways carry 1000-3000 vehicles/day. The state has only 900 km (560 miles) of four-lane roadways without shoulders. These roadways are most heavily concentrated in the range of 1000-5000 vehicles/day. For the remainder of this paper, the terms poor-boy and four-lane highway without shoulders are used interchangeably.

SCOPE OF STUDY

The TSDHPT does not have a documented data base for establishing design policies and practices for the upgrading of two-lane highways without paved shoulders to two-lane highways with paved shoulders or for upgrading two-lane highways with paved shoulders to poor-boy highways. One of the purposes of this research was to establish the accident effects related to the presence or absence of a paved shoulder. The planning division of TSDHPT considers any paved shoulder less than 6 ft wide as "none or inadequate" and codes the state's computerized geometric files in that manner. The same definition for shoulder was adopted for this study. The study was limited to rural Texas highways of the types shown in Figure 2.

BACKGROUND

The shoulder is one of the most extensively studied roadway elements; however, safety findings have not always been consistent. This has caused considerable confusion about the exact relation between shoulder characteristics and accident experience. Previous studies can be placed into three general groups: those that find that wider shoulders have adverse effects, those that indicate that wider shoulders have unclear or null effects, and those that indicate that wider shoulders have favorable effects.

Belmont's studies in California (1,2) and Blensly and Head's study in Oregon (3) found increases in accident rate, property damage, and accident severity for certain types of wide-shoulder conditions. Investigations by Perkins (4), Taragin and Eckhardt (5), Raff (6), and Foody and Long (7) indicated that shoulder effects on accidents were marginal or insignificant. Other researchers, including Stohner (8), Jorgensen (9), and Zegeer and Mayes (10), found a reduction in accident rates due to wider shoulders. These studies may be considered a partial list, since there are many other research efforts in the field.

It often seems that different conclusions have been reached in nearly every study on the subject. This diversity of opinion can be explained in part by consideration of the scope of the various studies. Many of the researchers examined data in limited areas of volume, geometry, and operational

Figure 1. Total mileage in Texas for different highway classes.

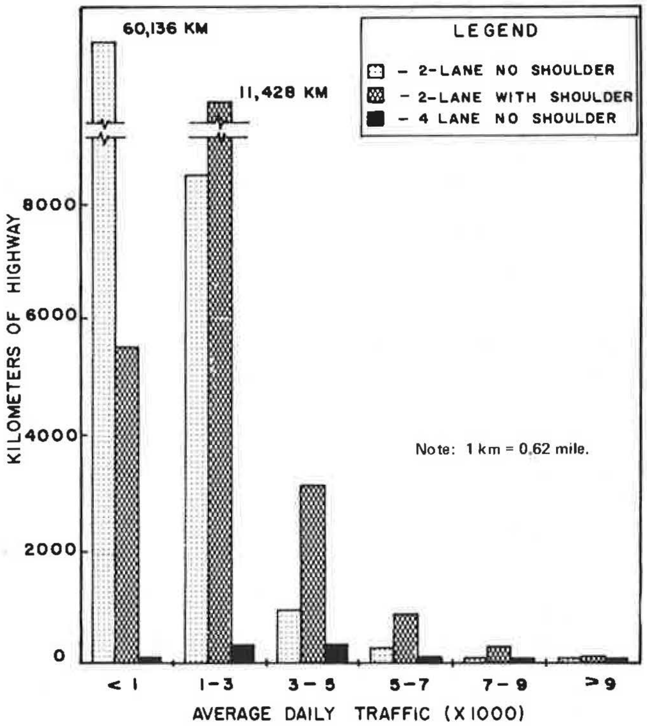


Figure 2. Typical examples of three highways classes: (top) two lanes with unpaved shoulders, (middle) two lanes with paved shoulders, and (bottom) four lanes with unpaved shoulders (poor-boy).

characteristics. It is necessary to isolate and examine all other roadway and operational characteristics in order to correctly determine the effect of paved shoulders on accident experience (11). The majority of the differences can be traced to flaws in gathering and interpreting the data. Because accidents normally occur infrequently, very large data samples are required to discern their effects. It is often impossible to find suitable sites without changes in traffic volumes, geometry, or accident reporting practices. The interactive nature of accident elements also creates complexity and makes analysis more difficult. The combined effects of the data constraints can be considered the major reason for the lack of harmonious results.

Recent research efforts have observed stricter controls in establishing and evaluating data. Heimbach, Hunter, and Chao (12) undertook a very thorough classification and matching procedure to study shoulders in North Carolina. They concluded that significantly less accident experience is associated with homogeneous highway sections that have 0.9- to 1.2-m-wide (3- to 4-ft-wide) paved shoulders than with identical highway sections that do not have shoulders. They also found that analyses of cost-effectiveness and investment return could be used to determine when to add shoulders to highways. Fambro, Turner, and Rogness (13) used a large data sample of Texas roadways to determine whether the presence of paved shoulders decreased accidents. Their conclusion was similar to that of Heimbach in that two-lane roadways with paved shoulders were found to have fewer accidents than roadways of the same type without shoulders. Rinde (14), using a before-and-after technique in California, found a reduction in accidents due to the addition of shoulders. He suggested optimum widths of pavement for various combinations of roadway type and volume. Several researchers (12,15,16) have gone a step further and have quantified procedures to determine when to add shoulders and how to establish the cost-effectiveness of such additions. The current emphasis on the absence or presence of paved shoulders, rather than the width of such shoulders, has clearly established their positive contribution in reducing accidents.

To summarize the literature review, it appears that, after conflicting earlier studies, better control of the data population has established the desirability of providing paved shoulders for rural highways. Recent research has found a decrease in accidents due to the presence of paved shoulders.

ACCIDENT INVESTIGATION

The level of safety performance is one of the major characteristics of a roadway. Traffic engineers routinely perform a variety of accident analyses in order to determine safety characteristics. Although there are many types of analyses, the most common purposes for such studies are to identify hazardous locations, examine the aptness of proposed corrective measures, or evaluate the effectiveness of previous improvements. In addition, studies are often performed to compare the safety characteristics of different types of roadways.

The characteristic patterns of traffic accidents exhibit only slight changes from year to year (17, p. 281). As long as roadway geometrics and traffic volumes remain relatively uniform, accident patterns should display the same general trends. Thus, the patterns applicable to specific roadways can be isolated and identified if the researcher uses care in selecting and analyzing the data. Accidents are assumed to be random occurrences that can be described by statistical theory and are thus quite sensitive to sample size and the bias of the observer. Jorgensen's landmark report on accident analysis procedures (9) points out that small sample sizes and random variations in nonroadway characteristics may seriously bias the results of a study. Jorgensen also states that the data must

Table 1. Site-selection data.

Class	Candidate RI-2-TLOG Segments Number	Candidate RI-2-TLOG Segments Length (km)	No. of Potential Sites	Selected Sites Number	Selected Sites Length (km)
1	10 029	8 376.10	152	10	182.60
2	1 441	938.23	32	10	95.48
3	440	218.36	11	8	68.16
4	10 515	11 428.52	260	10	219.32
5	3 969	3 033.52	51	10	193.07
6	1 346	831.26	32	10	162.56
7	554	358.89	19	9	119.29
8	253	76.96	6	4	31.35
9	150	52.69	7	4	33.91
10	453	300.83	15	10	148.93
Total	29 150	25 615.36	585	85	1254.67

Note: 1 km = 0.62 mile.

cover a sufficient period of time to give a statistically valid sample.

To ensure that there was a sound statistical basis for site selection and that the sample would be representative of Texas highways, three years of accident data were gathered for a series of carefully selected locations. Three years were selected because Gwynn (18) has shown that using more than three years of accident data does not significantly improve the statistical accuracy of such studies. The steps that were followed to select the sites and collect the data for the accident investigation are described in detail below.

Site Selection

A matrix of desired characteristics was created to stratify the sites by traffic volume, shoulder type, and number of lanes. The table below gives the nine classifications created to allow a comparative analysis of the effects of these variables on accident rates and characteristics (ADT = average daily traffic):

Class	Type of Highway	Type of Shoulder	ADT
1	Two-lane	Unpaved	1000-3000
2			3000-5000
3			5000-7000
4	Two-lane	Paved	1000-3000
5			3000-5000
6			5000-7000
7	Four-lane	Unpaved	3000-5000
8			5000-7000
9			7000-9000

To ensure a large and representative data sample, 10 sites in each class were studied, and each site contained 5 or more miles of consistent roadway.

Once the general site criteria had been defined, all roadways in Texas were screened as potential sites through a computer listing of the roadway geometric file, commonly referred to as the RI-2-TLOG. The file was carefully reviewed to obtain a list of potential rural sites that fit the requirements outlined in the table above. The initial screening was a substantial undertaking that involved an evaluation of more than 29 000 roadway segments.

The selection process is summarized in Table 1. A listing of the number of segments of Texas roadway in each study classification is given. Clearly, there was not a uniform distribution of segments among the classes. For instance, it was found that more than 77 percent of the total eligible kilometers of roadway fell in classes 1 and 4 (two lanes, with and without shoulder, and ADT of 1000-3000). Thus, the majority of candidate sites could be described as light-traffic, two-lane rural highways. On the other hand, only 3 percent of the total kilometers consisted of poor-boy highways (classes 7, 8, 9, and 10).

During this portion of the study, it became apparent that 10 study sites could not be obtained for some classifications. In fact, only 8 sites were found to meet the qualifications of class 3, 9 sites for class 7, and 4 sites each for classes 8 and 9. Classes 7-9 were restricted due to the limited quantity of poor-boy mileage in the state. At this point, it was decided to add a tenth class to the study (four lanes, unpaved shoulder, and 1000-3000 ADT) to handle low-volume poor-boy roadways. The addition of this category allowed direct comparison of accident rates at low traffic volumes for the three types of highways in the study.

A great deal of effort was expended in checking the eligible sites to ensure that they were typical of their respective categories. For example, ADT was reviewed for each location to verify that no major changes had occurred during the three-year study period. Pavement widths and shoulder types were scrutinized for uniformity for both each site and each class. Divided roadways were deleted from the investigation. Careful reviews were conducted by using county, highway district, and state maps to isolate and remove sites that contained major intersections, towns, or other factors that might bias the results of the study.

The 85 selected sites were taken from the original group of more than 29 000 segments contained in the RI-2-TLOG. Table 1 clearly shows several stages of refinement in achieving a representative sample of rural highways spread over a wide geographic area. The abundance of low-volume, two-lane roadways (classes 1 and 4) produced more than 400 potential sites, and the research staff was able to choose roadways of considerable length that had an optimal geographic distribution. On the other hand, the limited amount of poor-boy mileage (classes 7-10) caused a corresponding reduction in the number of sites available for study. It should be noted that at least 25 percent of the counties in the state were represented by at least one study site.

Collection of Accident Data

Once the sites had been finalized, accident data were gathered for the three most recent years available (1975-1977). The computerized accident files of TSDHPT were scanned for all accidents that occurred at the 85 sites during the three-year period. A total of 16 334 accidents were included in the study data base. Each comprehensive accident record contained 393 characters of information about the collision and the roadway on which it occurred. The detailed accident information allowed a very thorough examination and cross classification of the data base. Records were reviewed, and locations that exhibited unusual conditions were removed. For example, construction zones, unusual traffic-control devices, and railroad crossings were found to be major contributors to the accident rate at various locations. In each case, the study site was shortened to remove atypical situations.

Geometric data were taken from the RI-2-TLOG to calculate the vehicle miles of travel for each site. These figures were combined with the number of accidents on each roadway to yield accident rates. Since the number of intersections varied from site to site, a separate analysis was performed on nonintersection accidents to isolate and remove

Figure 3. Accident rates for different highway classes (all accidents).

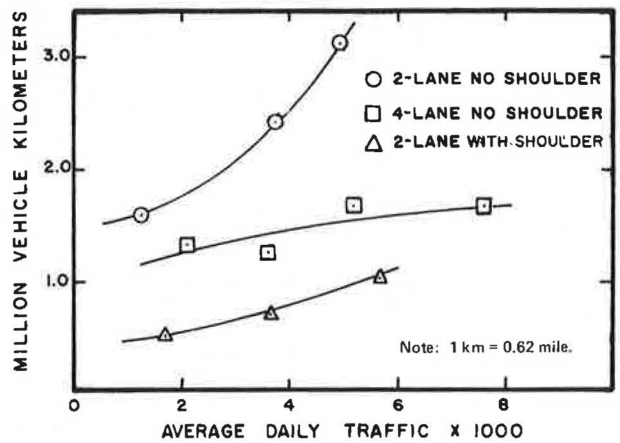
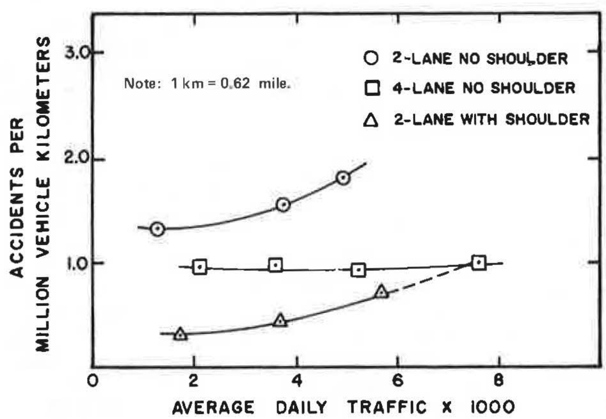


Figure 4. Rates of nonintersection accidents for different highway classes.



the disproportionate effects that this type of accident might cause among sites in the various classifications. Intersection accidents were identified from coded accident data. Without intersection accidents, the data base contained 8815 accidents. The two data sets are referred to here as the all-accident group and the non-intersection-accident group.

ACCIDENT RATES

Figures 3 and 4 show the results of the investigation of accident rates for all accidents and for nonintersection accidents, respectively. The two figures are similar: The non-intersection-accident curves generally have lower values and flatter slopes than the all-accident curves. The implications of the two figures are discussed below.

All Accidents

The most obvious feature of Figure 3 is that the accident rate tends to increase as ADT increases for all three types of roads in the study. The highest accident rate is associated with two-lane roads without shoulders. This type of road also has the curve with the steepest slope. Thus, it is the most sensitive to increases in ADT. The curve becomes very steep and approximates a straight line at traffic volumes greater than 4000 vehicles/day. In this region, the two-lane, no-shoulder roads have a distinctly higher accident rate than either of the other two road classes.

It is important to note that the lowest accident rate is associated with two-lane roads that have paved shoulders. The curve for this type of road has a gentle slope and is slightly parabolic in nature. The curve for the third type of road in the study, four lanes without shoulders, falls in between the curves for the other two types, being safer than two-lane roads without shoulders and not as safe as two-lane roads with shoulders.

It would seem that all four-lane roads should be safer than all two-lane roads, yet Figure 3 clearly indicates otherwise. The addition of paved shoulders to two-lane roads makes them safer than four-lane roads without shoulders (poor-boy). The benefit of adding paved shoulders is indicated in Figure 3 by sharply reduced accident rates. There are several logical reasons for these safety benefits. Paved shoulders provide recovery areas for vehicles that accidentally leave the roadway, provide a refuge for stalled vehicles, provide acceleration and deceleration lanes for right-turning vehicles, and instill a feeling of comfort and security in drivers. Conversely, there are reasons why poor-boy highways have relatively higher accident rates. When the paved shoulders are converted to travel lanes, the immediate recovery zone is removed for vehicles that accidently exit the travel lanes. The clear zone adjacent to the pavement is not extended outward and thus fixed objects are left in the recovery area. The recovery maneuver becomes more difficult and consequently leads to more accidents of the run-off-road and fixed-object varieties. In addition, the paved four-lane road encourages higher speeds at lower traffic volumes. This is only a partial list of reasons why two-lane roads with shoulders are safer than either of the other two types of roads in the study.

In summary, two major conclusions can be drawn from Figure 3. The first is that accident rates increase with ADT for all three types of roads in the study. The second finding is the position of the accident-rate curves. In descending order of accident rates, they are two-lane roads without shoulders, four-lane roads without shoulders, and two-lane roads with shoulders.

Nonintersection Accidents

The data on nonintersection accidents were summarized, and the results are shown in Figure 4. Highways in classes 1-3 (two-lane roads without shoulders) again displayed the highest accident rates. The figure shows that, up to an ADT of 5000 vehicles/day, the accident rate increases in a curvilinear manner. The curve for two-lane roads with shoulders (classes 4 and 6) has the same general shape and is generally parallel to the no-shoulder curve; however, the roads with shoulders have only two-thirds as many accidents as the roads without shoulders. The addition of shoulders to two-lane highways offers definite safety benefits by decreasing nonintersection accidents.

Once again, the accident rates for the four-lane roads without shoulders fell in between those of the other two types of roads. The curve has a gently ascending slope with a distinctly lower rate of increase than that for two-lane roads. In fact, the four-lane curve is nearly horizontal, and this implies that the accident rate is fairly uniform over the range of traffic volumes included in this study.

The curves for poor-boy roads and two-lane roads with shoulders appear to converge at higher volume levels. If the two-lane curve is extended, it may be noted that, at an ADT of about 7500 vehicles/day, the accident rate is the same for both types of roadways. The dashed extension of the lowest curve in Figure 4 indicates the location of this theoretical point of intersection. The data indicate that two-lane roads with shoulders are safer than poor-boys at all ADTs below that level. Based on these observations, consideration should be given to using poor-boy highways for ADTs greater than 7500 and two-lane roads with shoulders that carry lower traffic volumes if accidents are the major or only consideration for upgrading a facility.

Comparison of All Accidents and Nonintersection Accidents

The removal of intersection accidents caused the curves shown in Figure 3 to generally move downward and to the right. This signifies a lower accident rate, as would be expected. The different types of roads do not all experience the same amount of change. For example, two-lane, no-shoulder roads undergo a very noticeable alteration. The curve for nonintersection accidents is significantly displaced to the right and has a much flatter slope. This indicates that such roads are apparently very sensitive to intersection accidents, especially as volume increases. At an ADT of 2000 vehicles/day, there is only about a 10 percent difference in the two curves. This difference reaches 50 percent at an ADT of 6000 vehicles/day. This suggests that the construction of paved shoulders at rural intersections may be effective in reducing the accident rate on roads with high traffic volumes.

The same intersection-accident trend is also present for poor-boy highways. The curve for nonintersection accidents has a much flatter slope than the curve for all accidents. At low volume levels the curves are very close to the same value, but as ADT increases the curves diverge. When the ADT reaches 8000 vehicles/day, intersection accidents add 30 percent to the accident rate. This again points to increased intersection-accident sensitivity as volume increases.

The least amount of change is experienced by two-lane roads with shoulders. The curves for all accidents and nonintersection accidents are virtually parallel, which indicates that ADT has very little effect on the rate of intersection accidents. Thus, it appears that the presence of shoulders has a beneficial effect on reducing intersection accidents: For the two types of roads without shoulders, there were large changes in accident rates between the two data sets whereas, for the roads with shoulders, there were only minor changes between data sets.

ACCIDENT CHARACTERISTICS

The types of accidents that occur on a specific roadway or at a specific site often yield clues to the nature of safety-related problems. Corrective procedures are often formulated based on analyses of accident characteristics. The pertinent findings of the investigation of accident characteristics are summarized below.

Severity Analysis

Traffic accidents are usually placed in one of three severity classifications: fatal, injury, or property-damage collisions. The accident-rate curves shown in Figures 3 and 4 can be considered to represent inclusive property-damage curves. Curves for injury and fatal accidents have been plotted in Figure 5. The solid lines depict the rates of injury accidents and fall into a familiar pattern. The curves seem to display relations discussed previously in relation to Figures 3 and 4--i.e., accident rate rising as volume increases. The highest rates are associated with two-lane highways without shoulders and the lowest rates are for two-lane highways with shoulders. The curves for two-lane highways are approximately parallel, and both are parabolic in nature. The curve for four-lane roads without shoulders remains between the two-lane curves. The high degree of similarity between the curves for property-damage accident rate and the curves for injury-accident rate indicates that there are no unusual changes in the injury pattern as volume increases and that analysis of trends in the rate of property-damage accidents will suffice for analysis of trends in the injury-accident rate for the purpose of this study.

Curves for fatal-accident rate are shown by the dashed lines in Figure 5. The data points all fall in close proximity to one another, and there is not a great distinction between the curves for the three types of roads. The slopes are almost flat, which indicates that the fatal-accident rate is fairly constant for volumes between 1000 and 8000 vehicles/day. For the two types of roads without shoulders, there is a slight diversion from this trend in the area of 5000 vehicles/day. The implications of this diversion are not clear, but it could be that two-lane roads without shoulders have passed a critical volume level and have become slightly more susceptible to high-severity accidents. The lowest rates are for two-lane roads with shoulders, where the curve remains around 0.04 fatal accidents per million vehicle kilometers (0.06 accidents per million vehicle miles) of travel. This curve is virtually a straight line.

It is difficult to draw conclusions from the fatal-accident plot due to the low rates and data convergence. Perhaps the most significant conclusion would be the uniformity of the fatal-accident rate. The fact that these rates remain constant over a wide volume range while the other types of accidents increase implies that the additional accidents at high volume levels are not high-severity accidents.

Figure 6 shows the relation among the three accident severity rates for a single class of road (two-lane road with shoulder) for nonintersection accidents. Similar relations exist for the other types of roadways in the study. The position of the curves in Figure 6 indicates the relations of rates of total, injury, and fatal accidents. The rate of fatal accidents is 12-15 times lower than the total

Figure 5. Rates of injury and fatal accidents for different highway classes (nonintersection accidents).

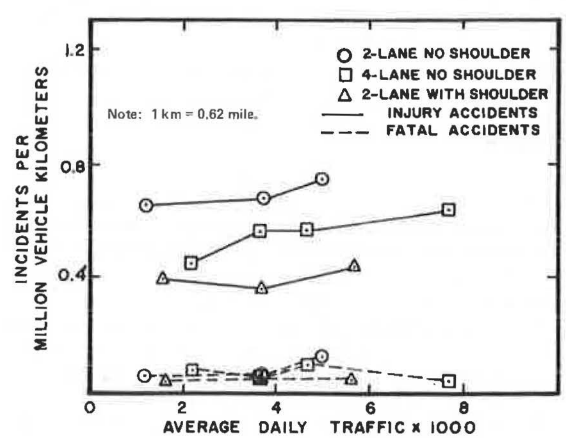


Figure 6. Severity of three types of accidents on two-lane roads with shoulders (nonintersection accidents).

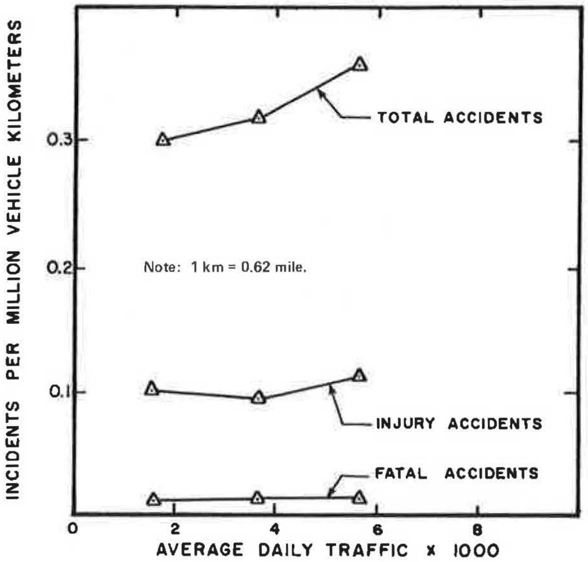
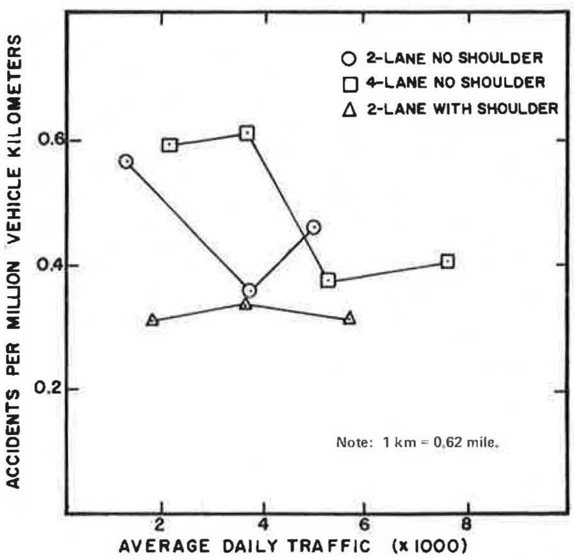


Figure 7. Rates of run-off-road accidents for different highway classes (nonintersection accidents).



accident rate, and the rate of injury accidents is approximately three times lower than the total accident rate.

The change in slope for the three curves in Figure 6 indicates their sensitivity to traffic volume. The most sensitive (steepest-sloped) curve is for total accidents, the next most sensitive is for injury accidents, and the least sensitive is for fatal accidents. This reinforces the previous observation that additional accidents at high traffic volumes are not high-severity accidents.

Run-Off-Road Accidents

The lack of a suitable recovery area for vehicles that leave the travel lanes should cause an increase in the run-off-road type of accident. This premise is supported by Figure 7, which indicates that two-lane roads with shoulders have the lowest level of occurrence of such accidents and a uniform level of incidence over a wide range in traffic volumes. On the other hand, both types of roadways without shoulders exhibit higher and more erratic rates of run-off-road accidents. The fluctuation of these curves would seem to conform to the level of driver workload. At low traffic volumes, driving is an easy task that requires few decisions and drivers are likely to become inattentive. As volume increases, drivers are forced to make more frequent decisions as they meet additional oncoming traffic and handle various driving situations. Thus, they are forced to become more attentive. At high traffic volumes, the driver workload becomes very heavy, requiring almost constant evaluation of vehicle position, speed, and similar items. The driver is likely to become fatigued or to miss important stimuli that might warn of approaching danger.

Driver inattentiveness at low traffic volumes could account for the high run-off-road accident rate shown in Figure 7. As volume and driver attentiveness increase, the rate drops for both types of highways without shoulders. For two-lane roads, this drop occurs in the 1000- to 3000-vehicles/day range. For four-lane roads, the additional paved lane allows drivers additional lateral freedom and decreases their workload. Thus, the increase in attentiveness does not occur until the volume level reaches 4000-5000 vehicles/day. During the period of increased attentiveness, both types of roads have lower incidence rates (for midvolume ranges). As the two-lane road reaches relatively high traffic volumes, the rate of run-off-road accidents begins to increase, probably due to driver work overload and high-volume accident situations that force the driver to make emergency maneuvers away from the normal travel lane. This is indicated by the rising accident rate around 5000 vehicles/day. For four-lane roads, this point has not been reached at the traffic volumes shown in Figure 7; however, the poor-boy curve is beginning to climb (at 8000 vehicles/day), which indicates that the rate of run-off-road accidents is beginning to increase with increasing volume. It could well be that the four-lane curve peaks at high volume in the same manner that the curve for two-lane, no-shoulder roads peaked at 5000 vehicles/day.

Types of Collisions

Another meaningful accident characteristic is the first harmful event in a collision sequence. Figure 8 shows how many times hitting another car was the first harmful event. A number of conclusions can be drawn from this figure. First, the increasing slopes of the lines indicate that the chances of hitting another vehicle increase as traffic volumes increase. Such is to be expected, since there are more vehicles on the highway as the volume increases. At low volumes, more vehicles are running off the road than are hitting other vehicles. A comparison of the curves for four-lane accident rate in Figures 7 and 8 shows that approximately twice as many vehicles run off the road as hit other cars at a volume level of 2000 vehicles/day. At volumes less than 4000 vehicles/day, the run-off-road accident is dominant.

Perhaps the most important finding that can be drawn from Figure 8 is the almost identical curves for two-lane roads with shoulders and the poor-boy roads. The curves are parallel and lie almost on top of one another. This indicates that both types of roads exhibit the same behavior for hit-other-car accidents. It is highly probable that the paved shoulders and extra paved travel lanes both serve the same function by providing additional maneuvering room for vehicles during the collision sequence.

Figure 9 emphasizes the increase in hit-other-car

Figure 8. Rates of hit-other-car accidents for different highway classes (non-intersection accidents).

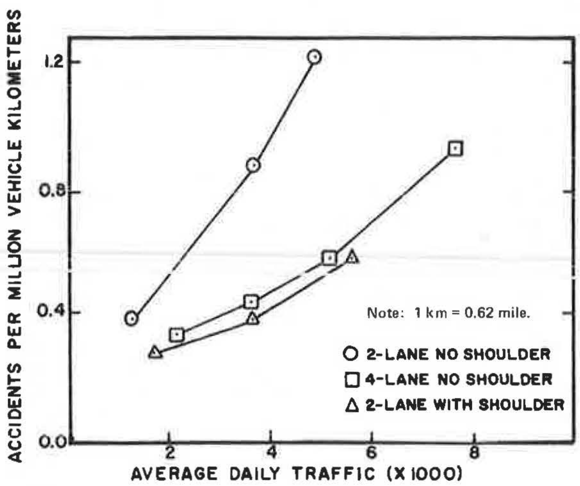
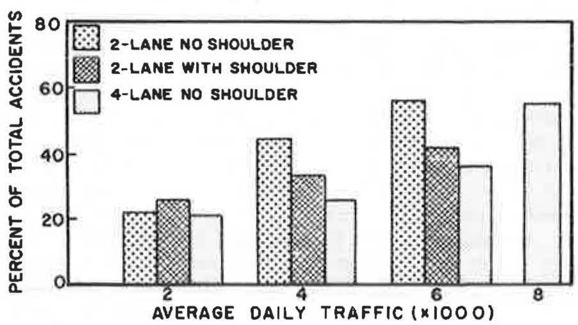


Figure 9. Increase in hit-other-car accidents with increasing traffic volume for different highway classes.



accidents as volume increases. The percentage of accidents that involved hitting another vehicle climbs steadily as traffic volume increases. As Figure 9 shows, the same trend exists for all three types of roads in this study. Both the percentage and the rate of hit-other-car accidents increased similarly for all three types of roads.

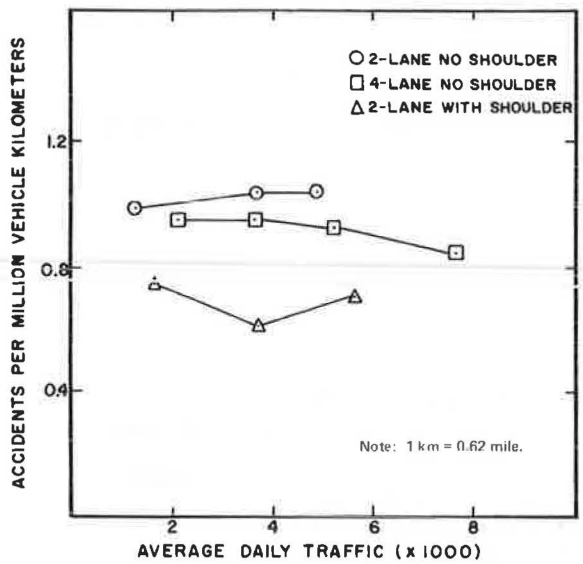
Fixed-Object Accidents

A fixed-object incident is often found in the chain of harmful events in a collision sequence. These types of accidents were examined and found to closely resemble the run-off-road accident pattern, especially for four-lane highways without shoulders and two-lane highways with shoulders. At low traffic volumes, the four-lane road exhibited a high rate of fixed-object collisions. The rate began to decrease as the volume rose above 4000 vehicles/day in the same manner noted earlier for run-off-road accidents. At 8000 vehicles/day, the rate of fixed-object accidents had dropped by 50 percent. This echoes the previous premise that accidents on low-volume, four-lane roadways are usually not the multiple-vehicle type but rather some function of driver inattentiveness and lack of paved recovery area.

Nondaylight Accidents

Figure 10 shows the rate for the number of nondaylight accidents compared with total vehicle travel

Figure 10. Rate of nondaylight accidents for different highway classes (non-intersection accidents).



(both daylight and nondaylight). The three curves hold their familiar position. The most dangerous type of road is the two-lane highway without shoulders, and the least dangerous is the two-lane highway with shoulders. The curves are fairly flat, which indicates uniform accident rates for all roadway types regardless of traffic volume. It is noteworthy that a disproportionate number of the accidents occurred at night. This is especially true for four-lane roads. For example, by comparing accident rates in Figures 4 and 10, it may be noted that approximately 60 percent of all accidents on four-lane roads occurred during nondaylight hours. The vast majority of rural traffic occurs during the day, which means that 60 percent of the accidents result from the small percentage of total traffic that occurs at night and that four-lane highways without shoulders are significantly more dangerous at night than during the day.

CONCLUSIONS

Based on the findings from this study, several conclusions can be drawn. These conclusions involve both accident rates and accident characteristics for three types of rural Texas highways and are discussed below.

Accident Rate

When all accidents that occurred at the study sites were considered, the accident rate for each roadway type increased as the traffic volume increased. Two-lane highways without paved shoulders have the highest accident rates and are the most sensitive to changes in traffic volume. Two-lane highways with paved shoulders had the lowest accident rates for the traffic volumes investigated in this study (1000-7000 vehicles/day). The third type of road investigated, four-lane roads without shoulders, was the least sensitive to traffic volume and had an accident rate that fell in between the rates for the other two types of roads. The presence of paved shoulders had a noticeable effect in reducing the accident rate.

When intersection accidents were removed from the data set, the road types retained the same rank: Two-lane roads without shoulders were the most dangerous, and two-lane roads with shoulders were the least dangerous. Four-lane roads were found to have a fairly uniform accident rate regardless of traffic volume, whereas the curves for the other two roads were almost parallel but noticeably different. Full-width paved shoulders were again shown to have positive effects in reducing accident rates. An extension of the data indicated that two-lane highways with paved shoulders had lower accident rates than poor-boy highways at traffic volumes of less than 7500 vehicles/day.

A comparison of the non-intersection-accident data group with the all-accident data group indicated that two-lane highways without paved shoulders are very sensitive to intersection accidents, especially at high traffic volumes. Four-lane roads are somewhat sensitive and two-lane roads with paved shoulders are relatively insensitive to such accidents. Thus, it appears that the construction of full-width paved shoulders at rural intersections may be effective in reducing the number of accidents on high-volume roads.

Accident Characteristics

The severity analysis revealed that curves for injury-accident rate are approximately parallel to curves for the total accident rate whereas curves for fatal-accident rate are very low and fairly uniform for all three types of roads over the range of traffic volumes included in the study. The lowest fatality rate was 0.04 fatal accidents per million vehicle kilometers of travel (0.06 accidents per million vehicle miles) for two-lane highways with paved shoulders.

The absence of full-width paved shoulders increased the rate of run-off-road accidents, especially at low traffic volumes. The two-lane roads with paved shoulders had a fairly uniform rate of run-off-road accidents, whereas the rates for the other two roads were of a higher level and varied considerably with volume. The most probable reason for the variations and the high run-off-road rates at low volumes is driver inattentiveness and lack of a paved recovery zone for vehicles that accidentally exit the travel lane.

Hit-other-car accidents tend to increase drastically with increasing traffic volumes; two-lane, no-shoulder highways have a greater incidence of these accidents than the other two highway types. The fourand two-lane highways with shoulders have virtually identical curves for hit-other-car accidents. At lower volumes, other types of collisions were more frequent. For example, on four-lane highways, the run-off-road accident is the most frequent type of accident at all volumes less than 4000 vehicles/day. On roads without shoulders, rates for fixed-object accidents were found to closely resemble rates for run-off-road accidents. Again, this reflects the lack of a paved recovery area and the presence of fixed objects in the clear zone adjacent to the roadway.

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The contents of this paper reflect our views, and we are responsible for the facts and accuracy of the data presented. The contents do not necessarily reflect the official views or policies of FHWA. This paper does not constitute a standard, specification, or regulation.

REFERENCES

1. D.M. Belmont. Effects of Shoulder Width on Accidents on Two-Lane Tangents. HRB, Bull. 91, 1954.
2. D.M. Belmont. Accidents Versus Width of Paved Shoulders on California Two-Lane Tangents: 1951 and 1952. HRB, Bull. 117, 1956, pp. 1-16.
3. R.C. Blensly and J.A. Head. Statistical Determination of Effect of Paved Shoulder Width on Traffic Accident Frequency. HRB, Bull. 240, 1960.
4. E.T. Perkins. Relationship of Accident Rate to Highway Shoulder Width. HRB, Bull. 151, 1956.
5. A. Taragin and H.G. Eckhardt. Role of Highway Shoulders in Traffic Operation. HRB, Bull. 151, 1957.
6. M.S. Raff. Interstate Highway Accident Study. HRB, Bull. 74, 1953.
7. T.J. Foody and M.D. Long. The Identification of Relationships Between Safety and Roadside Obstructions. Ohio Department of Transportation, Columbus, 1974.
8. W.R. Stohner. Relation of Highway Accidents to Shoulder Width on Two-Lane Rural Highways in New York State. Proc., HRB, Vol. 35, 1965, pp. 500-504.
9. Evaluation of Criteria for Safety Improvements on the Highway. Roy Jorgensen and Associates, Inc., Gaithersburg, MD, 1966.
10. C.V. Zegeer and J.G. Mayes. Cost Effectiveness of Lane and Shoulder Widening on Rural, Two-Lane Roads in Kentucky. Kentucky Department of Transportation, Frankfort, Res. Rept. 999, 1979.
11. J.M. Portigo. State-of-the-Art Review of Paved Shoulders. TRB, Transportation Research Record 594, 1976, pp. 57-64.
12. C.L. Heimbach, W.W. Hunter, and G.C. Chao. Paved Highway Shoulders and Accident Experience. Transportation Engineering Journal, Proc., ASCE, Vol. 100, No. TE4, 1974, pp. 889-907.
13. D.B. Fambro, D.S. Turner, and R.O. Rogness. Operational and Safety Effects of Driving on Paved Shoulders in Texas. Texas Transportation Institute, Texas A&M Univ., College Station, Res. Rept. 265-2F, Sept. 1980.
14. E.A. Rinde. Accident Rates Versus Shoulder Width. California Department of Transportation, Sacramento, Rept. CA-DOT-TR-3147-1-77-01, 1977.
15. D.I. Davis. Shoulder Improvements on Two-Lane Roads. TRB, Transportation Research Record 737, 1979, pp. 59-60.
16. C.V. Zegeer and D.D. Perkins. Effect of Shoulder Width and Condition on Safety: A Critique of Current State of the Art. TRB, Transportation Research Record 757, 1980, pp. 25-34.
17. L.J. Pignataro. Traffic Engineering: Theory and Practice. Prentice-Hall, Inc., Englewood Cliffs, NJ, 1973.
18. D.W. Gwynn. Accident Rates and Control of Access. Traffic Engineering, Nov. 1966.

Use of Box and Jenkins Time-Series Analysis to Isolate the Impact of a Pavement Improvement Policy

NANCY L. NIHAN AND KJELL O. HOLMESLAND

The impact of a freeway pavement improvement policy, in terms of its actual effects on freeway volumes and the duration of those effects as well as the effect of this intervention on the accuracy of model forecasts, is analyzed. Univariate and multivariate time-series models are used for the impact analysis. Both types of models exhibit accurate volume forecasts. Two approaches to isolating the impact effect are used. One approach is to fit a univariate time series model to predict trends prior to the intervention. A second approach fits a time-series model with intervention variables to the entire series. Both models yield similar results in terms of measured volume impacts.

This paper addresses two major issues concerning the use of time-series analysis in traffic planning. First, this study and a previous one by Nihan and Holmesland (1) indicate that time-series models can yield accurate short-term forecasts of traffic volumes for operational planning. The fact that such models appear to be quite accurate and are also inexpensive and easy to run suggests the need for further development in this area of traffic forecasting.

Time-series techniques can also be used to isolate the impact of certain policy interventions on traffic volumes. By using such techniques, we can statistically determine not only the impact itself but its duration as well. This second issue is of considerable importance to those involved in the day-to-day decisions that affect traffic operations. This paper shows how time-series analysis can be used to study the impact of one such decision. The policy to be studied is a three-month, 24-h pavement improvement program for a freeway bridge crossing that occurred during the months of July, August, and September 1975 in Seattle, Washington.

STATE OF THE ART

Time-series models have proved to be accurate forecasters in modeling socioeconomic phenomena in other disciplines. They have been very successful, for example, in economics, a field that is related to transportation in many respects since economic activity creates many of the conditions that encourage transportation. Economic time-series models have been shown to forecast quite accurately (2-6). Given the positive results in this related area, it stands to reason that this approach would be useful to traffic engineers. Yet little time-series modeling has occurred in traffic planning. This is somewhat puzzling, since traffic-volume data provide an excellent basis for time-series analysis. A few traffic studies have been performed by using the widely accepted Box and Jenkins time-series technique (7, p. 44) used in the current study. From these preliminary studies, the use of the Box and Jenkins technique for forecasting and determining the effects of external interventions (8) appears promising. The current study expands on this set of traffic studies.

Among the studies observed in transportation is one by Holmesland (9) that showed that traffic data are time-dependent and that dynamic models are needed in this area. Another study was conducted by Hammatuck (10) in Madison, Wisconsin, to determine the impact of a one-month transit strike on total transit revenue. The results demonstrated clearly that the method is applicable. There have also been three studies of the impact of interventions on traffic accidents. Atkins (11) looked at the effect of a speed-limit change on fatalities on freeways in British Columbia. Another study by Bhattacharyya and Layton (12) analyzed the effect on accidents of the safety-belt law introduced in the state of Queensland, Australia. A third study by Wiorkowski and Heckard (13) dealt with the impact of alcohol legislation on accidents in Texas. Finally, Box and Jenkins models have also been used in estimating aggregate transit demand in Montreal (14).

An important factor in the above-mentioned transportation studies is the fact that they show time dependency of data series exists in such different areas as traffic volumes, transit ridership and revenues, and traffic accidents.

RESEARCH DESIGN

Notation

Before a discussion of the actual experiments, the notation to be used throughout the remainder of this paper is defined. The symbols for the variables used in the model equations are as follows:

a_t = random noise term (residual) associated with month t,

$\mu_{1t} = \begin{Bmatrix} 0 \text{ if } t \neq \text{July 1975} \\ 1 \text{ if } t = \text{July 1975} \end{Bmatrix}$ = dummy intervention variable for July 1975,

$\mu_{2t} = \begin{Bmatrix} 0 \text{ if } t \neq \text{August 1975} \\ 1 \text{ if } t = \text{August 1975} \end{Bmatrix}$ = dummy intervention variable for August 1975,

$\mu_{3t} = \begin{Bmatrix} 0 \text{ if } t \neq \text{September 1975} \\ 1 \text{ if } t = \text{September 1975} \end{Bmatrix}$ = dummy intervention variable for September 1975, and

z_t = average weekday volume for month t.

The symbols for the operators used in the model equations are as follows:

B = backward shift operator,

$Bz_t = z_{t-1}$,

$B^n z_t = z_{t-n}$,

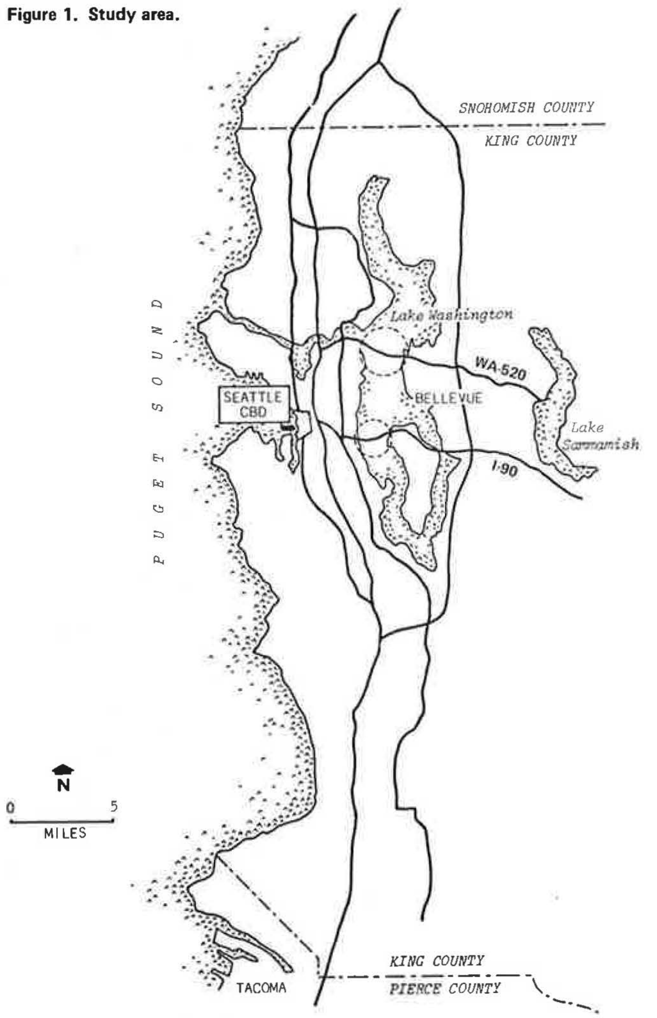
$\nabla\nabla_{12}$ = double difference operator with season of 12 months = $(1 - B)(1 - B^{12})$, and

$\nabla\nabla_n = (1 - B)(1 - B^n)$, where the first difference term is nonseasonal (single difference) and the second difference term is seasonal and represents a seasonal lag of n months.

Study Area

The focus of the study was on traffic volumes on two freeway bridge crossings that serve the Puget Sound metropolitan region (see Figure 1). The two bridges, which cross Lake Washington on I-90 and WA-520, are the only major transportation connectors between the suburban Eastside and the city of Seattle. During July, August, and the first half of September 1975, the I-90 bridge was under repair and severe capacity restrictions were imposed on it. The chosen policy of pavement improvement was a 24-h operation. During the off-peak hours, the bridge was in operation with two lanes (one in each direction) rather than the normal four lanes. During the

Figure 1. Study area.



rush hours, one lane was added for the main direction (since I-90 used reversible lanes during rush hours, this meant two plus one compared with the normal three plus one). In addition to the reduction in the number of lanes, there was also a reduction in the width of lanes, which further restricted capacity per lane.

The purpose of the modeling experiments that follow was to determine whether such a policy had a measurably detrimental effect on traffic volumes between Seattle and the Eastside and whether this effect was of long or short duration.

Data and Models

The two data series used for specification and diagnostic checking for WA-520 and I-90 were average weekday (AWD) traffic volumes per month from January 1968 to March 1980. Of interest were the impact of pavement improvements on I-90 during July, August, and September 1975 on traffic volumes and also the effect this intervention had on the use of the I-90 data base for future volume predictions.

Four types of models were specified for the two data series. These are summarized below:

1. Model A (I-90) was a univariate model fitted to the time series of monthly AWD volumes from January 1968 through December 1977. This model was used to forecast volumes for the months of January 1978 through March 1980 for diagnostic checking. No special treatment for the I-90 pavement improvement was included in this model.
2. Model B (I-90) was a multivariate model fitted to the same time series as model A, but binary policy variables were added to represent the months when pavement improvement was taking place.
3. Model C (I-90) was a univariate model fitted to the time series of monthly AWD volumes from January 1968 to June 1975--i.e., prior to the pavement improvement period. This model was used to forecast volumes from July 1975 to June 1978 to represent the expected volumes if no pavement improvement had occurred.
4. Model D (I-90) was a multivariate model fitted to the time series of monthly AWD volumes from January 1968 to December 1977. Binary variables were added to represent the months when pavement improvement on I-90 was taking place.

Thus, models A and C are univariate models for the I-90 data series fitted to different subsets of the data. Both models A and C ignore the effects of outside intervention (such as pavement improvements) on forecast volumes. Models B and D are multivariate models fitted to the time-series data for the same period months but policy variables were added. Model B is specified for I-90 data, and model D is specified for the WA-520 series. Both models B and D include binary variables to represent the months of the policy intervention of pavement improvement on I-90 (i.e., July, August, and September 1975).

The fitted models A, B, C, and D (Equations 1-4) are given in order below (the standard deviations of the individual coefficients are given below the line):

$$\nabla\nabla_{12} Z_t = (1 - \underset{0.081}{0.228}B - \underset{0.081}{0.562}B^2)(1 - \underset{0.031}{0.93}B^{12})\, a_t \qquad (1)$$

where chi-square = 20.5 on 23 df;

$$(1 + \underset{0.11}{0.15}B - \underset{0.11}{0.27}B^2)(1 + \underset{0.08}{0.73}B^{12})\, \nabla\nabla_{12} z_t = -\underset{1445}{12\,366}\, \nabla\nabla_{12}\mu_{1t} - \underset{1691}{16\,391}\, \nabla\nabla_{12}\mu_{2t} - \underset{1452}{6041}\, \nabla\nabla_{12}\mu_{3t} + a_t \qquad (2)$$

where chi-square = 13.8 on 23 df;

$$(1 + \underset{0.107}{0.3972}B + \underset{0.108}{0.3275}B^2)\, \nabla\nabla_{12} z_t = (1 - \underset{0.039}{0.8955}B^{12})\, a_t \qquad (3)$$

where chi-square = 20.6 on 23 df; and

$$\nabla\nabla_{12} z_t = (1 - \underset{0.095}{0.37}B - \underset{0.095}{0.166}B^2)(1 - \underset{0.036}{0.88}B^{12})\, a_t + \underset{1527}{5239}\, \mu_{1t} + \underset{1521}{1461}B^2\, \mu_{3t} \qquad (4)$$

where chi-square = 12.1 on 23 df. [In order for the model to be accepted, the residuals must be random according to the chi-square test. Due to the nature of time-series data, df is chosen differently than it is with ordinary data samples. A rule of thumb is that df is at least two seasons plus one or two observations less than one-third of the series, whichever is smallest (7). For all four models, the mean of residuals is not significantly different from zero.]

Experiments

The models given above were used for the following experiments, which were designed to investigate the

Table 1. Forecast results for I-90: model A.

Year	Month	Traffic Volume (no. of vehicles) Actual	Forecast	Forecast Error (%)
1978	January	56 892	52 754	7.27
	February	57 963	54 541	5.90
	March	60 324	56 800	5.84
	April	60 398	57 725	4.43
	May	60 494	57 425	5.07
	June	63 289	60 146	4.96
	July	61 555	57 348	6.83
	August	64 648	58 193	9.98
	September	60 731	56 198	7.46
	October	62 582	56 452	9.79
	November	58 664	55 253	5.81
	December	58 924	54 709	7.15
1979	January	56 434	51 535	8.68
	February	57 770	53 902	6.69
	March	59 645	56 161	5.84
	April	61 521	57 086	7.21
	May	59 636	56 786	4.78
	June	61 074	59 508	2.56
	July	57 342	56 709	1.10
	August	58 260	57 555	1.21
	September	56 141	55 560	1.03
	October	55 959	55 814	0.26
	November	53 924	54 614	-1.28
	December	52 519	54 070	-2.95
1980	January	47 511	50 897	-7.13
	February	52 122	53 264	-2.19
	March	54 478	55 523	-1.92

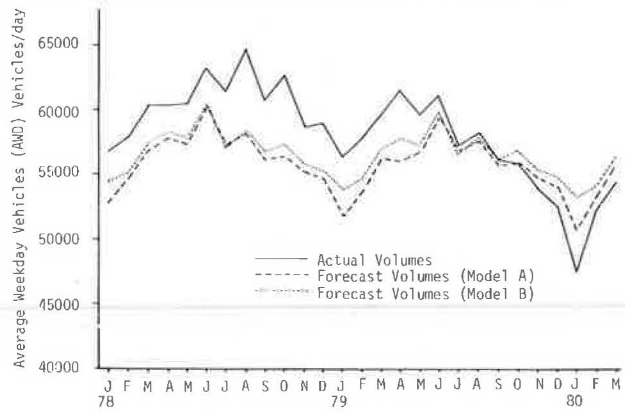
Table 2. Forecast results for I-90: model B.

Year	Month	Traffic Volume (no. of vehicles) Actual	Forecast	Forecast Error (%)
1978	January	56 892	54 454	4.28
	February	57 963	55 068	5.00
	March	60 324	57 456	4.75
	April	60 398	58 286	3.50
	May	60 494	57 814	4.43
	June	63 289	60 428	4.52
	July	61 555	57 151	7.16
	August	64 648	58 446	9.59
	September	60 731	56 633	6.75
	October	62 582	57 373	8.32
	November	58 664	55 987	4.56
	December	58 924	55 398	5.98
1979	January	56 434	53 760	4.74
	February	57 700	54 591	5.50
	March	59 645	56 914	4.58
	April	61 521	57 698	6.21
	May	59 636	57 266	3.97
	June	61 074	59 879	1.96
	July	57 342	56 589	1.31
	August	58 260	57 890	0.63
	September	56 141	56 079	0.11
	October	55 959	56 817	-1.53
	November	53 924	55 431	-2.79
	December	52 519	54 843	-4.42
1980	January	47 511	53 204	-11.98
	February	52 122	54 035	-3.67
	March	54 478	56 358	-3.45

I-90 pavement improvement and its impact on volumes for the two bridges and on model forecasts.

1. Experiment 1 was designed to compare the forecasts of models A and B with what actually happened during the months from January 1978 through March 1980. This is designed to determine (a) whether a 2.5-month, 24-h/day pavement improvement program in the summer of 1975 must be considered in forecasting a period three years later and (b) whether either model gives an acceptably accurate forecast.

Figure 2. Forecast volumes versus actual volumes for I-90: models A and B.



2. Experiment 2 was designed to compare the forecast of model C, which represents expected volumes if there were no pavement improvement, with the actual volumes. This is designed to determine what the impact of the pavement improvement on volumes was and how long this impact lasted.

3. Experiment 3 was designed to compare the coefficients of the binary policy variables in model B to the outcome of experiment 2 (these coefficients are another means of assessing the impact of the pavement improvements). The two results should be comparable, given adjustment for an expected error term.

4. Experiment 4 was designed to compare models B and D to assess the mutual effects of the I-90 improvement on both I-90 and WA-520.

RESULTS

The results of the forecasting experiment for models A and B for a 27-month time horizon are given in Tables 1 and 2. Model B performs slightly better than model A for most months. The average absolute percentage error for model A is 5.08 versus 4.66 for model B. Given the likelihood that traffic counters have an expected error within the range of ±5 percent, both models appear sufficiently accurate for forecasting purposes. Figure 2 further illustrates this comparison.

Thus, for purposes of attaining the required accuracy, it appears that a short-term policy intervention will not have a marked effect on the forecasting potential of the data base three years hence. However, one must examine the residuals of the first-pass model before determining the need for introducing binary policy variables. If the residuals for model A that correspond to the months of July, August, and September were significant, then model B would be chosen as the more accurate model. In fact, residuals for $t = 91$ and $t = 92$ that corresponded to the months of July and August 1975 were significant (the fact that the pavement improvement continued for only half the month of September may explain the fact that the residual for $t = 93$ was not identified as significant by the program). This, coupled with the fact that the estimated standard deviation of the residuals and the chi-square value for both models indicate a preference for model B, shows that the use of intervention variables increases the expected accuracy of the model

Thus, in time-series modeling it is important to take care of significant residuals. The process therefore becomes, in many instances, a two-stage estimation: First, a gross univariate estimation is made (no intervention variables); then the model is inspected for significant residuals (outliers) and reestimated by using dummy binary variables (intervention variables) corresponding to events that create explainable outliers. The coefficients of these dummy variables plus the type of forecast in experiment 2 give us information on the probable impact of the intervention in question.

The results of experiment 2 are given in Table 3. The comparison between actual and forecast volumes is also shown in Figure 3. We are assuming that model C represents the trends prior to the 1975 pavement improvement operation. Thus, the forecast volumes should represent what would have happened without the intervention plus an error term. Given the forecast errors in the previous example and in other time-series studies mentioned previously, a 5 percent error term appears to be reasonable expected error. Thus, the impacts on volumes indicated for July, August, and September 1975 are -27.16 ± 5 percent, -36.45 ± 5 percent, and -14.71 ± 5 percent, respectively. This represents the impact of the improvement on volumes during those months (note that the impact in the month of September is lower because the improvements were being made during the first half of September only). This impact dies off right after the project ceases. Judging from the residuals for 1976, the improvement may have a one-year lagged impact on volumes. However, since the residuals are not statistically significant, any lagged effect (i.e., reduced volumes due to memory of the previous year's experience) is probably minor.

Table 3. Forecast results for I-90: model C.

Year	Month	Traffic Volume (no. of vehicles) Actual	Forecast	Forecast Difference (%)
1975	July	45 717	58 135	-27.16
	August	43 709	59 641	-36.45
	September	49 765	57 088	-14.71
	October	54 774	56 519	-3.19
	November	54 144	56 005	-3.44
	December	53 610	55 794	-4.07
1976	January	52 845	53 122	-0.52
	February	53 713	54 967	-2.33
	March	54 347	57 355	-5.53
	April	55 117	58 069	-5.36
	May	55 885	58 094	-3.95
	June	57 532	61 320	-6.58
	July	56 538	59 762	5.70
	August	54 238	61 422	-13.25
	September	49 488	58 866	-18.95
	October	49 663	58 248	-17.29
	November	52 754	57 755	-9.48
	December	54 997	57 552	-4.65
1977	January	54 455	54 870	-0.76
	February	55 008	56 716	-3.10
	March	57 229	59 107	-3.28
	April	59 134	59 820	-1.16
	May	57 001	59 844	-4.99
	June	61 727	63 070	-2.18
	July	60 181	61 513	-2.21
	August	61 108	63 173	-3.38
	September	58 616	60 617	-3.41
	October	58 321	59 998	-2.88
	November	56 030	59 505	-6.20
	December	56 030	59 302	-5.84
1978	January	56 892	56 620	0.48
	February	57 963	58 466	-0.87
	March	60 324	60 857	-0.88
	April	60 398	61 570	-1.94
	May	60 494	61 594	-1.82
	June	63 289	64 821	-2.42

Comparison of models A and B provides another way of examining the impact of the I-90 pavement improvement on both I-90 and WA-520. The results of this comparison are given below:

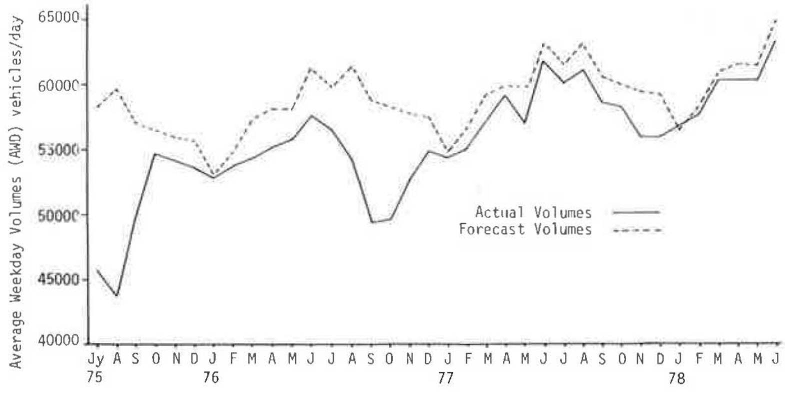
Bridge Affected	Month	Absolute Value	Change (%)
I-90	July	-12 366	-27.0
	August	-16 391	-37.5
	September	-6 041	-12.1
WA-520	July	+5 239	+8.9
	August	+5 035	+8.8
	September	+1 461	+2.8
Both	July	-7 217	-6.9
	August	-11 356	-11.3
	September	-4 580	-4.5

The coefficients of the policy variables for model A should be comparable to the results for July, August, and September 1975 obtained in the previous experiment for model C. If one takes an error term into account, the resulting reductions in I-90 volumes of 27.0, 37.5, and 12.8 percent correspond favorably to those obtained by using model C to forecast existing trends prior to 1975 and comparing with actual values. The results of a comparison of these impact measurements for I-90 made by using the two different approaches are summarized below:

Model	Month	Absolute Value	Change (%)
B	July	-12 366	-27.0
	August	-16 391	-37.5
	September	-6 041	-12.1
C	July	-12 418	-27.2
	August	-15 932	-36.5
	September	-7 323	-14.7

As the first table above further illustrates, the resurfacing of I-90 (a job that created severe traffic restrictions) had a twofold effect with regard

Figure 3. Forecast volumes versus actual volumes for I-90: model C.



to this study. First, the total traffic across Lake Washington decreased during the summer months of 1975, and these outliers (significant residuals) affected the model structure and the final estimate of the models. Second, although it was expected that traffic across Lake Washington would merely shift from I-90 to WA-520, the results show that there was a net decrease in across-the-lake traffic during this period. The traffic on I-90 decreased 20-30 percent, but WA-520 did not compensate for even half of this decrease in corresponding traffic increases. It must be concluded, therefore, that many of the trips crossing Lake Washington are a matter of convenience and are "noncaptive" trips.

SUMMARY

The experiments described in this paper indicate a promising potential for using the Box and Jenkins technique to assess the impact of short-term policy effects, such as pavement improvements on traffic volumes in the corridor in question and on parallel facilities. They also reemphasize the accuracy of short-term time-series forecasts (given the expected accuracy of existing traffic counters).

The experiments also indicate that models that introduce intervention variables corresponding to explainable outliers (significant residuals) are preferred over simple univariate models, although the impact of such outliers may be lessened over time. In other words, the impact of a short-run policy in 1975 may not significantly affect a forecast period three years hence [providing, of course, that all subsequent data (1975-1978) are included in the model specification].

The experiments also illustrated a high correlation between the two approaches to impact analysis. The first approach, in which a model was fitted to data prior to the intervention (model C) and then used to forecast what would have happened with no intervention, gave results that were very close to the results obtained with the multivariate model (model B). The reductions in volumes indicated by a comparison of actual and forecast volumes made by using model C were very close to the reductions indicated by the coefficients of the policy variables introduced in model B.

Finally, it appears that a pavement improvement policy such as the one analyzed here can have a significant and statistically measurable impact on traffic volumes for both the facility in question and the parallel facility. Such a policy also affects the absolute volumes of crossings regardless of route. The impact appears to die out almost immediately after the improvement is finished and to have no significant long-term effects.

REFERENCES

1. N.L. Nihan and K.O. Holmesland. Use of the Box and Jenkins Time-Series Technique in Traffic Engineering. Transportation, Vol. 9, 1980, pp. 125-143.
2. T.H. Naylor, T.G. Seaks, and D.W. Wichern. Box-Jenkins Methods: An Alternative to Econometric Models. International Statistical Review, Vol. 40, No. 2, 1972, pp. 123-137.
3. C.R. Nelson. Applied Time Series Analysis for Managerial Forecasting. Holden-Day, Inc., San Francisco, 1973, p. 7.
4. R.L. Cooper. The Predictive Performance of Quarterly Econometric Models of the United States. In Econometric Models of Cyclical Behavior, Studies of Income and Wealth (B.G. Hickman, ed.), Vol. 2, National Bureau of Economic Research, 1972, pp. 813-925.
5. G.V.L. Narasimnam, V.F. Castellino, and N.D. Singpurualla. On the Performance of the BEA Quarterly Economic Model and a Box-Jenkins Type of Arima Model. George Washington Univ., Washington, DC, 1974, pp. 1-24.
6. P. Newbold and C.W.J. Granger. Experience with Forecasting Univariate Time Series and the Combination of Forecasts. Journal of Royal Statistical Society, Series A, Vol. 137, 1974, pp. 131-146.
7. G.E.P. Box and G.M. Jenkins. Time Series Analysis: Forecasting and Control, rev. ed. Holden-Day, Inc., San Francisco, 1976.
8. G.E.P. Box and G.C. Tiao. Intervention Analysis with Applications to Economic and Environmental Problems. Journal of American Statistical Assn., Vol. 70, No. 349, 1975, pp. 70-79.
9. K.O. Holmesland. Use of Box and Jenkins Time Series Technique for Forecasting Commuter Traffic Volumes. Univ. of Washington, Seattle, Master's thesis, May 1978.
10. D.Z. Hammatuck. The Effects of a Service Interruption on Bus Ridership Levels in a Middle-Sized Community. Transportation Research, Vol. 9, 1975, pp. 43-53.
11. M.S. Atkins. A Case Study on the Use of Interventional Analysis Applied to Traffic Accidents. Journal of Operational Research, Vol. 30, No. 7, 1979, pp. 651-659.
12. M.N. Bhattacharyya and A.P. Layton. Effectiveness of Seat Belts Legislation on Queensland Road Toll: An Australian Case Study in Intervention Analysis. Journal of American Statistical Assn., Vol. 74, No. 367, 1979, pp. 596-603.
13. J.J. Wiorkowski and R. Heckard. The Use of Time Series Analysis and Intervention Analysis to Access the Effects of External Factors on Traffic Indices: A Case Study of the Effects of the Speed Limit Reduction and Energy Crisis in the State of Texas. Accident Analysis and Prevention, Vol. 9, No. 4, Dec. 1977, pp. 229-247.
14. M. Gaudry. An Aggregate Time-Series Analysis of Urban Transit Demand: The Montreal Case. Transportation Research, Vol. 9, 1975, pp. 249-258.

Evaluation of Network Traffic Performance Measures by Use of Computer Simulation Models

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The relation between traffic-signal-timing parameters and selected traffic performance measures of effectiveness (MOEs) was investigated by computer simulation of peak-hour flow conditions on an urban arterial in Madison, Wisconsin. The MOEs included delay, stops, fuel consumption, and exhaust emissions. A variety of signal-timing plans were generated by using time-space diagram methods and the TRANSYT signal-timing optimization model. Two computer simulation models, TRANSYT and NETSIM, were then used to develop traffic performance data for evaluation purposes. The results of the study showed that the signal-timing parameters that had the most significant effect on the MOEs were cycle length and the K-factor in the TRANSYT performance index. Speed of progression was highly correlated with number of stops: A higher value yielded a lower number of stops. Priority policy and split method did not show a significant impact on the MOEs. All MOEs can be improved when optimized timing plans are used instead of those developed by time-space diagram methods. In a comparison of the TRANSYT and NETSIM simulation models, the NETSIM model produced higher values for the MOEs under a given signal-timing plan. In a comparison of MOEs, number of stops and NO_X showed a close correlation whereas delay appeared to be a strong surrogate for the other principal MOEs.

Traffic behavior variables such as delay and stops have traditionally been used as indicators of the level of performance of a variety of traffic operations and control strategies. However, since the oil embargo of 1973, automobile fuel consumption has received increasing attention as an additional important performance measure. Recent research dealing with fuel consumption as a measure of effectiveness (MOE) has produced inconsistent findings with respect to its relation to certain traffic signal timing parameters, as well as other MOEs.

For example, in 1975 Bauer (1) developed a model of fuel consumption at signalized intersections based on Webster's equation for intersection delay. Testing of the model revealed that the cycle length at which fuel consumption is minimized apparently is significantly greater than the cycle length at which delay is minimized (see Figure 1). In a subsequent investigation, Courage and Parapar (2) found similar results for a network of 26 signalized intersections in Gainesville, Florida. By using estimates of delay and number of stops from the TRANSYT computer model (3) and applying Claffey's (4) fuel-consumption coefficients for a composite vehicle on level ground with an approach speed of 30 miles/h, Courage and Parapar found that minimum delay would be achieved at a 90-s cycle length and fuel consumption would be minimized at a 140-s cycle length (see Figure 2).

Dissimilar findings were reported in a 1979 study by Cohen and Euler (5), who used the NETSIM traffic flow simulation model (6) to evaluate the relation among fuel consumption, vehicle emissions, delay, and signal cycle length for an isolated intersection with a two-phase, fixed-time signal. Cohen and Euler found that the cycle length at which minimum delay occurs is the same as that at which minimum fuel consumption occurs (see Figure 3).

Figure 1. Delay and energy consumption versus cycle length for intersection with total critical flow of 1400 vehicles/h.

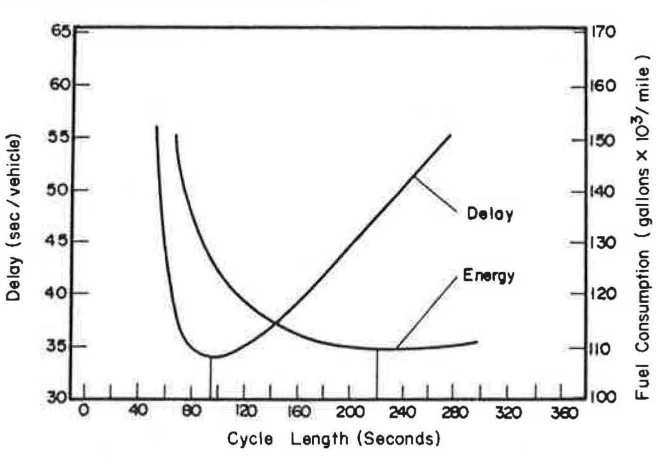


Figure 2. Fuel consumption and delay versus cycle length for signal system of Gainesville, Florida, central business district.

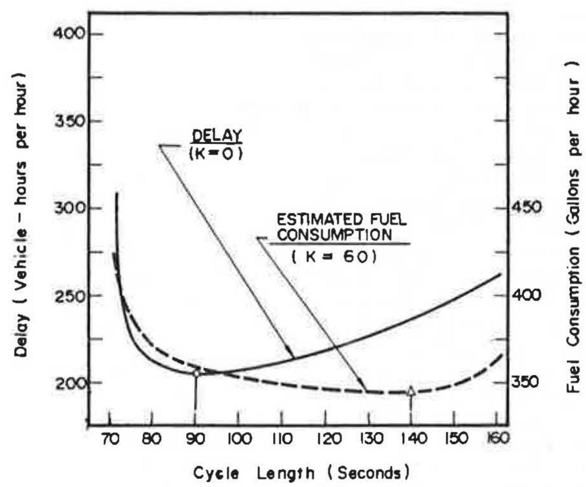


Figure 3. Fuel consumption and delay versus cycle length for isolated intersection with critical flows of 1800 vehicles/h (no left turn, 10 percent right turn) and 400 vehicles/h (no left turn, 20 percent right turn).

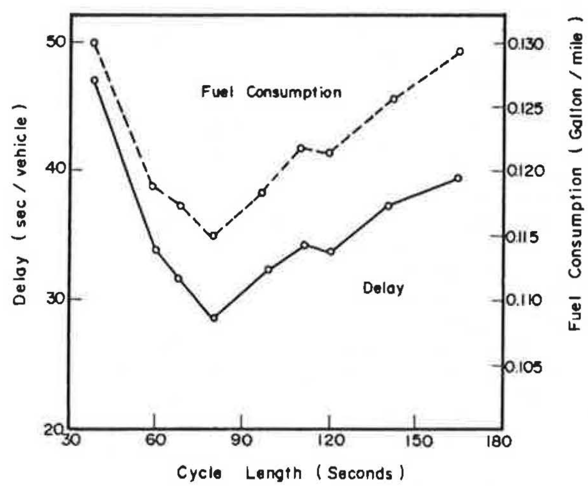
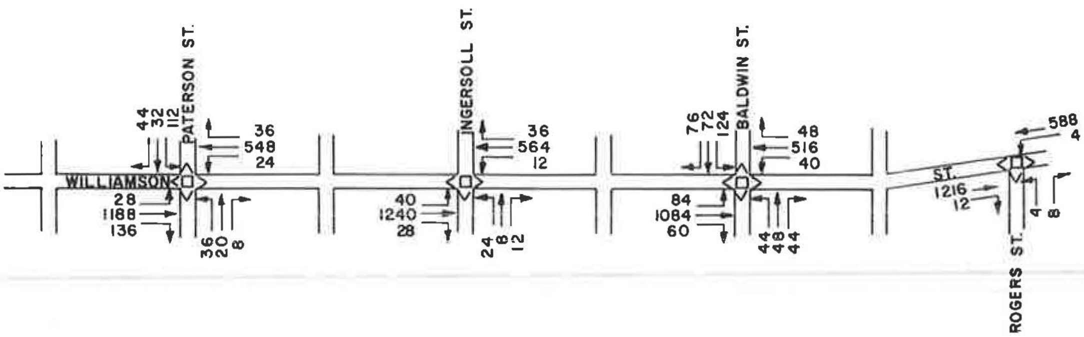


Figure 4. Peak-hour traffic flows for Williamson Street in Madison, Wisconsin.



The differences in the findings from these studies can probably be attributed to several factors. For example, both Bauer (1) and Courage and Parapar (2) applied Claffey's rate of idling fuel consumption (4) to the delay estimates produced by Webster's equation and the TRANSYT model, respectively. However, these delay values are estimates of overall delay, including that experienced during deceleration and acceleration. Therefore, the resulting estimates of fuel consumption attributable to the vehicle idling component would tend to exhibit an upward bias because of the inclusion of a certain amount of nonidling delay, especially when the proportion of stopped vehicles is high and average stopped delay is relatively small.

Another significant consideration is that the number of stops at an intersection is a more important factor in fuel consumption than is idling delay. For example, if one uses Claffey's composite fuel-consumption coefficients of 0.6 gal/vehicle-h of stopped delay and 0.01 gal/vehicle stop, a vehicle stop is equivalent to 1 min of idling delay in energy use, even though a vehicle stop without idling time causes less than 1 min of delay (for a 30-mile/h cruising speed, one stop-and-go cycle without idling delay causes about a 15-s delay).

In addition, the computer models that were used to estimate the number of stops do not yield directly comparable results. The TRANSYT model used in the Courage and Parapar study (2) can produce an overestimation of fuel consumption because any finite delay is assumed to cause a stop, even though in practice the vehicle involved may have undergone only a small deceleration and acceleration. On the other hand, the NETSIM simulation model accounts for the complete trajectory of each vehicle. However, because midblock delay and fuel consumption cannot be obtained separately from the NETSIM output, it is also difficult to isolate the intersection delay and fuel consumption that are affected by traffic-control signals. When the midblock delay on the links is small enough to be ignored, the NETSIM outputs of delay and fuel consumption can be considered to represent intersection traffic performance.

Another possible cause for the differing results of the previous studies lies in the queue-discharge logic of the models that were used. For example, the first vehicle in a queue accelerates directly up to cruising speed whereas cars farther back spend considerable time traveling at speeds lower than the cruising speed while moving up to the stop line. This type of movement generally consumes more fuel than traveling at the cruising speed. The microscopic queue-discharge behavior of the NETSIM model automatically includes this effect, whereas the TRANSYT model ignores it. The effect of multiple stops due to left-turning vehicles is also considered in NETSIM but not in TRANSYT. Consequently, in a strict sense, some of the output from these studies cannot be directly compared.

RESEARCH OBJECTIVES AND SCOPE

Given the above considerations, the primary objective of this research was to further evaluate the relations among delay, stops, fuel consumption, and vehicle emissions for various signal-control parameters at pretimed signalized intersections along an urban arterial under existing roadway and traffic-flow conditions (7). Although it would be preferable to examine a variety of urban arterial scenarios, cost and time constraints limited the research to a single case-study urban arterial.

The site selected for the study was a 5000-ft section of Williamson Street in Madison, Wisconsin (see Figure 4). Williamson Street is an arterial that has signalized intersections spaced 0.25 mile apart. Local street intersections occur between the signalized intersections, and traffic flow on the minor streets is relatively light compared with that on the arterial. Williamson Street is 50 ft in width and during peak hours operates with two traffic lanes in the peak flow direction and one traffic lane plus a parking lane in the opposite direction.

The experiments were designed to encompass a range of practical signal-timing plans developed by using both maximal-band-width time-space diagram methods and version 6C of the TRANSYT computer optimization program (8). Parameters such as directional priority, speed of progression, stop penalty, and split strategy were selectively varied. Both the TRANSYT and NETSIM computer models were used to simulate traffic performance under each of the timing plans. The resulting data were then subjected to statistical analysis by use of analysis of variance (ANOVA) techniques.

EXPERIMENTAL DESIGN

Research hypotheses were expressed in terms of the following seven questions:

1. Do the manually designed signal-timing plans and TRANSYT-optimized timing plans yield significantly different levels of performance?
2. Do the TRANSYT and NETSIM evaluation models yield significantly different results?
3. What is the effect of cycle length on each MOE?
4. Does the priority policy (peak-direction progression or balanced progression) make a significant difference?
5. What is the effect on MOEs of speed of progression in developing the manual timing plans?

Figure 5. Experimental design for manually developed timing plans.

Cycle Length (s)	Split	Priority Policy and Speed of Progression (miles/h)					
		Peak Direction Progression			Balanced Progression		
		20	25	30	20	25	30
60	Balanced V/C						
	Excess to arterial						
80	Balanced V/C						
	Excess to arterial						
100	Balanced V/C						
	Excess to arterial						
120	Balanced V/C						
	Excess to arterial						
140	Balanced V/C						
	Excess to arterial						

Figure 6. Experimental design for TRANSYT-optimized timing plans.

Cycle Length (s)	Priority Policy and K-Factor					
	Peak Direction Progression			Balanced Progression		
	K = 5	K = 60	K = 90	K = 5	K = 60	K = 90
60						
80						
100						
120						
140						

6. What is the effect of the TRANSYT stop weighting factor on MOEs?
7. Does the split strategy [balanced volume to capacity (V/C), or excess green to arterial with the minor street at level of service C] make a significant difference?

All of these questions were to be answered for each of six MOEs and two levels of aggregation (arterial and networkwide). The MOEs were delay; stops; fuel consumption; and hydrocarbon (HC), carbon monoxide (CO), and oxides of nitrogen (NO_x) emissions. All signal-timing-control parameters were independent variables.

Figure 5 shows the 60-cell experimental design matrix for the manually developed signal-timing plans. These timing plans were also used as the initial timings for the TRANSYT model. The TRANSYT model then generated optimal timing plans that minimized the following performance index:

$$PI = \sum_{\text{all links}} (\text{total delay}) + K \sum_{\text{all links}} (\text{number of stops}) \quad (1)$$

where PI is the performance index and K is a stop weighting factor.

Figure 6 illustrates the resulting 30-cell experimental design matrix for the TRANSYT timing plans. NETSIM evaluations were conducted by using one replication per cell, each replication being a simulation of a 15-min time period. Due to computer time limitations, the NETSIM evaluation of the manually developed timing plans was redesigned as a half-fractional factorial experiment.

The previously described hypotheses regarding the relations among the timing plans, the performance measures, and the evaluation models were tested by using ANOVA techniques. Performance data were aggregated at two levels: arterial links only and the overall network. The presence of interactions between variables and then the main effects of all factors were investigated. A multiple comparison of whether treatment (variable) means differed significantly was conducted after the ANOVA. These comparisons were made by testing the significance of particular linear combinations of the variable means. The procedure used was Duncan's multiple range test with a 0.01 significance level.

CALIBRATION OF TRANSYT AND NETSIM

Prior to the use of the TRANSYT6C and NETSIM computer models, a number of test runs were made for the existing evening peak-hour signal timing to calibrate those program-embedded parameters that showed significant differences from observed values. This procedure also provided a means of validating the results of selected experiments through actual field measurements. The model calibrations were primarily concerned with the following parameters: start-up delay, lag, stop estimate, saturation flow rate, free-flow speed, and amber phase response. The Wilcoxon matched-pairs signed-ranks test was used, and no significant difference was found between the TRANSYT, NETSIM, and field-observed values for the six selected traffic performance measures.

Because of the differences in the way each model defines a link connecting two intersections, as well as the way in which performance statistics are accumulated, special procedures had to be followed to ensure consistency in the comparisons of TRANSYT and NETSIM output. In TRANSYT, the delays actually incurred at the beginning of a receiving link and at the end of an approaching link are aggregated and assigned to the upstream link. This includes all through and left- and right-turning movements. For this reason, the TRANSYT network does not use exit links. On the other hand, in the NETSIM model, the link statistics are associated with the aggregate performance of all vehicles traveling from and to the respective stop lines that define the two ends of the link.

Since an internal link for both models encompasses some acceleration and deceleration delay, the accumulated statistics for the two models are not significantly different. For entry links, however, the difference between the two models is significant because the TRANSYT entry link includes both acceleration and deceleration delay at the stop line whereas the NETSIM entry link excludes the acceleration delay at that stop line. As a result, the flow statistics for TRANSYT entry links are usually much higher than those for NETSIM. Therefore, for the NETSIM and TRANSYT models to be consistent, the NETSIM network was coded with exit links that would account for the acceleration delays incurred in departing the stop lines of exit nodes.

A special adjustment to the TRANSYT output was also necessary in the calculation of the average speed on a link or for the overall network. For internal links, the average travel speed is obtained by dividing the distance traveled by the time spent on the link. Here, the time spent represents an actual travel time, including travel time for free-flow speed, and uniform and random delay. However, the program ignores the travel time on the entry link. Therefore, the time spent on the entry link is equivalent to the uniform plus random delays incurred in a queue. Consequently, average travel speed cannot be calculated from the data for distance traveled and time spent on the entry link. For the same reason, the networkwide average travel speed cannot be obtained by dividing the total distance traveled by the total time spent for the network as a whole. Therefore, for the purpose of comparing TRANSYT and NETSIM with respect to travel time and average speed, the TRANSYT output was adjusted by adding a reasonably estimated travel time to the uniform and random delay on each entry link. The additional time spent for each entry link in the TRANSYT output was calculated by multiplying the flow rate by its link length and then dividing by an average cruising speed observed in the field.

FINDINGS

The experimental design involved four basic experiments:

1. TRANSYT evaluation of manual timing plans,
2. TRANSYT evaluation of TRANSYT-optimized timing plans,
3. NETSIM evaluation of manual timing plans, and
4. NETSIM evaluation of TRANSYT-optimized timing plans.

Pairs of experiments were then coupled together to become a module for purposes of statistical and graphical comparison. The results of these comparisons are summarized below.

Traffic-Signal Parameters Versus MOEs

No interactions existed among the traffic-signal parameters. Each level of a traffic-signal parameter had a response curve that showed the same trend against each level of the other traffic-signal parameters. For example, total delay over each cycle length for K = 5 had the same trend as for K = 60 or 90, which showed that no interaction existed between K-factor and cycle length.

The principal results derived from the individual experimental analyses for the case-study site are summarized in Table 1 and discussed below:

1. Signal cycle length always had the most significant effect on each of the MOEs. Within the range of 80- to 140-s cycle lengths, delay, fuel consumption, and HC and CO emissions remained relatively constant. However, number of stops and NO_x emissions decreased as cycle length increased. The greatest inefficiencies for all MOEs occurred at the 60-s cycle length. This was due principally to the fact that, at cycle lengths less than or equal to 80 s, the minor streets received more green than necessary because of the minimum green interval constraint for accommodating pedestrians.
2. The stop weighting factor (K) used in the TRANSYT optimization model was the second most significant variable in terms of delay, stops, and fuel consumption. However, it had no significant effect on vehicle emissions. The number of stops and the amount of fuel consumption decreased as the K-value increased. However, delay in the overall network increased as the K-value increased, whereas delay on arterial links decreased. This is because of the trade-off between the delay to cross-street traffic and the delay to arterial traffic. The study results offered no evidence to support the reported hypothesis (2) that a K-value of 60 would provide a minimum fuel-consumption timing plan. Even though this study showed that a K-value of 90 yields the minimum fuel-consumption timing plan, this result is not sufficient to justify the conclusion that this would occur in every situation.
3. Speed of progression, as used in the manual signal-timing method, was the third most influential variable. It had a significant effect on total delay and number of stops. However, it also had some effect on fuel and vehicle emissions. The higher speeds of progression produced the lower MOE values.
4. Priority policy, as used in the manual signal-timing method, had a greater effect than the split method, but the significance of both variables was considered to be negligible. Usually the "peak-direction priority" and "excess green time to arterial" options would slightly reduce the delay to arterial traffic. The other MOEs were also reduced by using the peak-direction-priority option.

Manual Versus Optimized Timing Plans

In general, TRANSYT-optimized timing plans were found to improve all MOEs both on the arterial links

Table 1. Summary of interactions between signal-timing parameters and MOEs.

Timing Method	Parameter	Total Delay	Stops	Fuel Consumption	Emissions HC	CO	NO_x
Manual	Cycle length	⊕	⊕	⊕	⊕	⊕	⊕
	Speed of progression	+	⊕	+	+	+	+
	Priority policy	+	+	+	+	+	+
	Split method	+					
TRANSYT	Cycle length	⊕	⊕	⊕	⊕	⊕	⊕
	K-factor	⊕	⊕	⊕			
	Priority policy				+		

Note: + = main effect detected from TRANSYT output, and ⊕ = main effect detected from NETSIM output.

Table 2. Relative benefits of optimized timing plans: average percentage improvement at 120-s cycle length.

Area	Total Delay (%)	No. of Stops (%)	Fuel Consumption (%)	Emissions (%) HC	CO	NO_x
Arterial	22	25	5	6	12	2
Network	13	20	4	4	6	2

Table 3. Average percentage difference in MOEs from TRANSYT and NETSIM models at 120-s cycle length.

Area	Total Delay (%)	No. of Stops (%)	Fuel Consumption (%)	Emissions (%) HC	CO	NO_x
Arterial	38	18	17	-22	17	60
Network	17	14	14	-33	7	58

and the overall network, especially in the range of 80- to 140-s cycle lengths. The improvement of the optimized timings versus the manual timing plans increased as signal cycle length increased. The number of stops and total delay showed the most change; fuel consumption and vehicle emissions were less sensitive to the timing methods.

For example, Table 2 gives the improvements found for a 120-s cycle length. The percentage differences given in the table were computed from NETSIM simulation data and represent the relative improvement of the optimized timing plans with respect to the manually developed timing plans. The ability of the TRANSYT model to generate improved signal-timing plans compared with traditional methods has, of course, been noted and reported before.

TRANSYT Versus NETSIM

Except for the 60-s cycle lengths, the MOEs estimated by the two models were found to vary in a similar manner as cycle length ranged from 80 to 140 s and the NETSIM model was found to produce the larger MOE values. At the 60-s cycle length, almost all arterial links were oversaturated. Under these conditions, TRANSYT was found to generate very large delay estimates compared with NETSIM. The average difference in the MOEs estimated by the two models increased as signal cycle length increased within the range of 80-140 s. Average differences between the MOEs evaluated by the two models are given in Table 3 for a 120-s cycle length.

Except for HC emissions, all MOEs estimated by NETSIM have larger values than those produced by TRANSYT. The particularly large difference in delay for the arterial links may be caused in part by the difference in definition of the TRANSYT link and the NETSIM link. Because the MOEs estimated by the NETSIM model account for not only signal-related effects but also midblock interference, the NETSIM model should produce higher values for the MOEs. The models also differ in their estimates of fuel consumption and vehicle emissions. These differences are probably due in large part to the nature of the fuel-consumption and emissions subroutines within each model.

The difference between the TRANSYT and NETSIM models for the various MOEs might also be attributed to some extent to the error caused by having only one NETSIM run for each cell of the experimental design matrices. The NETSIM user's manual suggests at least two replications for each cell. A limited number of test runs conducted by using different random seeds for one of the cells showed a range in total network delay of approximately 7 percent and a slightly higher percentage for arterial delay. The range of total delay is almost the same size as the previously determined percentage change in the MOE due to the use of different K-factors but far less than that due to the model used.

Relations among MOEs

Possible relations among the various MOEs were of interest because this information would indicate whether one MOE could be used as a surrogate indicator for the others. Because cycle length was shown as the dominant signal-timing parameter, a comparison of each MOE over cycle length was made to reveal any correlation or similar response pattern among MOEs.

Figure 7 shows the relation over cycle length of various network MOEs as evaluated by the TRANSYT model. As shown in the figure, total delay, fuel consumption, and HC, CO, and total pollutants (combination of HC, CO, and NO_x) fall into a category that shows the same response pattern over cycle length. However, number of stops and NO_x emissions fall into another category that shows a steady decrease in the MOE as the cycle length increases. Figure 8 shows similar relations between cycle length and network MOEs evaluated by the NETSIM model.

Table 4 gives regression equations developed from the data in Figures 7 and 8. From the correlation coefficients for the NETSIM data, it is apparent that total delay is strongly associated with fuel consumption and HC, CO, and total emissions. The high correlation between total delay and fuel consumption implicitly supports the finding reported by Cohen and Euler (5) in their study of an isolated intersection: that delay and fuel consumption are minimized at approximately the same cycle length. Of the individual pollutants, only NO_x emissions were found to be well associated with number of stops. Fuel consumption and total delay were both reasonably well correlated with number of stops.

CONCLUSIONS

At the outset of the research, there were several fundamental questions to be resolved. The conclu-

Figure 7. Relation among network MOEs generated by TRANSYT.

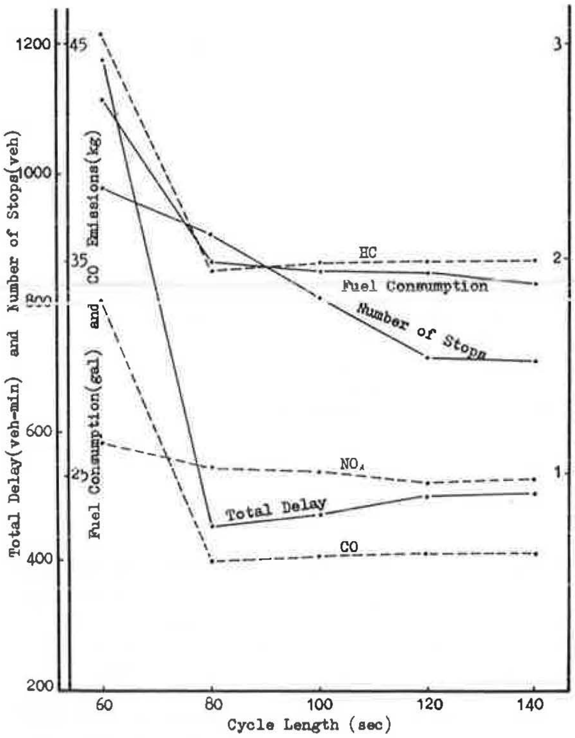


Figure 8. Relation among network MOEs generated by NETSIM.

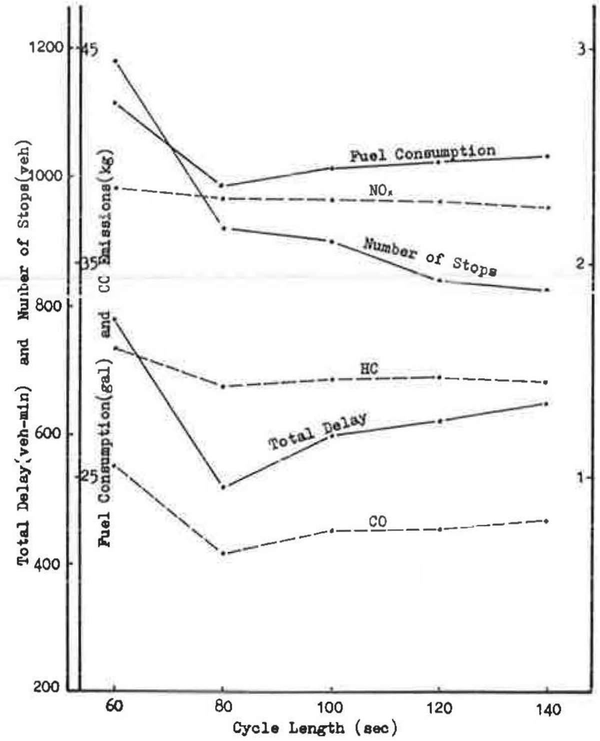


Table 4. Regression equations for relations among MOEs in network.

	TRANSYT		NETSIM	
MOEs	Equation	Correlation Coefficient	Equation	Correlation Coefficient
FC versus TD	FC = 28.9 + 0.012TD	0.973	FC = 30.8 + 0.015TD	0.958
HC versus TD	HC = 1212 + 1.6TD	0.998	HC = 1080 + 0.64TD	0.906
CO versus TD	CO = 12 530 + 18TD	0.997	CO = 13 945 + 14.4TD	0.976
TP versus TD	TP = 14 633 + 19.8TD	0.996	TP = 17 220 + 15.2TD	0.970
HC versus NS	HC = 1190 + 1.21NS	0.524	HC = 1183 + 0.33NS	0.766
NO_x versus NS	NO_x = 651 + 0.46NS	0.939	NO_x = 2090 + 0.22NS	0.743
TP versus NS	TP = 13 530 + 16NS	0.559	TP = 20 390 + 7NS	0.739
TD versus NS	TD = 31 + 0.7NS	0.488	TD = 308 + 0.35NS	0.584
FC versus NS	FC = 27.26 + 0.011NS	0.615	FC = 33.7 + 0.007NS	0.755

Note: FC = fuel consumption (gal), TD = total delay (vehicle-min), HC = hydrocarbon emissions (g), CO = carbon monoxide emissions (g), TP = total pollutant emissions (g), NS = number of stops (vehicles), and NO_x = nitrogen oxide emissions (g).

sions that can be drawn with respect to these questions are summarized below:

1. Among the various signal-timing parameters, cycle length and K-factor in the TRANSYT performance index are the most significant variables that affect the MOEs. However, the study failed to identify an optimal value of K that would produce a minimum level for each MOE. Speed of progression is highly correlated with number of stops: A higher value yields a lower number of stops. Priority policy and split method did not show a significant impact on any of the MOEs.
2. All MOEs can be improved, especially stops and delay on the arterial links, when optimized timing plans are used instead of those developed by use of traditional time-space diagram methods.
3. When the TRANSYT6C and NETSIM simulation models are compared, the NETSIM model produces similar, but larger, values for the MOEs under a given signal-timing plan. This is probably due simply to the differing simulation logic within the two models.
4. There appeared to be many correlations or similar response patterns among the various MOEs. Number of stops and NO_x emissions showed a close correlation, whereas delay appeared to be a strong surrogate for the other principal MOEs.

Because the study was performed for a single case-study site that had a unique set of traffic flows, it is unknown whether different traffic and roadway conditions would lead to significantly different results. In addition, the results of this study were limited in that it was only possible to conduct one replication for the cells of the experimental design matrices. This could create a large source of error or a loss in the power of the tests. Therefore, further research could be focused on multiple replications of other scenarios with different roadway and traffic conditions.

REFERENCES

1. C.S. Bauer. Some Energy Considerations in Traffic Signal Timing. Traffic Engineering, Feb. 1975, pp. 19-25.

2. K.G. Courage and S.M. Parapar. Delay and Fuel Consumption at Traffic Signals. Traffic Engineering, Nov. 1975, pp. 23-27.
3. D.I. Robertson and P. Gower. TRANSYT: A Traffic Network Study Tool. Road Research Laboratory, Crowthorne, Berkshire, England, Rept. 253, 1969.
4. P.J. Claffey. Running Costs of Motor Vehicles as Affected by Road Design and Traffic. NCHRP, Rept. 111, 1971.
5. S.L. Cohen and G. Euler. Signal Cycle Length and Fuel Consumption and Emissions. TRB, Transportation Research Record 677, 1979, pp. 41-48.
6. R.D. Worrall and E. Lieberman. Network Flow Simulation for Urban Traffic Control System, Phase II: Volume 4--User's Manual for UTCS-1 Network Simulation Model. Office of Research and Development, FHWA, Final Rept. FHWA-RD-73-86, May 1973.
7. C.-U. Do. An Evaluation of Traffic Flow Performance Measures in a Linear Arterial Network. Univ. of Wisconsin, Madison, Ph.D. thesis, 1980.
8. A.D. May. TRANSYT6C Model Workshop. Institute of Transportation Studies, Univ. of California, Berkeley, 1977.