Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/TX-04/0-4278-2		

4. Title and Subtitle	5. Report Date
LEFT-TURN AND IN-LANE RUMBLE STRIP TREATMENTS FOR RURAL INTERSECTIONS	September 2003
	6. Performing Organization Code
7. Author(s)	8. Performing Organization Report No.
Kay Fitzpatrick, Marcus A. Brewer, and Angelia H. Parham	Report 0-4278-2
9. Performing Organization Name and Address	10. Work Unit No. (TRAIS)
Texas Transportation Institute The Texas A&M University System College Station, Texas 77843-3135	11. Contract or Grant No. Project No. 0-4278
12. Sponsoring Agency Name and Address	13. Type of Report and Period Covered
Texas Department of Transportation Research and Technology Implementation Office P. O. Box 5080 Austin Texas 78763-5080	Research: September 2001-August 2003
	14. Sponsoring Agency Code

15. Supplementary Notes

Research performed in cooperation with the Texas Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.

Research Project Title: Safety Measures for Rural Intersections

16. Abstract

Studies were conducted on left-turn behavior, left-turn lane guidelines, and in-lane rumble strips. Behavior on the major road at a T-intersection is influenced by the width and type of the shoulder. When a wide level shoulder was provided, a large percentage of the drivers, up to 95 percent, drove on the shoulder at speeds near the operating speed of the roadway. At the site where the shoulder was retrofitted using available materials and widened from 3 ft (0.9 m) to 10 ft (3.1 m) just prior to the intersection, only 19 to 29 percent of the drivers used the shoulder. The Harmelink model is a widely accepted approach for determining whether to consider a left-turn lane and is the basis for the criteria included in the American Association of State Highway Transportation Officials (AASHTO) *Green Book* and the Texas Department of Transportation (TxDOT) *Roadway Design Manual*. Findings from current research suggest that Harmelink guidelines should be modified and these findings were used to revise the criteria. In order to gauge the effectiveness of in-lane rumble strips on driver speeds, rumble strips were installed on 14 approaches to rural intersections. An analysis of the speed data revealed a small and statistically significant decrease, generally 1 to 2 mph (1.6 to 3.2 km/h) in mean and 85th percentile speeds on the approaches. An additional objective for this project was to develop informational materials on rural intersection safety. The developed materials were incorporated as Chapter 6 in the TxDOT report *Treatments for Crashes on Rural Two-Lane Highways in Texas*, FHWA/TX-02/4048-2, April 2002.

17. Key Words	18. Distribution Statement
Rural Intersections, Left-Turn Behavior, Left-Turn Treatments, Rumble Strips	No restrictions. This document is available to the public through NTIS: National Technical Information Service 5285 Port Royal Road Springfield, Virginia 22161

19. Security Classif.(of this report)	20. Security Classif.(of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	168	

Form DOT F 1700.7 (8-72) **Reproduction of completed page authorized**

LEFT-TURN AND IN-LANE RUMBLE STRIP TREATMENTS FOR RURAL INTERSECTIONS

by

Kay Fitzpatrick, Ph.D., P.E.
Research Engineer
Texas Transportation Institute

Marcus A. Brewer
Associate Transportation Researcher
Texas Transportation Institute

and

Angelia H. Parham, P.E.
Assistant Research Engineer
Texas Transportation Institute

Report 0-4278-2
Project Number 0-4278
Research Project Title: Safety Measures for Rural Intersections

Sponsored by the
Texas Department of Transportation
In Cooperation with the
U.S. Department of Transportation
Federal Highway Administration

September 2003

TEXAS TRANSPORTATION INSTITUTE
The Texas A&M University System
College Station, Texas 77843-3135

DISCLAIMER

ACKNOWLEDGMENTS

The authors acknowledge the contributions of numerous persons who made the successful completion of this report possible.

Project Advisory Panel Members:

William Hale, P.E., Project Coordinator, TxDOT, Abilene District
Roy Wright, P.E., Project Director, TxDOT, Abilene District
Rick Collins, P.E., Project Advisor, TxDOT, Traffic Operations Division
Wade Odell, P.E., TxDOT, Research and Technology Implementation

Two technicians, Todd Hausman and Dan Walker, collected a substantial amount of data on this project and coordinated the use of the necessary equipment. There were also a number of students who collected and reduced data and assisted with report preparation. The authors wish to recognize the following individuals: Kerri Blaschke, Nathan Boone, Rob Buchanan, Cara Garcia, Melissa Ghrist, Carissa Mardiros, James McKenzie, Stephanie Sandt, Ty Thompson, Rose Mary Villagomez, Laura Wohlgemuth, and Tim Wolff.

The research reported herein was performed by the Texas Transportation Institute as part of a study entitled “Safety Measures for Rural Intersections” and was sponsored by the Texas Department of Transportation in cooperation with the U.S. Department of Transportation, Federal Highway Administration.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF FIGURES (Continued)

Figure Page

LIST OF FIGURES (Continued)

LIST OF TABLES

LIST OF TABLES (Continued)

CHAPTER 1

INTRODUCTION

BACKGROUND

Traffic conflicts due to turns at intersections and driveways are among the leading causes of crash problems associated with roadway design or traffic operations. In the United States in 2000, more than 2.8 million intersection-related crashes occurred, representing 44 percent of all reported crashes (*1*). About 8500 fatalities (23 percent of the total fatalities) and almost 1 million injury crashes occurred at or within an intersection (*1*). Of the fatal crashes at intersections, 47 percent involved left turns (or U-turns), 2 percent involved right turns, and 51 percent involved no turning maneuver. In Texas about half of the crashes in 2000 were associated with an intersection or driveway (see Table 1-1).

Table 1-1. Distribution of Crashes by Location (2000 TxDOT Data).

LOCATION	RURAL		URBAN		TOTAL	
	Frequency	**%**	**Frequency**	**%**	**Frequency**	**%**
Intersection	9085	17	31,592	26	40,677	23
Intersection Related	5970	11	23,429	20	29,399	17
Driveway Access	5183	9	10,062	8	15,245	9
Non Intersection	34,645	63	54,500	46	89,154	51
Total	**54,892**	**31**	**119,583**	**69**	**174,475**	**100**

CHARACTERISTICS OF RURAL INTERSECTIONS IN TEXAS

Rural roads represent four times the mileage of urban roads on the American highway system. The State of Texas maintains nearly 80,000 centerline-miles (128,000 km) of paved roadways, and over 62 percent of the centerline-miles are rural two-lane roads. Speeds on these roadways are often high, and crashes can be severe.

Department of Public Safety data for the year 2000 shows that 37 percent of rural crashes are intersection, intersection-related, or driveway-related, while over 54 percent of urban crashes are at those types of locations (see Table 1-1). As shown in Figure 1-1 and Table 1-2, the rural crashes at or near intersections or driveways may be further categorized as:

- 31 percent left-turn related,
- 25 percent angle related with no turns,
- 22 percent rear end,
- 12 percent straight (single vehicle),
- 8 percent right-turn related, and
- 2 percent other.

With the highest percentage of crashes at or near intersections being left-turn related, a better understanding of left-turn driver behavior is appropriate.

Table 1-2. Distribution of Near or At Intersection/Driveway Crashes by Movement (2000 TxDOT Data).

Movement	RURAL		URBAN	
	Frequency	%	Frequency	%
Left-Turn Related	6188	31	18,513	28
Right-Turn Related	1567	8	5168	8
Rear Ends	4467	22	19,470	30
Angle Related, No Turns	5108	25	17,386	27
Straight, Single Vehicle	2481	12	3031	5
Other	427	2	1515	2
Total	**20,238**	**100**	**65,083**	**100**

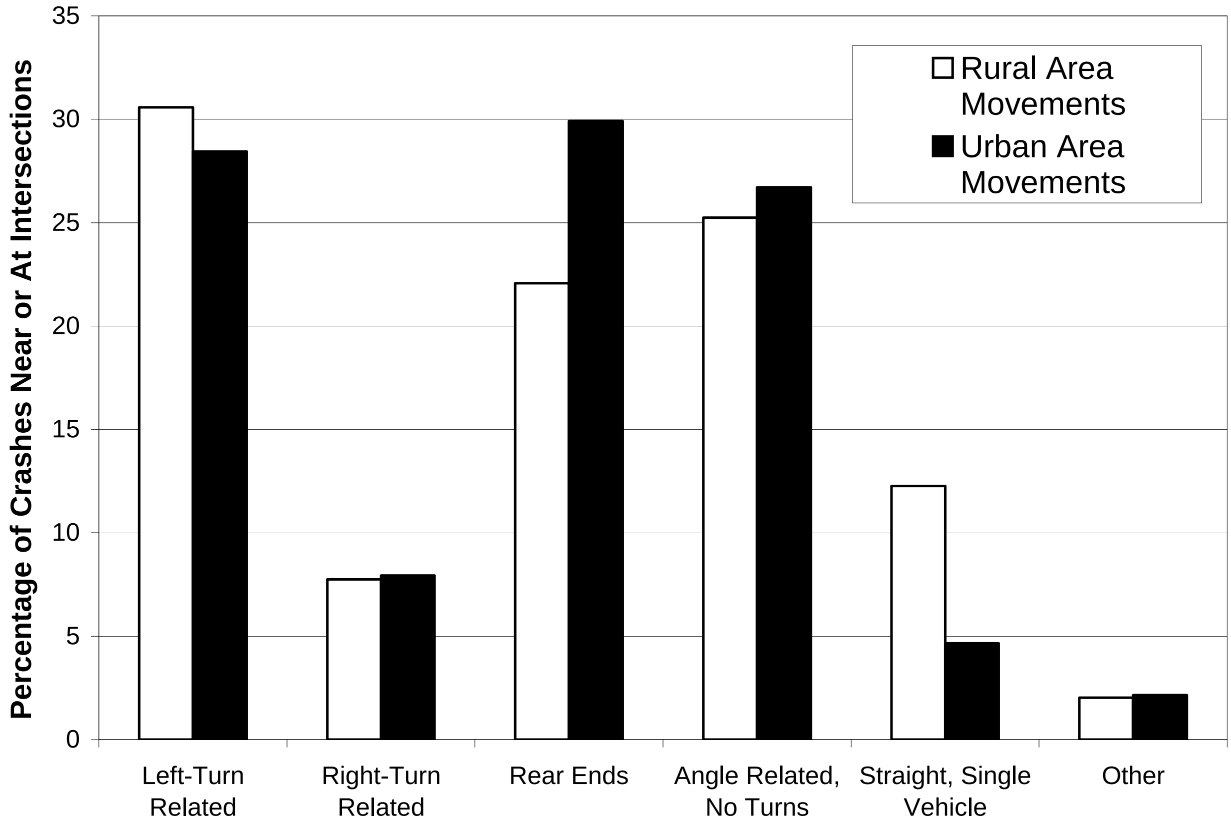


Figure 1-1. Percentage of Near or At Intersection/Driveway Crashes by Movement (2000 TxDOT Data).

The high percentage of straight, single-vehicle crashes in rural areas (12 percent) as compared to urban areas (5 percent) indicates that treatments warning drivers of a downstream intersection has greater need in rural than urban areas. Table 1-3 shows the distribution by intersection type.

Table 1-3. Distribution of Straight, Single-Vehicle Crashes by Intersection Type (2000 TxDOT Data).

Intersection Type	RURAL		URBAN		TOTAL	
	Frequency	%	Frequency	%	Frequency	%
Not Applicable	700	28	343	11	1043	19
3 Entering Roads T	1076	43	648	21	1724	31
3 Entering Roads Y	240	10	1031	34	1271	23
4 Entering Roads	462	19	1001	33	1463	27
5 Entering Roads	3	0	7	0	10	0
Traffic Circle	0	0	1	0	1	0
Total	**2481**	**100**	**3031**	**100**	**5512**	**100**

The distributions listed in Table 1-3 reveal that the most frequent straight, single-vehicle crashes in urban areas occur at Y-intersections (34 percent) with four entering roads being almost as great (33 percent). An interesting observation on the rural distribution is that almost half (43 percent) of the straight, single-vehicle crashes occur at T-intersections. Drivers are not recognizing the presence of the T-intersection and the need to stop before entering the crossroad. Figure 1-2 is an example of a collision diagram of a rural Texas intersection showing 3 years of crash data. The trends shown in the diagram support the observation that drivers are unaware of the T-intersection. Treatments suggested for this intersection included advance signing or markings, rumble strips on the approach, lighting at the intersection, oversized Stop signs, or flashing beacons.

A 1994 study *(2)* investigated variations in crashes as a function of geometric variables. The following summarizes their findings concerning variations in crash rates:

- An increase in average daily traffic (ADT) is the most significant factor in increasing the number of injury and fatality crashes at signalized intersections.
- Unsignalized intersections with higher posted speed limits (50 to 55 mph [81 to 89 km/h]) are prone to more crashes than comparable low speed intersections.
- The wider the pavement, the fewer the crashes.
- Shoulder width is not a significant factor in crashes on curves.

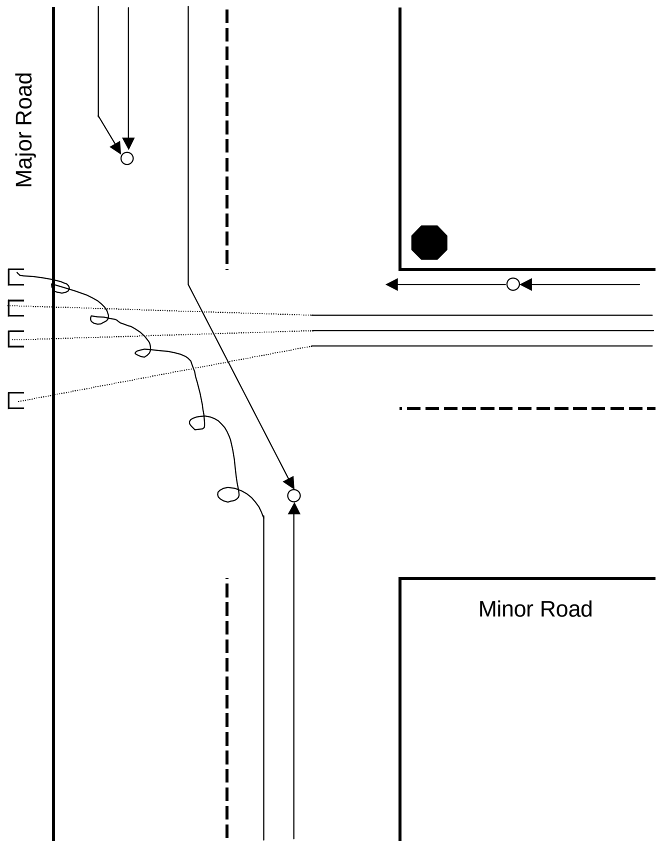


Figure 1-2. Collision Diagram of a Texas Intersection.

HUMAN FACTORS OF INTERSECTION SAFETY

Intersection safety is a product of the decisions that engineers make about the physical design and traffic control of each intersection (*1*). Drivers vary widely in their skills and willingness to take risks at intersections, and it is important to understand how drivers will react to road conditions and vehicle and pedestrian conflicts (*1*).

Driver Error

Crashes caused by drivers who fail to stop, or fail to yield the right-of-way to cross traffic after stopping, are becoming increasingly frequent at some rural intersections on the state highway system (*3*). The results of a number of studies suggest that crashes at two-way stop-controlled intersections are more closely related to driver error, such as failure to accurately judge the speed of major roadway vehicles, than to road geometry, sight distance, and driver compliance with traffic control devices (*3*).

Older Drivers

Older drivers (85 years of age and older) are more than 10 times as likely as drivers in the 40 to 49 age group to have multivehicle intersection crashes (*4*). More than one-half of fatal accidents for drivers 80 years and older occur at rural at-grade intersections, compared with 24 percent or less for drivers up to 50 years of age (*5*). According to Eck and Winn (*5*), the Federal Highway Administration's *Older Driver Highway Design Handbook* (*6*) states that the single greatest concern in accommodating older road users is the ability of these persons to safely maneuver through intersections.

Older drivers are usually much less inclined to take risks with narrow margins of error than are younger drivers (*1*). However, due to their diminished motor skills, poor vision, and reduced cognitive abilities, older drivers often make poor judgments at intersections that lead to a higher involvement in crashes. The motor skills that diminish in older drivers include backing, lane-keeping, maintaining speed, coming to a stop, and negotiating left turns (*5*). Older drivers also tend to suffer from performance-related problems involving speed (i.e., driving too slowly, misjudging speed, or excessive braking) and search patterns (i.e., inattention, inadequate scanning, failing to observe the rear, and pulling out without looking) (*7*). Driver situations involving complex speed-distance judgments under time constraints – the typical scenario for intersection operations – are more problematic for older drivers than for younger ones due to slower reaction times for any complex motor-cognitive task (*5*).

Younger Drivers

The youngest driver age groups (teenagers) have the highest traffic violation and crash involvement rates (*1*). This is often due to younger drivers' poor judgment, inexperience, and willingness to engage in risky behavior such as speeding, dangerous maneuvering, and violating red light signals and Stop signs.

PROJECT OBJECTIVES

The objectives of this project were to:

- identify and evaluate those measures that address safety at rural intersections and
- develop material on rural intersection safety.

During the project, researchers developed information regarding safety treatments at rural intersections. This information was incorporated as Chapter 6 in the TxDOT report *Treatments for Crashes on Rural Two-Lane Highways in Texas*, FHWA/TX-02/4048-2, April 2002 (*8*). The chapter was included as part of the report for TxDOT Project 4048 because it was closely related to the information developed regarding treatments for rural roadways in that project. Researchers developed the document to provide transportation practitioners with information on crash characteristics for rural roads in Texas and to help engineers identify problems at intersections and recommend potential countermeasures for installation. Report 4048-2 presents discussion on low-cost safety treatments used on highways and at intersections, along with their known effectiveness. The report also includes experiences with selected treatments in Texas, including whether the treatment would be considered elsewhere. The report will be produced in a three-ring binder to allow easy additions or changes as new or updated information is available on the effectiveness of crash treatments.

The information developed for the 4048-2 report and the responses from surveys regarding current practices were used to determine potential further evaluations as part of TxDOT Project 0-4278. Researchers determined that additional studies were needed for:

- transverse (or in-lane) rumble strips,
- left-turn driver behavior, and
- left-turn lane guidelines.

Current literature regarding in-lane rumble strips and left-turn lane treatment at rural intersections is summarized in Chapter 2. Research studies of left-turn driver behavior are discussed in Chapters 3 and 4, Chapter 5 presents the findings from a review of left-turn lane guidelines, a discussion of the simulation of a rural T-intersection is included in Chapter 6, and studies of rumble strips are discussed in Chapter 7. The report's final chapter presents the conclusions of the project.

CHAPTER 2

LITERATURE REVIEW

IN-LANE RUMBLE STRIPS

In-lane rumble strips are grooved or raised patterns on a roadway surface that provide an audible and tactile warning system to motorists approaching a decision point. They are well-suited for rural roadways that typically have low volumes, infrequent traffic control devices, and motorists who may not be as attentive to roadway conditions as urban drivers. In-lane or transverse rumble strips have been used for many years in work zones, intersections, and other areas that merit special measures for alerting drivers. Although rumble strips are not a speed-control device, they are generally thought to be effective in reducing speed and increasing stop compliance (*9*).

Figure 2-1 illustrates examples of a set of in-lane rumble strips and a close-up view of an in-lane rumble strip.

(a) Set of In-Lane Rumble Strips. **(b) Close-Up View.**

Figure 2-1. Examples of an In-Lane Rumble Strip.

Section 6F.78 of the 2000 *Manual on Uniform Traffic Control Devices (MUTCD)* (*10*) states:

> "Rumble strips consist of intermittent narrow, transverse areas of rough-textured or slightly raised or depressed road surfaces that alert drivers to unusual motor vehicle traffic conditions. Through noise and vibration they attract the driver's attention to such features as unexpected changes in alignment and to conditions requiring a stop."

Characteristics of Rumble Strips

A Minnesota Department of Transportation (DOT) synthesis states that in-lane rumble strips are installed in or on the driving lane, perpendicular to the traveling vehicle (*11*). They can cover the entire driving lane or can be set up to only run the width of the vehicle's wheel path. The rumble strips that are as wide as a wheelpath are designed to allow drivers familiar with the area to straddle the rumble strip in order to avoid driving over it.

A report by Zaidel et al. (*12*) describes four basic characteristics of rumble strips:

- They involve certain degrading of the roadway pavement surface smoothness.
- The basic treatment element is either a groove in the pavement about 0.5 inch (12 mm) deep by 4 inches (10 cm) wide or a tacked-on strip of rough pavement material 0.38 to 0.75 inch (10 to 20 mm) high and 4 inches (10 cm) to many meters wide.
- The basic elements are repeatedly placed as transverse strips across the roadway in a certain geometric pattern, starting some distance upstream and stopping some distance before the critical location. The treatment distance should correspond to the declaration distance of the 85th speed percentile, empirically observed before the treatment; about 32 to 49 ft (10 to 15 m) of pavement should be left clear before the stop line.
- The rumble treatment is assumed to provide drivers with visual, auditory, and tactile-vibratory stimulation, thus compelling them to be attentive to the demands of the situation.

The following sections provide further detail on specific characteristics and properties of rumble strips.

Materials

Raised rumble strips can be made from many materials (e.g., rubber, plastic, exposed aggregates, etc.), but asphalt strips are the most commonly used type of raised rumble strip (*13*). Removable rumble strips, most often used in advance of work zones, are typically made from rubber or plastic. Raised pavement markers (RPMs) have also been used to create the rumble effect (*13*).

Cross-Section

Cross-sections of rumble strips vary widely to include both raised and grooved and simple and complex geometries.

According to a 2003 Kansas report (*13*), rumble strips can be rectangular, trapezoidal, domed, or any other shape. Asphalt rumble strips of the appropriate shape and size are often created using wooden forms, and they usually have a domed cross-section. The width of the strips ranges from 2 to 12 inches (5 to 31 cm), though they are most often between 4 and 8 inches (10 and 20 cm). The height of the strips ranges from 0.125 to 1.5 inches (0.32 to 3.8 cm). Since grooved rumble strips require permanently altering the pavement by cutting or grinding in grooves, most temporary installations are raised strips. The Kansas DOT typically uses rectangular grooved rumble strips that are 0.375 inch (10 mm) deep and 4 inches (10 cm) wide for approaches to intersections.

Layout

A wide variety of rumble strip configurations have been used as well. These include a continuous stretch of textured pavement, single strips, and – what is currently the most widely-used configuration – intermittently spaced sets of strips (*9*).

A 1993 synthesis of practice on rumble strips provided typical values for in-lane rumble strips summarized from the design practices of 24 state highway agencies. Both raised and grooved rumble strips were represented in the summary. Table 2-1 summarizes the information provided in the 1993 synthesis on rumble strips used on an intersection approach. The table shows that rumble strip design practices vary widely. For example, the number of bars in a set varied from 4 to 25. Practices for spacing between rumble strip sets also varied widely (see Table 2-1).

Table 2-1. Dimensions and Design Criteria for In-Lane Rumble Strips (Data Summarized from 1993 Synthesis) (*14*).

State	R/G*	Length of Set (ft)	# of Strips in Set	Strip Length (in)	Height or Depth (in)	Strip Width	# of Strip Sets	Location, in order of increasing distance from the intersection (ft)
Alabama	R	6.67	5	18	0.625	Variable	5	250 to 600 (depends on approach speed), plus 50, plus 50, plus 100, plus 100
Arkansas	G	10.33	5	3.5 to 5	1.5	Full Lane	N.P.	N.P.
Colorado	G	11.33	12	4	0.500	Full Lane	5	300, 400, 500, 700, 1000 from intersection
Florida	R	5.17	6	2	0.500	Full Lane - 1.5 ft	7	100, 150, 200, 250, 350, 450, 650 from intersection
Georgia	R	20	N.P.	N.P.	N.P.	Full Lane	3	400, 585, 805 from intersection; NOTE: STOP AHEAD sign @ 735 from intersection
Hawaii	R (RPM)	24	9	N.P.	N.P	3 ft	N.P.	
Idaho	G, A pattern	11	8	6	N.P.	N.P.	6	400, 430, 470, 520 from intersection
	G, B pattern	17	12	6	N.P.	N.P.		650, 830 from intersection
Illinois	G	25	25	4	0.188	Full Lane	3	300, 500 from intersection; 200 upstream of STOP AHEAD sign; NOTE: Location of STOP AHEAD sign is variable
	R	25	19	8	0.188	Full Lane		
Iowa	G	24	25	4	0.375	Full Lane – 1.5 ft	3	300 from intersection, halfway between the two other locations; 200 upstream of STOP AHEAD sign; NOTE: Location of STOP AHEAD sign is variable
Kansas	G	24	25	4	0.375	Full Lane	3	1350, 1450, 1550 from intersection; NOTE: STOP AHEAD signs located @ 550 and 1250 from the intersection

Table 2-1. Dimensions and Design Criteria for In-Lane Rumble Strips (Data Summarized from 1993 Synthesis) (*14*) (continued).

State	R/G*	Length of Set (ft)	# of Strips in Set	Strip Length (in)	Height or Depth (in)	Strip Width	# of Strip Sets	Location, in order of increasing distance from the intersection (ft)
Kentucky	R	24.67	10	8	0.375 0.500	Full Lane	N.P.	N.P.
Michigan	G	3.33	4	4	0.375 - 0.500	Full Lane	3	400, 500, 700 upstream of the STOP AHEAD sign; NOTE: STOP AHEAD sign located from 400 to 750 from the intersection
Mississippi	R	8.5	9	4-8	0.500 1.000	Full Lane	5	200, 275, 375, 525, 725 from intersection
Nebraska	R	24.5	17	6	Max 0.75	2 @ 3.5 ft	2	1600, 1675 from intersection; NOTE: STOP AHEAD sign is located 1500 from the intersection
	G	24.33	19	4	Max 0.75	Full Lane – 1ft		
North Dakota	G (in AC pavt)	25.33	26	4	0.375	Full Lane	6	250, 305 from intersection; 70, 135, 235, 360 upstream of junction sign; NOTE: Location of junction sign is variable
		15.33	16	4	0.375	Full Lane		
	R (epoxy)	24.67	19	8	N.P.	Full Lane		
		15.33	12	8	N.P.	Full Lane		
Ohio	R	12.67	10	8	0.25	Full Lane	10	Rumble strip spacing vary as a function of approach speed
	G	11.33	12	4	0.500	Full Lane		
Oklahoma	G	2.85	5	8	0.875	Full Lane	3	500, 1000, 2000 from intersection; NOTE: STOP AHEAD sign located 900 upstream from the intersection
	G	2.85	5	8	0.875	Full Lane		
	G	4.33	5	12	0.500	Full Lane		
	R	20	15	8	0.50 - 0.75	Full Lane		
Oregon	N.P.	4.33	5	N.P.	N.P.	N.P.	5	Rumble strip spacing is variable from 150 to 500
South Dakota	R	24.5	17	6	0.50	2 @ 3.5 ft	2	300 to 600 from intersection (depends on approach speed); 150 upstream of STOP AHEAD sign; NOTE: Location of STOP AHEAD sign is variable
West Virginia	G	11.33	12	4	Max 0.75	Full Lane	10	Rumble strip spacings vary as a function of approach speed
Wisconsin	R or G	4.33	4	4	0.375	Full Lane	3	300, 425, 900 from intersection; NOTE: STOP AHEAD sign is located 700 upstream from the intersection
	G	23.25	24	3	0.500	Full Lane		

*R=raised, G=grooved, AC pavt = asphaltic concrete pavement
N.P. = information not provided
1 ft = 0.305 m, 1 in = 2.54 cm

Sound and Vibration

There are differing perspectives concerning the purpose of rumble strips. One is that rumble strips are intended to produce noise and vibration that will reduce the speed at which drivers feel comfortable traveling (*9*). The other viewpoint is that the noise and vibration caused by the treatment will simply alert the driver to upcoming roadway conditions that warrant special attention. In either case, the sound and vibration produced by the rumble strips must be severe enough to gain the driver's attention in order to have any positive effect on safety *(9)*.

An evaluation conducted by Walton and Meyer *(9)* examined the effect that changes made to rumble strip configurations have on the level of sound and vibration produced. These researchers measured in-vehicle sound and vehicle vibration for various rumble strip configurations. The measurements were then analyzed to study the relationships between various configuration parameters and the level of the sound and vibration produced. They found that removable rumble strips with a rectangular cross-section produced nearly the same sound and vibration as dome-shaped asphalt strips, which were more than twice the height. The only exception was that the asphalt strips were significantly louder than the removable strips in a truck. They concluded that a single thickness of 0.15 inch (4 mm) generated insufficient levels of sound and vibration. However, in most cases, using the double thickness of the removable strips generated levels of sound and vibrations that were comparable to, and sometimes greater than, the levels generated by the asphalt strips, and they concluded that removable strips could be a viable substitute for the asphalt strips. After testing several variables, the authors presented the results below:

- A 9-inch (23 cm) center-to-center spacing produced the greatest "rumble," and a 125-inch (320 cm) center-to-center spacing produced the greatest "jolt."
- Of the parameters isolated in the various comparisons, cross-section and spacing appeared to have the greatest effect on the levels of sound and vibration generated.
- The vibration experienced while driving over a set of rumble strips is not only affected by the configuration of the strips and the speed of the vehicle, but by the different properties of the vehicle as well (i.e., tires, wheelbase, suspension, etc.). Typical traffic mixes should be considered before adopting any particular configuration or strip type.
- The current 24-inch (61 cm) spacing standard is appropriate.
- Configurations with an offset generated lower levels of sound and vibration.
- When the length of the rumble area was increased by adding more strips, neither the sound nor vibration increased overall.

In a similar article, Meyer *(13)* found that the only parameter that appeared to have an identifiable effect on sound and vibration was the height of the rumble strips. The greater height produced greater sound, and the tests indicated that the Kansas DOT standard configuration produced the greatest overall sound and vibration level.

A study by Higgins and Barbel *(15)* had three objectives: (1) to measure the noise amplitude and frequency of rumble strips at various distances from the highway, (2) to measure the vibration inside a semi trailer tractor as it passes over the strips, and (3) to analyze how selected strip configuration affects both outside and inside vehicle noise and vibration when the tractor passes over it. This study concluded that rumble strips create a noise that is different from normal traffic noise and notes that the highway designer should be aware of strip placement near residential areas. The report also concluded that:

- Rumble strips that are formed rather than cut into the pavement create better driver perception.

- Outside noise (the type heard by adjacent property owners) does not significantly vary with the different types and configurations of rumble strips.
- A berm can significantly reduce high-frequency noise, but it is not effective in reducing low-frequency noise, such as that produced by a vehicle passing over a rumble strip.

State Practice

The FHWA website on rumble strips (*16*) revealed that policies or guidelines for in-lane rumble strips are not nearly as common as shoulder rumble strips or centerline rumble strips. Common applications for those states that use in-lane rumble strips include approaches to rural, high-speed intersections, tight horizontal curves, and toll facilities. Table 2-1 summarized the dimensions and design criteria for 24 states that participated in an early 1990 survey. Following is additional information available on in-lane rumble strips for selected states.

Texas

A 1969 article (*17*) states that Texas has used rumble strips since 1956. At that time, Texas used a ceramic bar or strip anchored to the roadway with an epoxy resin. Currently, a popular in-lane rumble strip in Texas is a private product that is purchased in strips and then glued to the pavement. Another type of in-lane rumble strip that has been used frequently in Texas is the application of transverse sections of seal-coat, sometimes called seal-coat strips.

New Jersey

According to Bellis, New Jersey had been experimenting with “singing lanes” since as early as 1949 to warn drivers that they were encroaching on an adjacent lane (*17*). Bellis’ 1969 article also notes that tests conducted by the New Jersey Department of Transportation (NJDOT) provided the following conclusions:

- The rumble strip would best serve its purpose:
  - in rural areas whose roads are intersected by state roads on which maximum speed limit is allowed and the traffic on either intersecting road is relatively light.
  - at approaches to traffic circles in rather unpopulated areas and, even though “signed,” not readily discernable to the not-too-alert driver.
  - in areas where an operating traffic light is obscured by the vertical contour of the highway or by a dog-leg or curve.
- A strip-to-critical-area distance of 800 ft (244 m) at the test location with a 55 mph (89 km/h) maximum speed limit was effective.
- From two sets of tests, researchers determined that a 10 ft, 5 inch (3.2 m) center-to-center spacing produced optimum jolting and a 9 ft (2.7 m) center-to-center spacing produced optimum vibration.
- A reduction in crashes was seen on the approach where rumble strips were installed.
- Consideration should also be given to:
  - whether a series of strip patterns, rather than only one, would better suit the purpose.
  - whether they should be installed on more than one of the roads that form the dangerous intersection.

- the possible necessity of developing ways and means for preventing the local motorist, familiar with the installation, from deliberately driving around it. This is dangerous to the motorist and may encourage other non-local motorists to follow the local driver in this behavior.
- establishing the proper distance between the warning device and the critical area. If the distance is too great, acceleration rather than deceleration can be influenced by the determined aggressive motorist; if too short, the alert motorist, who however, is exceeding the speed limit, is in trouble.

Florida

The Florida DOT developed a standard practice that is included in their specification drawings (*18*). Florida also specifies that when any portion of a curve falls within the limit of rumble strips, additional rumble strip sets spaced at 197 ft (60 m) shall be constructed throughout the remainder of the approaching curve.

California

The *CalTrans Traffic Manual* contains the following information regarding the use of both shoulder and in-lane rumble strips (*19*):

> The use of rumble strips on State highways requires approval by the District Traffic Engineer. Requests should include a description of location, reasons for use, the alternatives that were considered, collision history and a discussion of standard traffic control devices that have been or are in place.
>
> 1. TRAVELED WAY RUMBLE STRIPS
> Rumble strips on the traveled way are 0.07 in (19 mm) or less in height if raised or 0.10 in (25 mm) or less in depth if indented and generally extend across the travel lanes.
>
> Typical locations where rumble strips on the traveled way have been used include:
>
> - end of a freeway,
> - in advance of toll booths,
> - within a construction zone in advance of workers, and
> - in advance of a T-intersection where the motorist is not expecting to stop.

Ohio

An Ohio Best Practice/Policy (*20*) states that Ohio places thermoplastic rumble strips transversely across the travel lane(s) heading into a long-term work zone. These strips are 4 inches wide and 0.25 inches (6 mm) thick with the following spacing: two sections – 10 transverse strips, 6 ft (1.8 m) apart, then 90 ft (27.5 m) away the next section starts with 10 transverse strips, 4.5 ft (1.4 m) apart. The Ohio Department of Transportation (ODOT) District 12 (Cleveland area) has been using this practice for 1 year.

Minnesota

A 1968 article (*21*) reports that rumble strips were installed on the approaches to a number of stop-controlled intersections in southeastern Minnesota between 1962 and 1964. The ADT at these locations at the time of construction ranged from approximately 650 to 1400, except at one location where it was 2700. At the rumble strip installation sites, strips of coarse-aggregate seal coat were placed 50 to 100 ft (15.3 to 30.5 m) apart on the smoother pavement surface. The effects of these rumble strips are described by Owens's 1967 article summarized later in the section on rumble strip effectiveness.

According to a Minnesota DOT synthesis (*11*), 56 of the 68 Minnesota counties responding to a rumble strip survey use in-lane rumble strips. Most of these counties (48 of the 56) use two sets of rumble strips prior to the intersection or change in traffic control. Many of the counties indicated that they install rumble strips at all paved road intersections that have a stop condition. However, in-lane rumble strips are not included in the Minnesota *MUTCD* as an approved traffic control device.

Other States

A 1969 article (*17*) notes Maryland as being the second most active state for rumble strips with 238 installations of strips consisting of slag or stone laid on a bed of bitumen. Nebraska was reported to have 20 sets of bonded aggregates cemented to the road surface with epoxy and Illinois had 10 similar installations. North Carolina had one experimental installation with strips made from sand. Both Colorado and Indiana were performing tests at that time as well.

New Brunswick

A 1999 paper by Mason (*22*) reports that there are currently seven four-way stop-controlled locations in New Brunswick where the New Brunswick Department of Transportation (NBDOT) has installed rumble strips. They were installed laterally to the centerline of the stop approaches after determining that there was an intersection run-through problem at the stop approaches. Typically, five sets of grooves 0.5 inch (12 mm) deep, 0.59 inch (150 mm) wide, with a clear distance of 0.59 inch (150 mm), and with 10 in a set, are installed for each approach. The cost for each approach is approximately $5000, and the rumble strips can usually be installed in 1 to 2 days. The NBDOT has found that rumble strip maintenance, involving squaring up the edges with a portable milling machine, is required approximately every 2 years and represents about one-half of the capital cost.

Effectiveness of In-Lane Rumble Strips

Crash Reduction

According to the 1993 synthesis, 89 percent of state highway agencies have installed rumble strips at important locations such as intersection approaches, horizontal curves, and work zones (*14*). Unfortunately, not all of the installations prove to be 100 percent effective, and many of the reports were inconclusive. Harwood reports that safety evaluations in the literature generally show that the installation of rumble strips is effective in reducing crashes on intersection approaches (*14*). Table 2-2 summarizes the results of the studies referenced, all of which used a before and after study design. These studies show that the crash reduction effectiveness of in-lane rumble strips can range from 14 to 100 percent; however, the studies were generally small and varied greatly in quality and completeness. Only two of the studies in Table 2-2 found a statistically significant crash reduction from rumble strip installation. Harwood notes that despite the lack of rigor in the evaluation design, rumble strips can be effective and should be considered at locations where rear-end crashes and ran-Stop-sign crashes occur. These types of crashes involve an apparent lack of driver attention.

Table 2-2. Crash Reduction Effects for In-Lane Rumble Strips (Summarized from 1993 Synthesis) (*14*).

Study and Date	Location	Num of Sites	Safety Measure	Percent Change in Safety Measure	Statistically Significant?
Kermit & Hein, 1962	California	4	Total crashes	-59 to -100	Not Stated
Kermit, 1968	California	1	Ran-Stop-Sign Crashes	-50	Not Stated
Owens, 1967	Minnesota	2	Total Crashes	-50	No
Illinois, 1970	Illinois	5	Total Crashes Ran-Stop-Sign Crashes	+5 -50	Not Stated Not Stated
TRRL, 1977	U.K.	10	Total Crashes Related Crashes	-39 -50	No Yes
Virginia, 1981	Virginia	9	Total Crashes Fatal Crashes Injury Crashes PDO Crashes Total Crash Rate Related Crash Rate	-37 -93 -37 -25 -44 -89	Not Stated Not Stated Not Stated Not Stated Not Stated Not Stated
Carstens, 1982	Iowa	21	Primary Highway Approach Total Crash Rate Ran-Stop-Sign Crashes Secondary Highway Approach Total Crash Rate Ran-Stop-Sign Crashes	 -51 -38 -1 +3	 Yes No No No
TRRL = Transport and Road Research Laboratory PDO = Property Damage Only					

Table 2-2. Crash Reduction Effects for In-Lane Rumble Strips (Summarized from 1993 Synthesis) (*14*) (continued).

Study and Date	Location	Num of Sites	Safety Measure	Percent Change in Safety Measure	Statistically Significant?
Zaidel, Hakkert, & Barkan, 1986	Israel	1	Right-Angle Crashes	-50 to -67	No
Moore, 1987	Louisiana	24	Total Crashes Fatal and Injury Crashes Daytime Crashes Nighttime Crashes	-29 -14 -14 -50	Not Stated Not Stated Not Stated Not Stated
PennDot, 1991	Pennsylvania	8	Total Crashes Ran-Stop-Sign Crashes	-40 -59	Not Stated Not Stated
TRRL = Transport and Road Research Laboratory PDO = Property Damage Only					

A 1999 New Brunswick study (*22*) reported that the overall performance of grooved rumble strips at intersections has been very good. In the several years since the installation of rumble strips, run-through type crashes have been virtually eliminated. In 1999, the rumble strips were considered to be the most effective countermeasure to prevent run-through type incidents, and there were no plans to discontinue their use in the future. Police reports indicated that the installations have been very effective and that no complaints have been received about the noise that they generate.

A 1962 study (*23*) investigated the effectiveness of rumble strips installed at four different locations in Contra Costa County, California. The rumble strips consisted of a series of 25 ft (7.6 m) long areas of spaced overlays placed on the road surface at 50 to 100 ft (15.3 to 30.5 m) intervals using 0.75 inch (1.9 m) stones and seal coat techniques with three of the locations using a synthetic resin formulation to hold the stones in place and increase the durability of the strips. The researchers determined that crash rates were greatly reduced, Stop sign violations significantly reduced, vehicle speeds and deceleration rates before a sharp curve were reduced, and before-and-after motion pictures show marked changes in driver behavior. The number of crashes per year at each crash decreased by at least half, and crash severity decreased greatly as well, despite the fact that the total crashes on county roads in Contra Costa increased over the time period involved in the studies. Before the rumble strips were installed, most of the deceleration occurred immediately before the intersection; however, after they were installed, deceleration took place over a greater distance – beginning after the first three rumble strips were crossed – and was consequently much more gradual. The study also describes four general circumstances in which the strong stimulus of a rumble strip is required to prevent drivers from making serious mistakes:

- where there are other distractions competing for the driver’s attention (e.g., red neon advertising signs in the vicinity of the traffic signal);
- where the driver may become bored, fatigued, or drowsy from driving on long, monotonous stretches of rural roads;

- where the driver is preconditioned to easy, rural driving conditions then suddenly enters an urban community; and
- where the driver's previous experience may lead him or her to ignore information or warnings because he or she feels capable of judging the situation (e.g., where motorists disregard Stop signs at low-volume intersections with good sight distances).

Deceleration Behavior

The 1993 synthesis on rumble strips concluded that previous studies show that rumble strip installations on intersection approaches do result in a small reduction in vehicle speeds (*14*). Some vehicles are slowed more than others, however, and it appears that speed variance on the intersection approach may be increased. Harwood (*14*) reported on the following studies that investigated the effects of rumble strips on approach speeds:

- A 1962 study in California used rumble strips on one approach to a T-intersection where drivers must slow to turn but are not required to stop. Table 2-3 shows that rumble strips increased the deceleration rate used by drivers between 450 and 1000 ft (137 and 305 m) from the intersection and decreased the deceleration rate used by drivers within 450 ft (137 m) of the intersection.
- A 1967 Minnesota study examined the effects of rumble strips on vehicle speed for six intersection approaches. Speeds of free-flowing vehicles were measured at distances of 1500, 1000, 500, and 300 ft (458, 305, 153, and 92 m) from the stop. The study found that the presence of rumble strips reduced average speeds by 2 to 3 mph (3.2 to 4.8 km/h) at each of the observation points (see Table 2-4). While rumble strips decreased vehicle speeds, they increased the speed variance on the intersection approaches at all distances from the intersection greater than 300 ft (92 m).
- A 1977 study in the United Kingdom studied the effects of rumble areas on speeds at 10 sites where they were installed upstream of roundabouts, four-way intersections, T-intersections, horizontal curves, and small towns. The effects of the rumble areas on speed were not consistent. In some cases the presence of the rumble area appeared to cause drivers to choose a larger speed reduction between the 1312 ft and 164 ft (400 m and 50 m) points on the approach; however, at other sites the opposite appeared to be true.
- A 1992 study evaluated the effectiveness of rumble strips in reducing vehicle speeds on seven approaches to T-intersections in Ohio. The speeds were measured 300 ft (92 m) downstream of the first rumble strip pattern. On five of the seven approaches, there was a statistically significant reduction in the mean vehicle speed (see Table 2-5).

Table 2-3. Speeds on an Intersection Approach Where Drivers Are Required to Slow But Not Stop (*14*).

Location	Measurement	Before Rumble Strip Installation	After Rumble Strip Installation
A	85th percentile speed at 1000 ft (305 m) from the intersection and before the first rumble strip	44.0 mph (71 km/h)	46.0 mph (74 km/h)
	Average deceleration rate from Location A to B	0.57 ft/sec^2 (0.17 m/sec^2)	1.43 ft/sec^2 (0.44 m/sec^2)
B	85th percentile speed at 450 ft (137 m) from the intersection after three rumble strips have been crossed	41 mph (66 km/h)	37 mph (60 km/h)
	Average deceleration rate from Location B to C	3.46 ft/sec^2 (1.1 m/sec^2)	2.70 ft/sec^2 (0.8 m/sec^2)
C	85th percentile speed at intersection	14.8 mph (23.8 km/h)	15.1 mph (24.3 km/h)

Table 2-4. Mean Speeds at Specified Distances for Six Approaches to Stop-Controlled Intersections in Minnesota (*14*).

Distance From Intersection (ft)	Average Speed in mph (km/h)			Statistically significant before/after difference?
	Before Rumble Strip Installation	After Rumble Strip Installation	Difference	
300	31.01 (49.9)	27.99 (45.1)	3.02 (4.9)	Yes
500	36.57 (58.9)	33.59 (54.1)	2.98 (4.8)	Yes
1000	43.70 (70.4)	41.39 (66.6)	2.31 (3.7)	Yes
1500	47.26 (76.1)	44.47 (71.6)	2.79 (4.5)	Yes
Away from area	52.09 (83.9)	52.58 (84.7)	-0.49 (-0.8)	No

Table 2-5. Effect of Rumble Strips on Vehicle Speed on Seven Approaches to Stop-Controlled Intersections in Ohio (*14*).

Site	Mean Speed in mph (km/h) 300 ft (92 m) downstream of first rumble strip			Statistically Significant at 95% Confidence Level?
	Before	After	Reduction	
1	41.9 (67.5)	35.9 (57.8)	6.0 (9.7)	Yes
2	47.9 (77.1)	39.9 (64.2)	8.0 (12.9)	Yes
3	43.9 (70.7)	45.9 (73.9)	-2.0 (-3.2)	No
4	45.9 (73.9)	41.9 (67.5)	4.0 (6.4)	Yes
5	51.9 (83.6)	49.9 (80.3)	2.0 (3.2)	Yes
6	53.9 (86.8)	51.9 (83.6)	2.0 (3.2)	No
7	53.9 (86.8)	49.9 (80.3)	4.0 (6.4)	Yes

Zaidel et al. (*12*) evaluated the use of rumble strips on one stop-controlled rural intersection approach in Israel. Speeds were monitored for over 2500 lead vehicles at eight points along the 1377 ft (420 m) leading to the intersection. A total of 38 rumble strips were placed over a distance of 883 ft (269 m) upstream of the stop line. Figure 2-2 shows the 85th percentile speed before and after the rumble-strip treatment was installed. The researchers found that mean speeds were reduced by 5 to 43 percent while the speed variance increased (between –2 and 60 percent) with the installation of the strips. The researchers concluded that:

- Rumble strips lowered speeds by an average of 40 percent.
- Rumble strips had a small positive effect on compliance rate.
- With no pavement treatment, deceleration began at 492 ft (150 m) and peaked within the last 197 ft (60 m); with the rumble strips, most of the deceleration took place before the vehicle passed the first strip, followed by an additional deceleration within the last 197 ft (60 m).
- Deceleration became more uniform and moderate.
- Rumble strip effects remained stable after a year.
- A 492 ft (150 m) treatment of 0.5 inch (12 mm) strips is long enough to produce the positive effects of rumble strips.

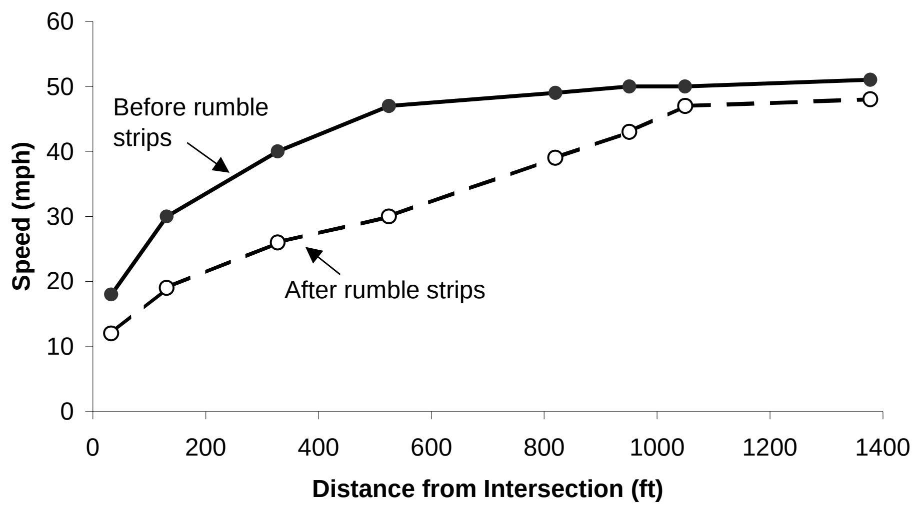


Figure 2-2. 85th Percentile Speed Before and After Rumble Strip Treatment (*4*).

A Minnesota simulation experiment (*24*) addressed the alerting effect of in-lane rumble strips on the stopping performance of alert drivers. The 32 participants used a wrap-around driving simulator on a simulated two-lane highway with a varied number of rumble strips (none, two, or three) on a stop-controlled approach. For the two-rumble strip scenario, the strips were placed at 359 and 722 ft (109 and 220 m) prior to the intersection with a Stop Ahead sign located at 487 ft (148 m). The study concluded that the presence of rumble strips had no effect on the point at which the drivers began to slow down (by removing their foot from the accelerator) or on the distance away from the intersection at which they actually stopped, but the rumble strips did affect the point at which they began to brake. The drivers in the study braked more and earlier when they were further away from the intersection when the rumble strips were installed than when they were not (see Figure 2-3). The study also compared driver reaction to full-width and wheelpath rumble strips and concluded that drivers brake more and earlier when full-width rumble strips were present than when wheelpath rumble strips were installed. The results seem to indicate that rumble strips cause drivers to use their brakes more and earlier and that they in turn allow safer, more controlled braking behavior at intersections. However, the downside may be that more early braking could be associated with increased rates of rear-end collisions from following cars that are not yet expecting to brake, and further investigation was recommended.

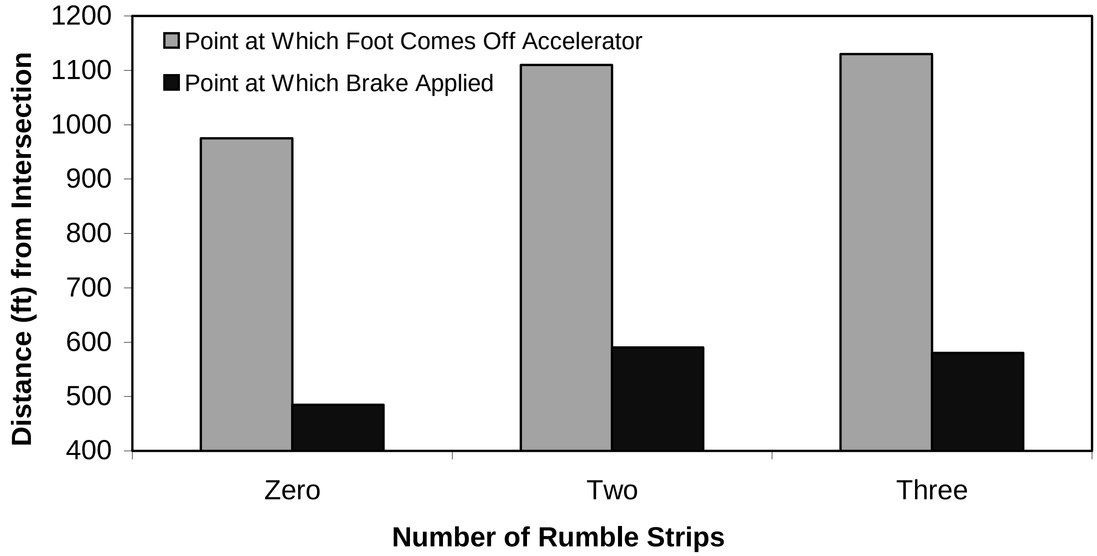


Figure 2-3. Effect of Rumble Strips on Point at Which Brake is First Applied (24).

In a Kansas study, Meyer *(13)* explored the effectiveness of orange removable strips at a bridge repair site in rural Kansas. Vehicle speeds were recorded with only the standard asphalt rumble strips in place; then the removable strips were installed and more speed data were collected. Installation and removal times were also compared. The rumble strips were easily applied and removed. Due to the difference in individual strip width, the total length of a group of removable rumble strips was 7 ft (2.1 m), compared with the 11 ft (3.4 m) of the asphalt rumble strips. The audible and tactile effects of the strips were weak due to their 0.125 inch (3.2 mm) thickness in comparison with the 0.5 to 0.75 inch (12.7 to 19 mm) thickness of standard asphalt rumble strips. However, the orange removable rumble strips were found to have a significant effect on vehicle speeds, attributable to their high visibility. Kansas Department of Transportation (KDOT) plans to conduct another evaluation using a version of the rumble strip that is 20 percent thicker. Another suggested technique is double height (produced by placing one strip on top of another). Meyer believed that more strips would have improved the effectiveness, and that the 1-ft (0.3 m) spacing was not the optimal selection. However, it was clear that, overall, there were significant advantages of the visible warning provided by the orange strips.

Vehicle Compliance

The 1993 synthesis on rumble strips identified five studies that evaluated the effect of rumble strips on driver compliance with Stop signs *(14)*. The studies generally found that it was rare for motorists to proceed through a Stop sign without stopping at all. However, installation of rumble strips on an intersection approach generally increased the proportion of drivers who made a full stop. Specific findings included the following:

- A California study on stop-controlled intersections saw an increase in the percentage of drivers making a full stop from 46 to 76 percent. Drivers making either a full or partial stop increased from 96 to 100 percent.

- A Minnesota study found an increase from 37 to 63 percent in the number of full stops.
- An Illinois study found that the number of vehicles stopping or partially stopping was 95 percent for the five intersections with rumble strips while only 91 percent at four comparable locations without rumble strips.
- An Iowa study compared the performance at one site with rumble strips and one site without. About 77 percent of the vehicles that did not encounter a conflict stopped or nearly stopped at the intersection with rumble strips while 66 percent stopped at the intersection where there were no rumble strips.
- An Israeli study found that installation of rumble strips on an intersection approach increased Stop sign compliance from 91 to 95 percent.

Other Considerations

Along with deciding what configuration to use and where to locate the rumble strip treatments, there are several other factors which should be taken into account as well.

Advance Notice

The Bellis article (*17*) suggests that serious consideration should be given to the question "Does the motorist require advance notice of the oncoming rumble strip? And if so, how should it be given?" At first this might seem to be a ridiculous question, since the purpose of the rumble strip is to warn and alert the motorist. However, quite a few alert drivers, given no advance notice of the experience and apparently not acquainted with such experience, pull off to the side and examine their car for mechanical trouble (*17*). This type of reaction slows down traffic and can be hazardous to the motorist.

Driver Reactions

The Walton and Meyer article (*9*) acknowledges that, for whatever reason, some drivers will drive on the shoulder or, worse, cross the centerline to avoid driving over the strips. Presumably, the more severe the noise and vibration, the greater the percentage will be of drivers likely to make such erratic maneuvers. Care should be taken to design the treatments so that they neither cause a loss of control over the vehicle (especially of concern for motorcycles), nor be so excessive as to cause physical damage or great discomfort to the driver or neighboring residents (*9*).

Overuse

A 1999 New Brunswick DOT paper (*22*) recommends that, as with other traffic control devices, care must be taken not to overuse rumble strips. In the paper, Mason states that "the effect of gaining the motorist's attention is because passing over the rumble strips is a relatively new experience. If motorists were to encounter rumble strips too frequently, they would lose their effectiveness where they are truly needed."

Disadvantages

The *CalTrans Traffic Manual* (*19*) notes several other significant disadvantages to the use of rumble strips across the travel lanes. These include:

- An abrupt rise or depression in the roadway can present problems to bicyclists and motorcyclists. For this reason, there should be provisions made for cyclists to safely transverse through or around rumble strips.
- Nearby residents may be subjected to continuous noise and vibration in residential areas, prompting citizens' complaints.
- All motorists are subjected to the noise and vibration, whereas only a few are in need of this effect to be alerted.
- Motorists may make unusual maneuvers to avoid rumble strips.

LEFT-TURN LANES

According to Iowa State University researcher Richard Storm, "various studies have shown that rear-end collisions account for 18 to 23 percent of all crashes," and 10 percent of those rear-end crashes involved a vehicle making or preparing to make a turning movement (*25*). A large number of crashes involving turning movements can be attributed to violations of driver expectancy by the actions of other drivers. When one driver slows or stops before making a turning maneuver, the following driver may be unprepared for the vehicle in its path, and a rear-end collision may result, especially if the location involves limited sight distances (*25*). The installation of left- and right-turn lanes may be an effective solution to counter the high crash rates involving vehicle-turning movements.

A recent FHWA study developed algorithms to predict the expected safety performance of rural two-lane highways (*26*). The predicted algorithms combined elements of historical crash data, predictions from statistical models, results of before-after studies, and expert judgments made by experienced engineers. As part of the research, an expert panel of safety researchers developed crash modification factors (AMFs) for specific geometric design and traffic control features. The panel developing the AMF for left-turn lanes made two conclusions:

- There has been no well-designed before-after study of intersection left-turn lanes and no single study was considered more reliable than others. (Since this conclusion, FHWA sponsored a major study of intersection turn lanes. The findings from that study are presented below.)
- The panel combined results from several studies and developed AMFs for left-turn lanes. The panel estimated that installation of a left-turn lane along one major approach reduces intersection-related crashes by 18 to 24 percent, depending upon the type of traffic control and the number of legs, and installation of left-turn lanes along both major approaches to a four-leg intersection reduces intersection-related crashes by 33 to 42 percent, depending upon the type of traffic control.

A FHWA study (*27*) on at-grade left- and right-turn lanes evaluated 280 three- and four-leg intersections at which projects that added either left-turn lanes, right-turn lanes, both left- and right-turn lanes, or an extension to the length of an existing turn lane at an intersection were implemented. The following results indicate when a left-turn installation would become cost-effective:

- **Rural three-leg unsignalized intersections –** for a major-road ADT of 4000 vehicles/day with 10 percent of the major-road volume on the minor road and at 2000 vehicles/day with 50 percent of the major-road volume on the minor road.
- **Rural four-leg unsignalized intersections –** for a major-road ADT of 3000 vehicles/day with 10 percent of the major-road volume on the minor road and at 1000 vehicles/day with 50 percent of the major-road volume on the minor road.

Table 2-6 presents the percentages of expected reduction in total crashes due to the installation of left-turn lanes on the major-road approaches to intersections.

Table 2-6. Expected Effectiveness of Left-Turn Lanes on Crash Reduction (*27*).

Intersection Type	Intersection Traffic Control	Number of Major-Road Approaches on Which Left-Turn Lanes are Installed (%)	
		One Approach	**Both Approaches**
RURAL			
Three-leg intersection	Stop Sign Traffic Signal	44 15	
Four-leg intersection	Stop Sign Traffic Signal	28 18	48 33
URBAN			
Three-leg intersection	Stop Sign Traffic Signal	33 7	
Four-leg intersection	Stop Sign Traffic Signal	27 10	47 19

CHAPTER 3

LEFT-TURN DRIVER BEHAVIOR ON TWO-LANE RURAL HIGHWAYS

To obtain a better understanding of left-turn driver behavior in Texas, researchers collected data at several intersections. The types of intersections studied were rural T-intersections where a minor arterial intersected with a major arterial with the minor arterial being controlled by a Stop sign. At several of the intersections, a significant number of vehicles that were impeded by left-turning vehicles used the shoulder of the arterial as a bypass lane. When minimal or no shoulders were present, queues began to form behind the stopped left-turning vehicles.

This effort used the following sites as study locations:

- Site 1: FM 60 and Copperfield Drive (Bryan)
- Site 2: SH 30 and Associates Avenue (College Station)
- Site 3: RM 150 and High Meadows (San Marcos)
- Site 4: RM 150 and Wheatfield Way (San Marcos)
- Site 5: RM 150 and several entrances to Elementary School (San Marcos)
- Site 6: SH 21 and FM 2001 (Niederwald)

DATA COLLECTION METHODOLOGY

At the sites, data were collected by two methods: laser and video. The lidar gun was used to collect spot speeds on vehicles traveling through the study site. At some sites, sight distance constraints required that two lidar guns be used to collect speed data on vehicles that traveled through the study site. The video data were collected using a camera mounted on a pole. The video provided a visual record of all traffic at the study site.

To collect the laser data with the lidar gun, the observers would position themselves upstream or downstream from the intersection. As vehicles traveled through the study site, speed data were collected and downloaded directly to a laptop computer, to record speed and distance measurements.

Video data were collected through the use of a trailer with a video camera mounted on an elevated telescoping arm, connected to a videocassette recorder. The trailer was generally located downstream from the intersection. Observers periodically monitored the video trailer to confirm proper operation and to replace used videotapes. Table 3-1 summarizes the data collection periods.

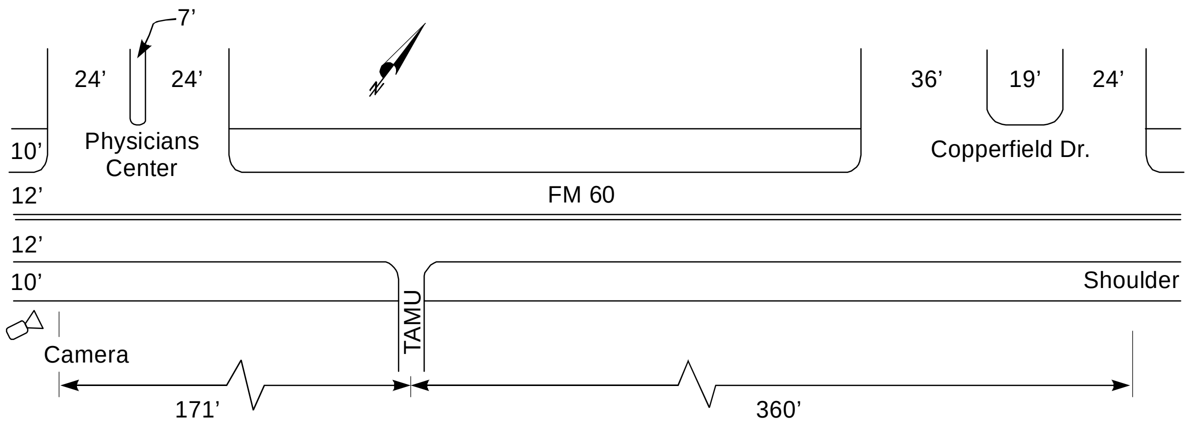
Table 3-1. Data Collection Periods for Study Sites.

Site	Video		Laser	
	Date	Time	Date	Time
1	Monday, March 18, 2002 Tuesday, March 19, 2002 Wednesday, March 20, 2002	6:00 am – 7:00 pm 6:00 am – 6:00 pm 6:00 am – 7:00 pm	Monday, March 18, 2002 Tuesday, March 19, 2002 Wednesday, March 20, 2002	7:30 am – 6:30 pm 7:00 am – 6:00 pm N/A
2	Thursday, November 14, 2002	8:00 am – 6:00 pm	Thursday, November 14, 2002	2:30 pm – 5:45 pm
3	Wednesday, October 2, 2002	11:30 am – 7:15 pm 6:30 am – 11:20 am	Wednesday, October 2, 2002	2:00 pm – 4:00 pm 6:50 am – 8:30 am
4	Thursday, October 3, 2002 Friday, October 4, 2002	11:40 am – 6:55 pm 7:55 am – 11:55 am	Thursday, October 3, 2002 Friday, October 4, 2002	3:45 pm – 5:55 pm 6:50 am – 8:30 am
5	Monday, September 30, 2002 Tuesday, October 1, 2002 Wednesday, October 2, 2002	11:00 am – 7:00 pm 6:30 am – 11:30 am 6:40 am – 11:10 am	Monday, September 30, 2002 Tuesday, October 1, 2002 Wednesday, October 2, 2002	1:45 pm – 3:45 pm 1:45 pm – 3:30 pm 6:55 am – 8:30 am
6	Wednesday, December 18, 2002 Thursday, December 19, 2002	1:00 pm – 12:00 am 12:00 am – 1:00 pm	Wednesday, December 18, 2002 Thursday, December 19, 2002	3:00 pm – 4:40 pm 4:45 pm – 5:45 pm 8:10 am – 8:35 am 8:45 am – 9:45 am

STUDY LOCATIONS

Following are descriptions of the study locations. Each description contains an intersection detail in plan view form.

Study site 1 is the intersection of FM 60 (University Drive East) and Copperfield Drive in Bryan, Texas. West of the intersection is a small driveway to a Texas A&M University (TAMU) facility and a large driveway to a Physicians Center. This site is at approximately level grade. The posted speed limit on FM 60 at the time of data collection was 65 mph (105 km/h). Figure 3-1 illustrates the geometry of the data collection area, and Figure 3-2 shows a photograph of the site.

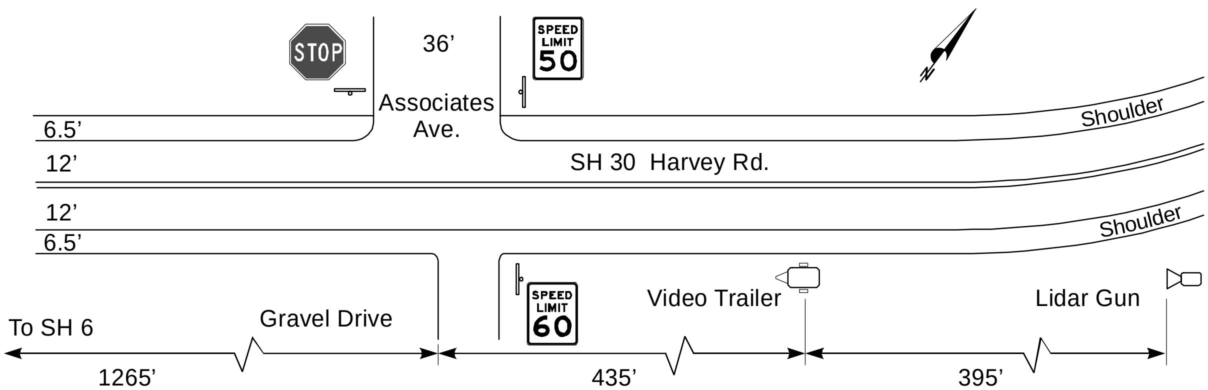


(1 ft = 0.305 m)

Figure 3-1. Intersection Detail at Site 1.

Figure 3-2. Site 1 (Copperfield Drive).

Site 2 is the intersection of SH 30 (Harvey Road) and Associates Avenue in College Station, Texas. This intersection is located approximately 0.25 mi (0.4 km) northeast of a freeway and is at approximately level grade. The posted speed limit on SH 30 was 50 mph (81 km/h) on the approach to Associates Avenue with a speed limit of 60 mph (97 km/h) posted 50 ft (15 m) beyond the intersection. Figure 3-3 illustrates the geometry of the data collection area, and Figure 3-4 shows photographs of the site.



(1 ft = 0.305 m)

Figure 3-3. Intersection Detail at Site 2.

(a) Just Downstream of Intersection **(B) Close-Up of Shoulder**

Figure 3-4. Site 2 (Associates Avenue).

Sites 3, 4, and 5 are three intersections on Ranch to Market Road (RM) 150. Figures 3-5 through 3-7 are photographs of the sites, respectively, with Figure 3-8 showing a schematic of the sites. They are located approximately 7 mi (11.3 km) northeast of San Marcos, Texas, and 3.5 mi (5.6 km) southeast of Kyle, Texas. Each intersection is at level grade. RM 150 is a two-lane rural road with 11 ft (3.4 m) wide lanes and no shoulders.

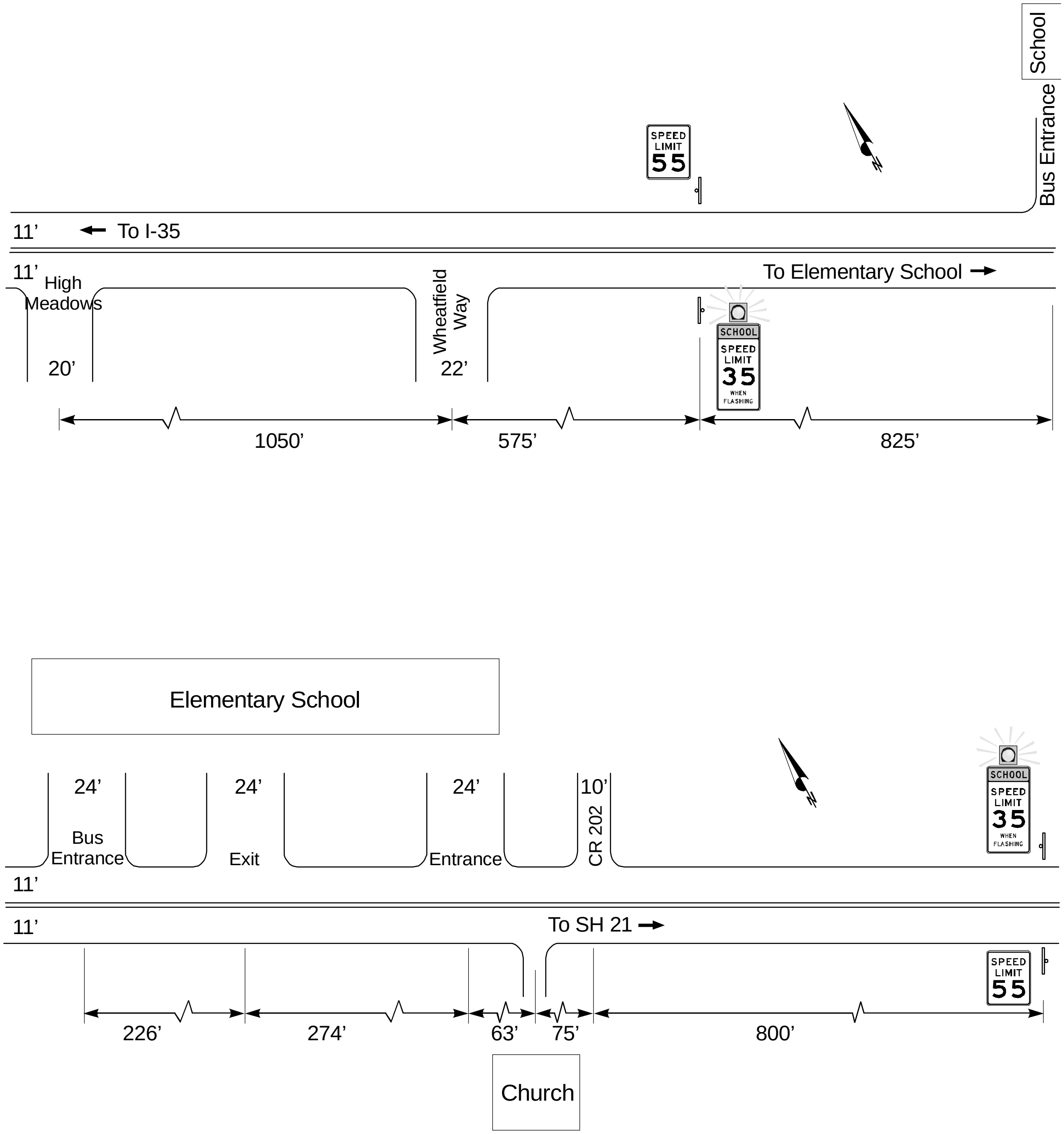
Most intersections on RM 150 are small driveways ranging from 10 to 27 ft (3.1 to 8.2 m) wide. East of the elementary school are two driveways, one for a church located on the south side of RM 150 (27 ft [8.2 m] wide) and one for a county road (CR) (CR 202, 10 ft [3.1 m] wide) located on the north side of RM 150. The school has three driveways, each approximately 25 ft (7.6 m) in width. The easternmost driveway is a combination entrance and exit. Approximately 270 ft (82 m) to the west (centerline to centerline) is the second driveway, which is an exit only. The last driveway is a school bus entrance, approximately 220 ft (67m) west of the second driveway.

The next intersection is approximately 1600 ft (488 m) west of the elementary school bus entrance. Wheatfield Way (22.5 ft [6.9 m] wide) is an entrance to a subdivision. High Meadows Lane (20 ft [6.1 m] wide) is also an entrance to a subdivision and is approximately 1050 ft (320 m) west of Wheatfield Way.

Figure 3-5. Site 3 (High Meadows).

Figure 3-6. Site 4 (Wheatfield Way).

Figure 3-7. Site 5 (Elementary School).



(1 ft = 0.305 m)

Figure 3-8. Intersection Detail at Site 3 (High Meadows), Site 4 (Wheatfield Way) and Site 5 (Entrances to Elementary School).

Site 6 is the intersection of SH 21 and FM 2001 near Niederwald, Texas. The intersection is located in Travis County, southwest of Niederwald. The site is at approximately level grade. The posted speed limit on SH 21 was 60 mph (97 km/h). Figure 3-9 presents the plan view of the data collection area, and Figure 3-10 shows photographs of the site.

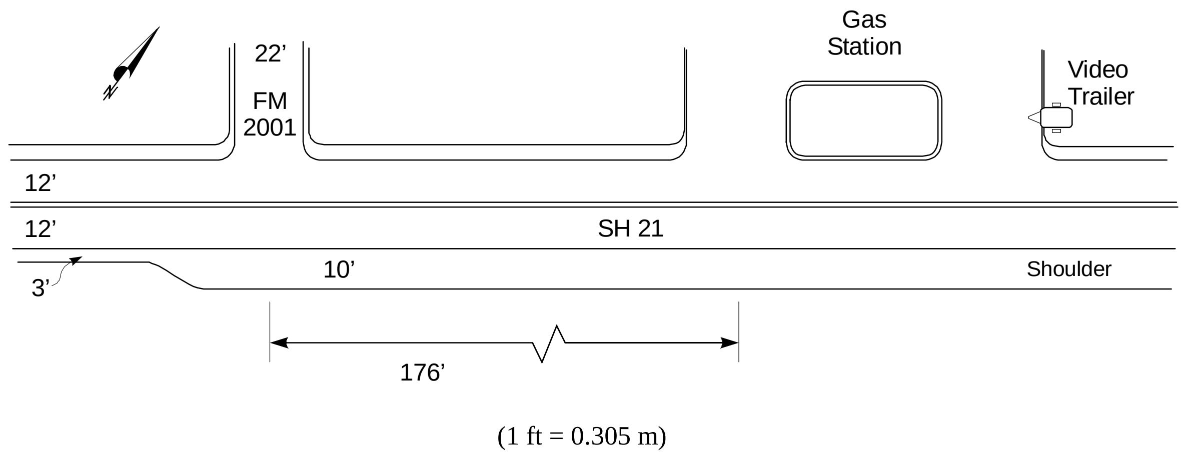


Figure 3-9. Intersection Detail at Site 6.

(a) FM 2001.

(b) Gas Station.

Figure 3-10. Site 6.

DETERMINATION OF PEAK HOUR

Vehicle volume counts of all directions and maneuvers were collected in 15-minute intervals from the video of the intersections, except at the Bryan, Texas, site where 20-minute intervals were used. The consecutive intervals were summed to identify peak and off-peak hours. The peak and off-peak hour periods are presented in Table 3-2.

Table 3-2. Peak and Off-Peak Data for Sites 1 through 6.

Site	Peak		Off-Peak	
	Time	Vehicles	Time	Vehicles
1 BD	4:40 pm – 5:40 pm	810	3:40 pm – 4:40 pm	576
2 EB	4:45 pm – 5:45 pm	460	8:00 am – 4:45 pm 5:45 pm – 6:00 pm	822
2 BD	5:00 pm – 6:00 pm	782	8:00 am – 5:00 pm	1857
3 WB	7:00 am – 8:00 am	201	4:00 pm – 5:00 pm 8:00 am – 9:00 am	97 102
3 EB	7:00 am – 8:00 am	173	4:00 pm – 5:00 pm 8:00 am – 9:00 am	151 78
4 WB	5:00 pm – 6:00 pm	111	8:00 am – 9:00 am 3:00 pm – 4:00 pm	83 95
4 EB	5:00 pm – 6:00 pm	166	8:00 am – 9:00 am 4:00 pm – 5:00 pm	63 112
5 WB	7:00 am – 8:00 am	132	5:00 pm – 6:00 pm	89
5 EB	7:00 am – 8:00 am	227	5:00 pm – 6:00 pm	127
6 BD	5:15 pm – 6:15 pm	692	8:45 am – 9:45 am 4:15 pm – 5:15 pm 3:15 pm – 4:15 pm	342 643 502

Notes:
BD = Both Directions
EB = Eastbound
WB = Westbound

SUBJECT MOVEMENT ANALYSIS

Once researchers established critical hours, they undertook a detailed study of those intervals. For each site, the movement of every subject vehicle was recorded during the critical hours. By using the movement codes, the movements were categorized into three areas:

1. through movements unaffected by turning movement,
2. through movements affected by turning movement, and
3. right- or left-turning movement.

The description of the codes is listed in Table 3-3. Tables 3-4 through 3-13 summarize the data available from the sites.

Table 3-3. Codes Used to Describe Movements at Intersections.

Digit	Codes
1st	• T = through movement • L = left-turn movement • R = right-turn movement • L_o = left-turn movement from sidestreet or entrance • R_o = right-turn movement from sidestreet or entrance
2nd	• U = unimpeded movement • S = vehicle used shoulder to pass • R_L = vehicle reduced speed because of left-turning vehicle • G = waited for opposing traffic to clear • Q = waited in queue for others to turn • R_E = slowed down due to vehicle turning from sidestreet or entrance to arterial • R_S = vehicle reduced speed and waited for car to enter arterial from shoulder
3rd	• L = turned left at sidestreet or entrance
4th	• G = waited for opposing traffic to clear
Other Inappropriate Driving Movements	• E* = performed said movement early and used opposing lanes • -B = movement was performed by a school bus and not another type of vehicle
Before the data were analyzed, a coding system was developed to aid in identifying the types of movements performed by the drivers approaching or driving through the subject intersection. The code for a movement can include as many as 5 digits. The first digit reflects the primary movement by the vehicle such as through, left, right, etc. The second digit provides information on how a left-turning vehicle affects the subject vehicle. The third digit is used when the subject vehicle is turning left before the intersection of interest, and the fourth digit reflects when the turning vehicle is waiting for an adequate gap. Subscripts are used on these digits to identify the street or entrance to a business being used by the subject vehicle. An additional digit was used to indicate if the movement was performed early and used the opposing lane ("E*") or if the subject vehicle was a school bus ("-B"). Site 5 was at an elementary school and the data for the school buses were tabulated separately.	

Table 3-4. Movements at Site 1 (Physicians Center).

Code	Movement	Peak Period			Off-Peak Period		
		Vehicles	S %*	C %**	Vehicles	S %	C %
Through movements unaffected by Physicians Center left turns							
TU	Straight through	115	24.1%	26.4%	81	26.3%	31.8%
$L_{O1}U$	Left out of Physicians Center, straight through	6	1.3%	1.4%	2	0.6%	0.8%
	Through movements affected by Copperfield Drive Left Turns[+]	**75**	**15.7%**	**17.2%**	**33**	**10.7%**	**12.9%**
	Left Turn movements at Copperfield[+]	**239**	**50.0%**	**54.9%**	**139**	**45.1%**	**54.5%**
	Total through movements unaffected by Physicians Center Left Turns	435			255		
Through movements affected by Physicians Center left turns							
TS_1L_3	**Shoulder at Physicians Center, left at Copperfield**	**5**	**1.0%**	**26.3%**	**9**	**2.9%**	**42.9%**
TS_1	**Shoulder at Physicians Center, straight through**	**8**	**1.7%**	**42.1%**	**4**	**1.3%**	**19.0%**
$TS_{1\text{-}3}$	**Shoulder from Physicians Center to Copperfield, straight through**	**4**	**0.8%**	**21.1%**	**5**	**1.6%**	**23.8%**
TS_E	**Shoulder early (prior to Physicians Center), straight through**	**1**	**0.2%**	**5.3%**	**0**	**0.0%**	**0.0%**
TS_1L_3G	**Shoulder at Physicians Center, waited for westbound traffic at Copperfield**	**0**	**0.0%**	**0.0%**	**1**	**0.3%**	**4.8%**
TR_{L1}	**Slowed and waited for car to turn at Physicians Center, straight through**	**0**	**0.0%**	**0.0%**	**1**	**0.3%**	**4.8%**
$TR_{L1}L_3$	**Slowed due to car turning at Physicians Center, left at Copperfield**	**1**	**0.2%**	**5.3%**	**1**	**0.3%**	**4.8%**
	Total through movements affected by Physicians Center Left Turns	19			21		
Left Turns at Physicians Center							
L_1U	Left at Physicians Center	18	3.8%	75.0%	28	9.1%	87.5%
L_1G	**Left at Physicians Center, waited for westbound traffic**	**5**	**1.0%**	**20.8%**	**4**	**1.3%**	**12.5%**
L_1E*	Turned early at Physicians Center, briefly drove in westbound lane	1	0.2%	4.2%	0	0.0%	0.0%
	Total Left Turns	24			32		
	Total Vehicles	**478**			**308**		

Note: Bold Text indicates impeded vehicles
* S % = Movement / Total Site Volume * 100
** C % = Movement / Total Category Movements * 100
[+] As listed in Table 3-6 and not shown elsewhere in this table.

Table 3-5. Movements at Site 1 (TAMU Driveway).

Code	Eastbound Movement	Peak Period		Off-Peak Period	
		Vehicles	%	Vehicles	%
Right Turns at TAMU					
R_2U	Right at TAMU	6	85.7%	2	100.0%
$L_{O1}R_2$	Left out of Physicians Center, right at TAMU	1	14.3%	0	0.0%
	Total Right Turns	7		2	

Table 3-6. Movements at Site 1 (Copperfield Drive).

Code	Movement	Peak Period			Off-Peak Period		
		Vehicles	S %*	C %**	Vehicles	S %*	C %**
Through movements unaffected by Copperfield Drive left turns							
TU	Straight through	115	25.2%	88.5%	81	29.2%	91.0%
TS_1	**Shoulder at Physicians Center, straight through**	**8**	**1.8%**	**6.2%**	**4**	**1.4%**	**4.5%**
$L_{O1}U$	Left out of Physicians Center, straight through	6	1.3%	4.6%	2	0.7%	2.2%
R_{O2}	Right out of TAMU, straight through	1	0.2%	0.8%	1	0.4%	1.1%
TR_{L1}	**Slowed and waited for car to turn at Physicians Center, straight through**	**0**	**0.0%**	**0.0%**	**1**	**0.4%**	**1.1%**
	Total through movements unaffected by Copperfield Drive Left Turns	130			89		
Through movements affected by Copperfield Drive left turns							
TS_3	**Shoulder at Copperfield, straight through**	**66**	**14.5%**	**81.5%**	**28**	**10.1%**	**73.7%**
TS_{1-3}	**Shoulder from Physicians Center to Copperfield, straight through**	**4**	**0.9%**	**4.9%**	**5**	**1.8%**	**13.2%**
TR_{E3}	**Straight through, slowed down due to car turning out of Copperfield**	**1**	**0.2%**	**1.2%**	**0**	**0.0%**	**0.0%**
TR_{L3}	**Slowed and waited for car to turn left at Copperfield**	**6**	**1.3%**	**7.4%**	**3**	**1.1%**	**7.9%**
TS_E	**Shoulder early (prior to Physicians Center), straight through**	**1**	**0.2%**	**1.2%**	**0**	**0.0%**	**0.0%**
$R_{O2}L_3$	**Right out of TAMU, slowed for turning car**	**1**	**0.2%**	**1.2%**	**0**	**0.0%**	**0.0%**
TR_S	**Slowed and waited for car to enter lane after shoulder**	**1**	**0.2%**	**1.2%**	**1**	**0.4%**	**2.6%**
$L_{O1}S_3$	**Left out of Physicians Center, shoulder at Copperfield, straight through**	**1**	**0.2%**	**1.2%**	**1**	**0.4%**	**2.6%**
	Total through movements affected by Copperfield Drive Left Turns	81			38		

Table 3-6. Movements at Site 1 (Copperfield Drive) (continued).

Code	Movement	Peak Period			Off-Peak Period		
		Vehicles	S %*	C %**	Vehicles	S %*	C %**
Left turns at Copperfield Drive							
L_3U	Left at Copperfield	147	32.2%	60.0%	128	46.2%	85.3%
TS_1L_3	**Shoulder at Physicians Center, left at Copperfield**	**5**	**1.1%**	**2.0%**	**9**	**3.2%**	**6.0%**
L_3G	**Left at Copperfield, waited for westbound traffic**	**28**	**6.1%**	**11.4%**	**8**	**2.9%**	**5.3%**
L_3Q_3	**Left at Copperfield, waited for others**	**62**	**13.6%**	**25.3%**	**2**	**0.7%**	**1.3%**
L_3E*	Turned early at Copperfield, briefly drove in westbound lane	1	0.2%	0.4%	0	0.0%	0.0%
$TR_{L1}L_3$	**Slowed due to car turning at Physicians Center, left at Copperfield**	**1**	**0.2%**	**0.4%**	**1**	**0.4%**	**0.7%**
$L_{O1}L_3$	Left out of Physicians Center, left at Copperfield	1	0.2%	0.4%	1	0.4%	0.7%
TS_1L_3G	**Shoulder at Physicians Center, waited for westbound traffic at Copperfield**	**0**	**0.0%**	**0.0%**	**1**	**0.4%**	**0.7%**
	Total Left Turns	245			150		
	Total Vehicles	**456**			**277**		

Note: Bold Text indicates impeded vehicles
* S % = Movement / Total Volume * 100
** C % = Movement / Total Category Movements * 100

Table 3-7. Movements at Site 2.

Code	Movement	Peak Period (5:00 pm - 6:00 pm)			Off-Peak Period (9 hours of data)		
		Vehicles	S %*	C %**	Vehicles	S %*	C %**
Through movements unaffected by major road left turns							
TU	Straight through	385	83.7%	98.5%	1968 (219)	85.2%	99.6%
TR_{E1}	**Slowed down due to car turning out of Associates Avenue**	**6**	**1.3%**	**1.5%**	**7 (1)**	**0.3%**	**0.4%**
	Total through movements unaffected by major road left turns	391			1975 (219)		
Through movements affected by major road left turns							
TS_1	**Shoulder due to car turning onto Associates Avenue**	**27**	**5.9%**	**87.1%**	**92 (10)**	**4.0%**	**79.3%**
TR_{L1}	**Slowed and waited for car to turn left at Associates Avenue**	**4**	**0.9%**	**12.9%**	**24 (3)**	**1.0%**	**20.7%**
	Total through movements affected by major road left turns	31			116 (13)		
Left turns							
L_1U	Left at Associates Avenue	18	3.9%	47.4%	136 (15)	5.9%	61.8%
L_1G	**Left at Associates Avenue, waited for gap**	**16**	**3.5%**	**42.1%**	**76 (8)**	**3.3%**	**34.5%**
L_1Q_1	**Left at Associates Avenue, waited for others**	**4**	**0.9%**	**10.5%**	**8 (1)**	**0.3%**	**3.6%**
	Total Left Turns	38			220 (24)		
	Total Vehicles	**460**			**2311 (257)**		

Note: Bold Text indicates impeded vehicles
* S % = Movement / Total Volume * 100
** C % = Movement / Total Category Movements * 100
Parentheses indicate the number of vehicles per hour

Table 3-8. Movements at Site 3.

Code	Movement	Peak Period (7:00 am - 8:00 am)			Off-Peak Period (4:00 pm - 5:00 pm)		
		Vehicles	**S %***	**C %****	**Vehicles**	**S %***	**C %****
Through movements unaffected by major road left turns							
TU	Straight through	196	97.5%	99.5%	97	100%	100%
TR_{E1}	**Slowed down due to car turning out of High Meadows**	**1**	**0.5%**	**0.5%**	**0**	**0**	**0**
	Total through movements unaffected by major road left turns	**197**			97		
Through movements affected by major road left turns							
TS_1	**Shoulder due to car turning onto High Meadows**	**0**	**0**	**0**	**0**	**0**	**0**
TR_{L1}	**Slowed and waited for car to turn left at High Meadows**	**2**	**1%**	**100%**	**0**	**0**	**0**
	Total through movements affected by major road left turns	2			0		
Left turns							
L_1U	Left at High Meadows	1	0.5%	50%	0	0	0
L_1G	**Left at High Meadows, waited for gap**	**1**	**0.5%**	**50%**	**0**	**0**	**0**
L_1Q_1	**Left at High Meadows, waited for others**	**0**	**0**	**0**	**0**	**0**	**0**
	Total Left Turns	2			0		
	Total Vehicles	**201**			**97**		

Note: Bold Text indicates impeded vehicles
* S % = Movement / Total Volume * 100
** C % = Movement / Total Category Movements * 100

Table 3-9. Movements at Site 4.

Code	Movement	Peak Period (5:00 pm - 6:00 pm)			Off-Peak Period (8:00 am - 9:00 am)		
		Vehicles	S %*	C %**	Vehicles	S %*	C %**
Through movements unaffected by major road left turns							
TU	Straight through	100	90.1%	100%	80	96.4%	100%
TR_{E1}	**Slowed down due to car turning out of Wheatfield**	**0**	**0**	**0**	**0**	**0**	**0**
	Total through movements unaffected by major road left turns	100			80		
Through movements affected by major road left turns							
TS_1	**Shoulder due to car turning onto Wheatfield**	**0**	**0**	**0**	**0**	**0**	**0**
TR_{L1}	**Slowed and waited for car to turn left at Wheatfield**	**1**	**0.9%**	**100%**	**0**	**0**	**0**
	Total through movements affected by major road left turns	1			0		
Left turns							
L_1U	Left at Wheatfield	8	7.2%	80%	3	3.6%	100%
L_1G	**Left at Wheatfield, waited for gap**	**1**	**0.9%**	**10%**	**0**	**0**	**0**
L_1Q_1	**Left at Wheatfield, waited for others**	**1**	**0.9%**	**10%**	**0**	**0**	**0**
	Total Left Turns	10			0		
	Total Vehicles	**111**			**83**		

Note: Bold Text indicates impeded vehicles
* S % = Movement / Total Volume * 100
** C % = Movement / Total Category Movements * 100

Table 3-10. Movements at Site 5 (School Entrance).

Code	Movement	Peak Period (10/02/02) (7:00 am - 8:00 am)			Off-Peak Period (10/01/02) (5:00 pm - 6:00 pm)		
		Vehicles	**S %***	**C %****	**Vehicles**	**S %***	**C %****
Through movements unaffected by major road left turns							
TU	Straight through	86	37.9%	100%	104	81.9%	100%
TR_{E1}	**Slowed down due to car turning out of School Entrance**	**0**	**0**	**0**	**0**	**0**	**0**
	Total through movements unaffected by major road left turns	86			104		
Through movements affected by major road left turns							
TS_1	**Shoulder due to car turning into School Entrance**	**0**	**0**	**0**	**0**	**0**	**0**
TR_{L1}	**Slowed and waited for car to turn left at School Entrance**	**30**	**13.2%**	**100%**	**8**	**6.3%**	**100%**
	Total through movements affected by major road left turns	30			8		
Left turns							
L_1U	Left at School Entrance	41	18.1%	36.9%	14	11.0%	93.3%
L_1G	**Left at School Entrance, waited for gap**	**34**	**15.0%**	**30.6%**	**1**	**0.8%**	**0.7%**
L_1Q_1	**Left at School Entrance, waited for others**	**36**	**15.9%**	**32.4%**	**0**	**0**	**0**
	Total Left Turns	111			15		
	Total Vehicles	**227**			**127**		

Note: Bold Text indicates impeded vehicles

* S % = Movement / Total Volume * 100

** C % = Movement / Total Category Movements * 100

Table 3-11. Movements at Site 5 (Bus Entrance).

Code	Movement	Peak Period (10/02/02) (7:00 am – 8:00 am)			Off-Peak Period (10/01/02) (5:00 pm - 6:00 pm)		
		Vehicles	**S %***	**C %****	**Vehicles**	**S %***	**C %****
Through movements unaffected by major road left turns							
TU	Straight through	184	88.0%	97.9%	123	95.3%	100%
TR_{E1}	**Slowed down due to car turning out of Bus Entrance**	**4**	**1.9%**	**2.1%**	**0**	**0**	**0**
	Total through movements unaffected by major road left turns	188			123		
Through movements affected by major road left turns							
TS_1	**Shoulder due to car turning into Bus Entrance**	**0**	**0**	**0**	**0**	**0**	**0**
TR_{L1}	**Slowed and waited for car to turn left at Bus Entrance**	**5**	**2.4%**	**100%**	**1**	**0.8%**	**100%**
	Total through movements affected by major road left turns	5			1		
Left turns							
L_1U	Left at Bus Entrance	14	6.7%	87.5%	5	3.9%	100%
L_1G	**Left at Bus Entrance, waited for gap**	**2**	**1.0%**	**12.5%**	**0**	**0**	**0**
L_1Q_1	**Left at Bus Entrance, waited for others**	**0**	**0**	**0**	**0**	**0**	**0**
	Total Left Turns	16			5		
	Total Vehicles	**209**			**129**		

Note: Bold Text indicates impeded vehicles
* S % = Movement / Total Volume * 100
** C % = Movement / Total Category Movements * 100

Table 3-12. Movements at Site 6 (FM 2001).

Code	Movement	Peak Hour (5:15 pm - 6:15 pm)			Off-Peak Period (3 hours of data)		
		Vehicles	**S %***	**C %****	**Vehicles**	**S %***	**C %****
Through movements unaffected by FM 2001 left turns							
TU	Straight through, unimpeded	154	64.2%	91.1%	430 (143)	66.8%	88.1%
	Through movements affected by left turns into Station+	**7**	**2.9%**	**4.1%**	**23 (8)**	**3.6%**	**4.7%**
	Left turns into Station+	**8**	**3.3%**	**4.7%**	**35 (12)**	**5.4%**	**7.1%**
	Total through movements unaffected by FM 2001 left turns	169			488 (163)		
Through movements affected by FM 2001 left turns							
TS_1L	**Shoulder at FM 2001, left at Station**	**0**	**0**	**0**	**1 (0)**	**0.2%**	**1.4%**
TS_1	**Shoulder at FM 2001, straight through**	**6**	**2.5%**	**18.8%**	**11 (4)**	**1.7%**	**15.1%**
TR_{L1}	**Slowed and waited for car to turn left at FM 2001**	**25**	**10.4%**	**78.1%**	**59 (20)**	**9.2%**	**80.8%**
$TR_{L1}L_2$	**Slowed due to car turning at FM 2001, left at Station**	**1**	**0.4%**	**3.1%**	**2 (1)**	**0.3%**	**2.7%**
	Total through movements affected by FM 2001 left turns	32			73 (24)		
Left turns at FM 2001							
L_1U	Left at FM 2001, unimpeded	14	5.8%	35.9%	46 (15)	7.1%	55.4%
L_1G	**Left at FM 2001, waited for westbound traffic**	**19**	**7.9%**	**48.7%**	**33 (11)**	**5.1%**	**39.8%**
L_1Q_1	**Left at FM 2001, waited in queue for others to turn off SH 21 to FM 2001**	**6**	**2.5%**	**15.4%**	**4 (1)**	**0.6%**	**4.8%**
	Total Left Turns	39			83 (28)		
	Total Vehicles	240			644 (215)		

Note: Bold text indicates impeded vehicles
* S % = Movement / Total Volume * 100
** C % = Movement / Total Category Movements * 100
+ As listed in Table 3-13 and not shown elsewhere in this table.
Parentheses indicate the number of vehicles per hour

Table 3-13. Movements at Site 6 (Gas Station).

Code	Movement	Peak Hour (5:15 pm - 6:15 pm)			Off-Peak Period (3 hours of data)		
		Vehicles	**S %***	**C %****	**Vehicles**	**S %***	**C %****
Through movements unaffected by left turns into Station							
TU	Straight through, unimpeded	154	76.6%	83.2%	430 (143)	76.6%	86.0%
TS_1	**Shoulder at FM 2001, straight through**	**6**	**3.0%**	**3.2%**	**11 (4)**	**2.0%**	**2.2%**
TR_{L1}	**Slowed and waited for car to turn left at FM 2001**	**25**	**12.3%**	**13.5%**	**59 (20)**	**10.5%**	**11.8%**
	Total through movements unaffected by left turns into Station	185			500 (167)		
Through movements affected by left turns into Station							
TS_2	**Shoulder at Station, straight through**	**2**	**1.0%**	**28.6%**	**1 (0)**	**0.2%**	**4.3%**
TR_{L2}	**Slowed and waited for car to turn left at Station**	**5**	**2.5%**	**71.4%**	**19 (7)**	**3.4%**	**82.6%**
TR_{E2}	**Straight through, slowed down due to car turning out of Station**	**0**	**0**	**0**	**3 (1)**	**0.5%**	**13.0%**
	Total through movements affected by left turns into Station	7			23 (8)		
Left turns into Station							
L_2U	Left at Station	4	2.0%	44.4%	26 (9)	5.2%	68.4%
L_2G	**Left at Station, waited for westbound Traffic**	**3**	**1.5%**	**33.3%**	**8 (3)**	**1.4%**	**21.1%**
L_2Q_2	**Left at Station, waited for others to turn off SH 21 into Station**	**1**	**0.5%**	**11.1%**	**1 (0)**	**0.2%**	**2.6%**
TS_1L	**Shoulder at FM 2001, left at Station**	**0**	**0**	**0**	**1 (0)**	**0.2%**	**2.6%**
$TR_{L1}L_2$	**Slowed due to car turning at FM 2001, left at Station**	**1**	**0.5%**	**11.1%**	**2 (1)**	**0.4%**	**5.3%**
	Total Left Turns	9			38 (13)		
	Total Vehicles	201			561 (187)		

Note: Bold text indicates impeded vehicles
* S % = Movement / Total Volume * 100
** C % = Movement / Total Category Movements * 100
Parentheses indicate the number of vehicles per hour

IMPEDED MOVEMENT ANALYSIS

The movement categories were analyzed according to impeded vehicles. The impeded vehicle categories are indicated by bold text in Tables 3-4 through 3-13. These are either through or left-turning vehicles whose movements were impeded by a left-turning vehicle or opposing traffic. The vehicles studied had either a choice to wait for either the vehicle(s) to clear, to wait for a gap to perform a movement, or to use the shoulder to pass the vehicle. Tables 3-14 through 3-22 summarize the impeded movements for the through movements and left-turning movements. The impeded vehicle percentage is the number of impeded vehicles divided by the total number of vehicles. The shoulder vehicle percentage is that of the number of vehicles using the shoulder as compared to the total number of impeded vehicles.

Table 3-14. Impeded Vehicle Summary at Site 1 (Physicians Center).

	Peak Hour		Off-Peak Hours	
	No. Vehicles	**%**	**No. Vehicles**	**%**
Total Vehicles	478		308	
Impeded Vehicles	19	4*	21	7*
Shoulder Drivers	18	95**	19	90**

* Percent = Impeded Vehicles / Total Vehicles
** Percent = Shoulder Drivers / Impeded Vehicles

	Peak Hour		Off-Peak Hours	
	No. Vehicles	**%***	**No. Vehicles**	**%***
Total Left Turn Vehicles	24		32	
Impeded Vehicles	5	21	4	13

* Percent = Impeded Vehicles / Total Vehicles

Table 3-15. Impeded Vehicle Summary at Site 1 (Copperfield Drive).

	Peak Hour		Off-Peak Hours	
	No. Vehicles	%	No. Vehicles	%
Total Vehicles	456		277	
Impeded Vehicles	81	18*	38	14*
Shoulder Drivers	72	89**	34	89**

* Percent = Impeded Vehicles / Total Vehicles
** Percent = Shoulder Drivers / Impeded Vehicles

	Peak Hour		Off-Peak Hours	
	No. Vehicles	%*	No. Vehicles	%*
Total Left Turn Vehicles	245		150	
Impeded Vehicles	96	39	21	14

* Percent = Impeded Vehicles / Total Vehicles

Table 3-16. Impeded Vehicle Summary at Site 2.

	Peak Hour		Off-Peak Hours	
	No. Vehicles	%	No. Vehicles	%
Total Vehicles	460		2311 (257)	
Impeded Vehicles	31	7*	116 (13)	5*
Shoulder Drivers	27	87**	92 (10)	79**

* Percent = Impeded Vehicles / Total Vehicles
** Percent = Shoulder Drivers / Impeded Vehicles

	Peak Hour		Off-Peak Hours	
	No. Vehicles	%*	No. Vehicles	%*
Total Left Turn Vehicles	38		220 (24)	
Impeded Vehicles	20	53	84 (9)	38

* Percent = Impeded Vehicles / Total Left Turn Vehicles

Table 3-17. Impeded Vehicle Summary at Site 3.

	Peak Hour		Off-Peak Hour	
	No. Vehicles	**%**	**No. Vehicles**	**%**
Total Vehicles	201		97	
Impeded Vehicles	2	1*	0	0*
Shoulder Drivers	0	0**	0	--**

* Percent = Impeded Vehicles / Total Vehicles
** Percent = Shoulder Drivers / Impeded Vehicles

	Peak Hour		Off-Peak Hour	
	No. Vehicles	**%***	**No. Vehicles**	**%***
Total Left Turn Vehicles	2		0	
Impeded Vehicles	1	50	0	--

* Percent = Impeded Vehicles / Total Left Turn Vehicles

Table 3-18. Impeded Vehicle Summary at Site 4.

	Peak Hour		Off-Peak Hour	
	No. Vehicles	**%**	**No. Vehicles**	**%**
Total Vehicles	111		83	
Impeded Vehicles	1	1	0	0*
Shoulder Drivers	0	0**	0	--**

* Percent = Impeded Vehicles / Total Vehicles
** Percent = Shoulder Drivers / Impeded Vehicles

	Peak Hour		Off-Peak Hour	
	No. Vehicles	**%***	**No. Vehicles**	**%***
Total Left Turn Vehicles	10		3	
Impeded Vehicles	3	30	0	0

* Percent = Impeded Vehicles / Total Left Turn Vehicles

Table 3-19. Impeded Vehicle Summary at Site 5 (School Entrance).

	Peak Hour (10/02/02)		Off-Peak Hour (10/01/02)	
	No. Vehicles	**%**	**No. Vehicles**	**%**
Total Vehicles	227		127	
Impeded Vehicles	30	13*	8	6*
Shoulder Drivers	0	0**	0	0**
* Percent = Impeded Vehicles / Total Vehicles ** Percent = Shoulder Drivers / Impeded Vehicles				
	Peak Hour (10/02/02)		**Off-Peak Hour (10/01/02)**	
	No. Vehicles	**%***	**No. Vehicles**	**%***
Total Left Turn Vehicles	111		15	
Impeded Vehicles	70	63	1	1
* Percent = Impeded Vehicles / Total Left Turn Vehicles				

Table 3-20. Impeded Vehicle Summary at Site 5 (Bus Entrance).

	Peak Hour (10/02/02)		Off-Peak Hour (10/01/02)	
	No. Vehicles	**%**	**No. Vehicles**	**%**
Total Vehicles	209		129	
Impeded Vehicles	5	2*	1	1*
Shoulder Drivers	0	0**	0	0**
* Percent = Impeded Vehicles / Total Vehicles ** Percent = Shoulder Drivers / Impeded Vehicles				
	Peak Hour (10/02/02)		**Off-Peak Hour (10/01/02)**	
	No. Vehicles	**%***	**No. Vehicles**	**%***
Total Left Turn Vehicles	16		5	
Impeded Vehicles	2	13	0	0
* Percent = Impeded Vehicles / Total Left Turn Vehicles				

Table 3-21. Impeded Vehicle Summary at Site 6 (FM 2001).

	Peak Hour		Off Peak Hours	
	No. Vehicles	**%**	**No. Vehicles**	**%**
Total Vehicles	240		644 (215)	
Impeded Vehicles	32	13*	73 (24)	11*
Shoulder Drivers	6	19**	12 (4)	16**

* Percent = Impeded Vehicles / Total Vehicles
** Percent = Shoulder Drivers / Impeded Vehicles

	Peak Hour		Off Peak Hours	
	No. Vehicles	**%***	**No. Vehicles**	**%***
Total Left Turn Vehicles	39		83 (28)	
Impeded Vehicles	25	64	37 (12)	45

* Percent = Impeded Vehicles / Total Left Turn Vehicles

Table 3-22. Impeded Vehicle Summary at Site 6 (Gas Station).

	Peak Hour		Off Peak Hours	
	No. Vehicles	**%**	**No. Vehicles**	**%**
Total Vehicles	201		561 (187)	
Impeded Vehicles	7	3*	23 (8)	4*
Shoulder Drivers	2	29**	1 (0)	4**

* Percent = Impeded Vehicles / Total Vehicles
** Percent = Shoulder Drivers / Impeded Vehicles

	Peak Hour		Off Peak Hours	
	No. Vehicles	**%***	**No. Vehicles**	**%***
Total Left Turn Vehicles	9		38 (13)	
Impeded Vehicles	5	56	12 (4)	32

* Percent = Impeded Vehicles / Total Left Turn Vehicles

SPEED DATA RESULTS

Researchers calculated speed statistics for through movements, movements involving the use of the shoulder, and all the combined movements. The speed statistics used the speeds measured within 25 ft (7.6 m) of either side of the intersection or target area. Tables 3-23 through 3-28 present the statistical values for three of the sites.

The following statistics were determined using the specified ranges for each site:

- number of vehicles recorded with the lidar gun within the specified range,
- total number of vehicles recorded with the lidar gun,
- percentage of vehicles recorded with lidar gun in specified range,
- 85th percentile speeds,
- average of speeds recorded,
- standard deviation of speeds recorded,
- minimum speed recorded, and
- maximum speed recorded.

Table 3-23. Statistical Analysis of Speed Data at Site 1.

Movement	No. Recorded* (veh)	No. Lasered** (veh)	%***	85th Percentile (mph)	Average (mph)	Standard Deviation (mph)	Minimum (mph)	Maximum (mph)
TU	15	381	4	65	57	6.6	47	71
TS_3	18	18	100	64	55	9.1	22	67
All Movements	225	543	41	61	49	11.5	11	71

* The number of cars that were recorded within the range of 3387 ft and 3437 ft
** The total number of cars recorded with the lidar gun
*** Percentage of cars that were recorded within the range of 3387 ft and 3437 ft
1 mph = 1.6 km/h, 1 ft = 0.305 m

Table 3-24. Statistical Analysis of Speed Data at Site 2.

Movement	No. Recorded* (veh)	No. Lasered** (veh)	%***	85th Percentile (mph)	Average (mph)	Standard Deviation (mph)	Minimum (mph)	Maximum (mph)
TU	186	261	71	56	51	5.9	11	65
TS_1	1	1	100	52	–	–	–	–
All Movements	191	283	67	56	50	8.2	8	65

* The number of cars that were recorded within the range of 1238 ft and 1288 ft
** The total number of cars recorded with the lidar gun
*** Percentage of cars that were recorded within the range of 1238 ft and 1288 ft
1 mph = 1.6 km/h, 1 ft = 0.305 m

Table 3-25. Statistical Analysis of Speed Data at Site 3.

Movement	No. Recorded* (veh)	No. Lasered** (veh)	%***	85th Percentile (mph)	Average (mph)	Standard Deviation (mph)	Minimum (mph)	Maximum (mph)
TU	161	259	62	56	49	6.9	34	73
TS_1	0	0	0	–	–	–	–	–
All Movements	170	273	62	56	48	7.9	5	73

* The number of cars that were recorded within the range of 197 ft and 247 ft
** The total number of cars recorded with the lidar gun
*** Percentage of cars that were recorded within the range of 197 ft and 247 ft
1 mph = 1.6 km/h, 1 ft = 0.305 m

Table 3-26. Statistical Analysis of Speed Data at Site 4.

Movement	No. Recorded* (veh)	No. Lasered** (veh)	%***	85th Percentile (mph)	Average (mph)	Standard Deviation (mph)	Minimum (mph)	Maximum (mph)
TU	179	268	67	57	49	8.5	25	70
TS_1	0	0	0	–	–	–	–	–
All Movements	198	290	68	57	47	10.6	6	70

* The number of cars that were recorded within the range of 155 ft and 205 ft
** The total number of cars recorded with the lidar gun
*** Percentage of cars that were recorded within the range of 155 ft and 205 ft
1 mph = 1.6 km/h, 1 ft = 0.305 m

Table 3-27. Statistical Analysis of Speed Data at Site 5.

Movement	No. Recorded* (veh)	No. Lasered** (veh)	%***	85th Percentile (mph)	Average (mph)	Standard Deviation (mph)	Minimum (mph)	Maximum (mph)
TU	197	245	80	52	40	10	21	65
TS_1	0	0	0	--	--	--	--	--
All Movements	345	453	76	49	34	12	5	65

* The number of cars that were recorded within the range of 175 ft and 225 ft
** The total number of cars recorded with the lidar gun
*** Percentage of cars that were recorded within the range of 175 ft and 225 ft
1 mph = 1.6 km/h, 1 ft = 0.305 m

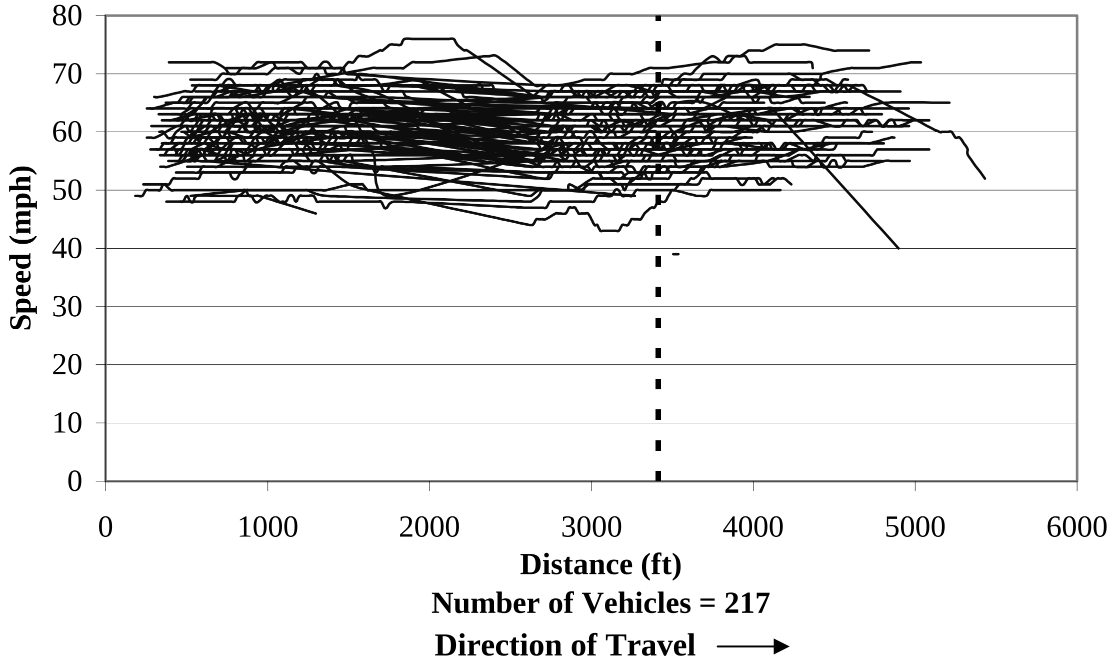
Table 3-28. Statistical Analysis of Speed Data at Site 6.

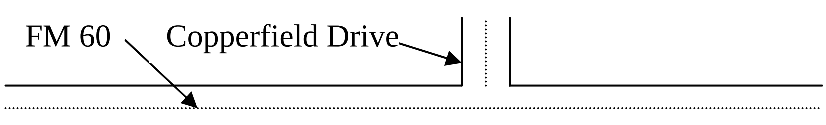
Movement	No. Recorded* (veh)	No. Lasered** (veh)	%***	85th Percentile (mph)	Average (mph)	Standard Deviation (mph)	Minimum (mph)	Maximum (mph)
TU	181	325	56	52	56	6.1	38	71
TS_2	1	1	100	37	–	–	–	–
All Movements	213	394	54	61	51	12.2	12	71

* The number of cars that were recorded within the range of 355 ft and 405 ft
** The total number of cars recorded with the lidar gun
*** Percentage of cars that were recorded within the range of 355 ft and 405 ft
1 mph = 1.6 km/h, 1 ft = 0.305 m

For each of the study sites, the speed data were used to construct a speed profile plot. The speed profile plots show the speeds of each vehicle recorded by the lidar gun over the distance from the observer. The speed profiles are separated by movement. Also plotted on the graph is the location(s) of each of the intersections or in the case of Site 6, the target distance (i.e., change in pavement). Figure 3-11 to Figure 3-29 show the recorded speed profiles.

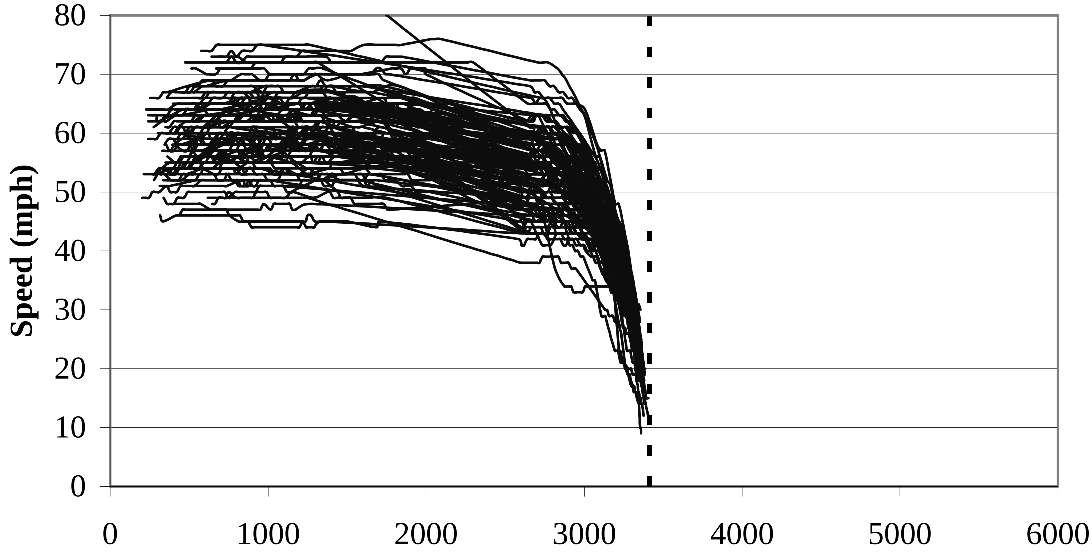
Figure 3-28 compares the 85th percentile speeds along sites 3 to 5 during and not during an active school speed zone. These free-flow speeds are approaching the school in the eastbound direction (moving left to right on the graph) for distances greater than 2000 ft (610 m) and approaching Wheatfield and High Meadows in the westbound direction for distances less than 1600 ft (488 m). Each point represents between 14 and 333 recorded speeds with an average of 181 speeds per point. Figure 3-28 illustrates that the school zone is more effective near the entrance of the school.





(1 mph = 1.61 km/h, 1 ft = 0.305 m)

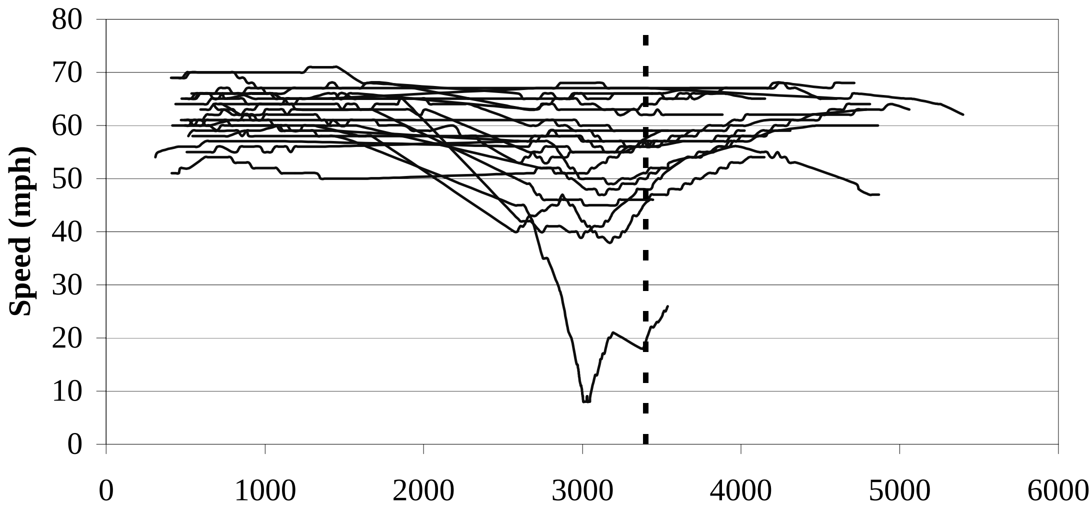
Figure 3-11. Speed Profile for Straight Through Unimpeded at Site 1.



Distance (ft)
Number of Vehicles = 280
Direction of Travel ⟶

(1 mph = 1.61 km/h, 1 ft = 0.305 m)

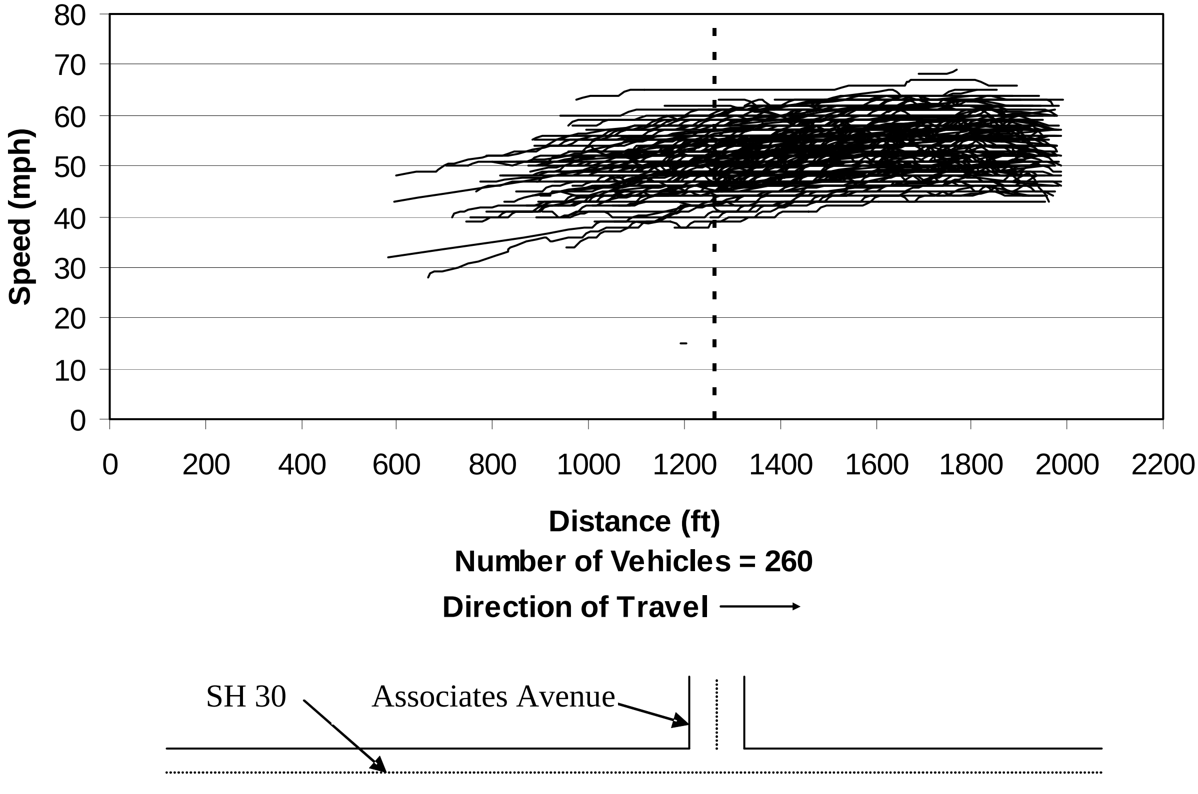
Figure 3-12. Speed Profile for Left Turns at Site 1 (Copperfield).



Distance (ft)
Number of Vehicles = 10
Direction of Travel ⟶

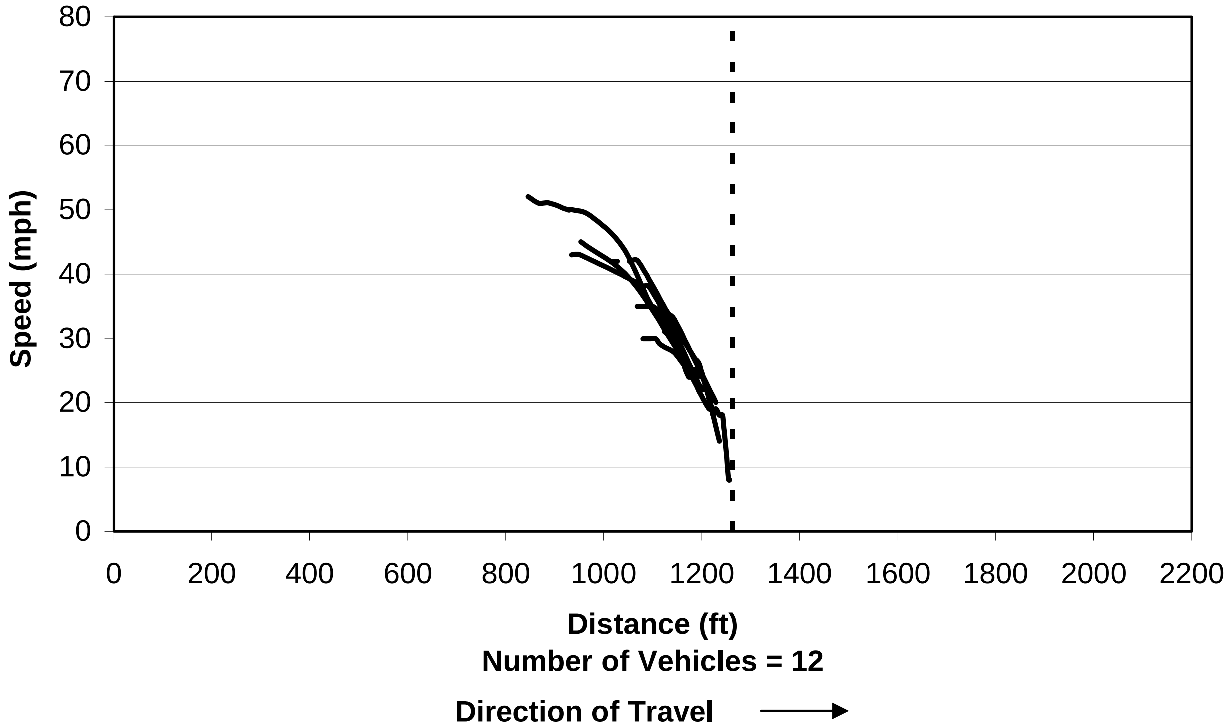
(1 mph = 1.61 km/h, 1 ft = 0.305 m)

Figure 3-13. Speed Profile for Those Driving on Shoulder and Then Straight Through at Site 1 (Copperfield).



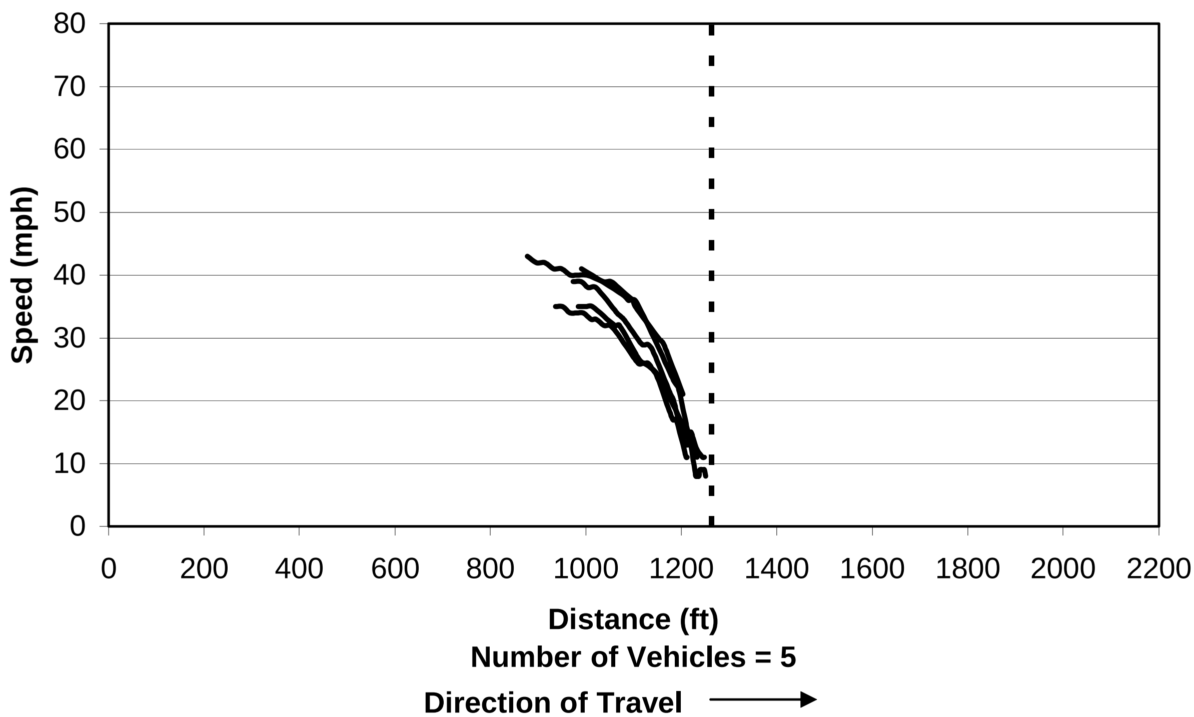
(1 mph = 1.61 km/h, 1 ft = 0.305 m)

Figure 3-14. Speed Profile for Straight Through Unimpeded at Site 2.



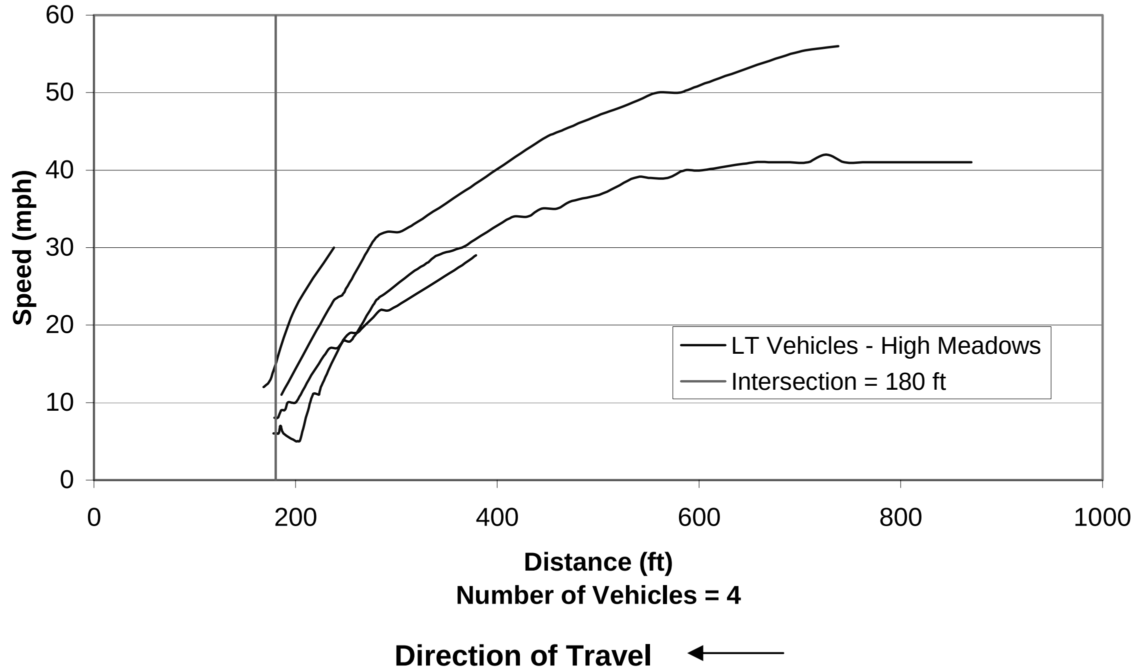
(1 mph = 1.61 km/h, 1 ft = 0.305 m)

Figure 3-15. Speed Profile for Left Turn at Site 2.



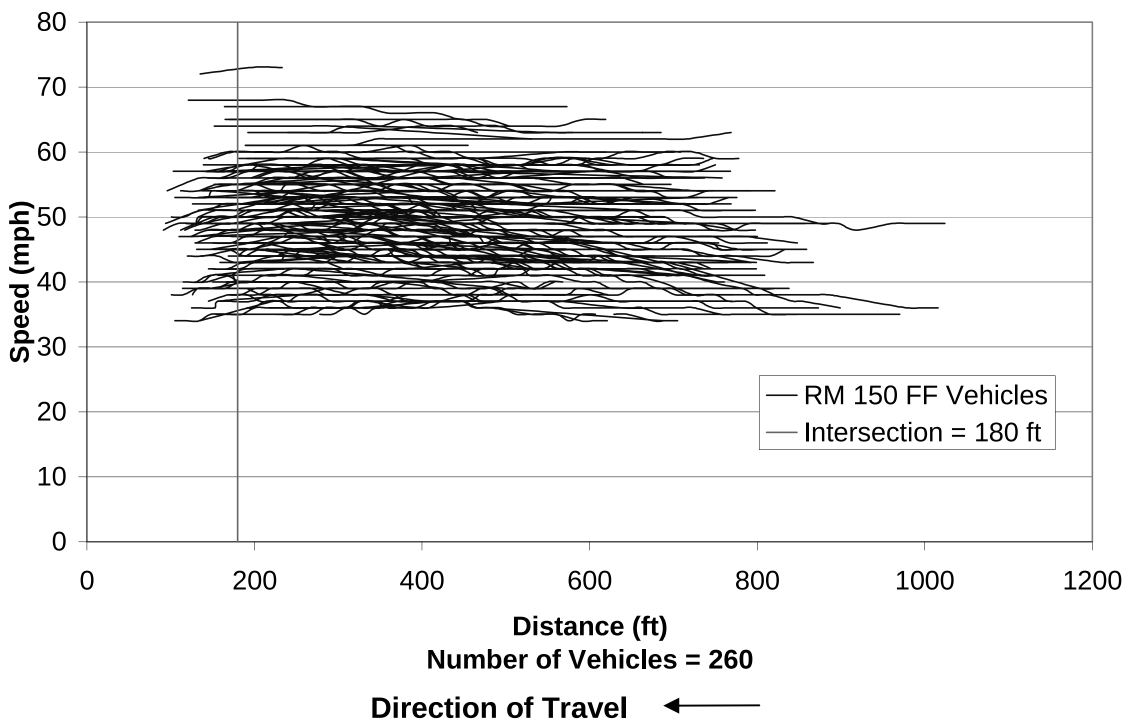
(1 mph = 1.61 km/h, 1 ft = 0.305 m)

Figure 3-16. Speed Profile for Left Turn after Waiting for a Gap at Site 2.



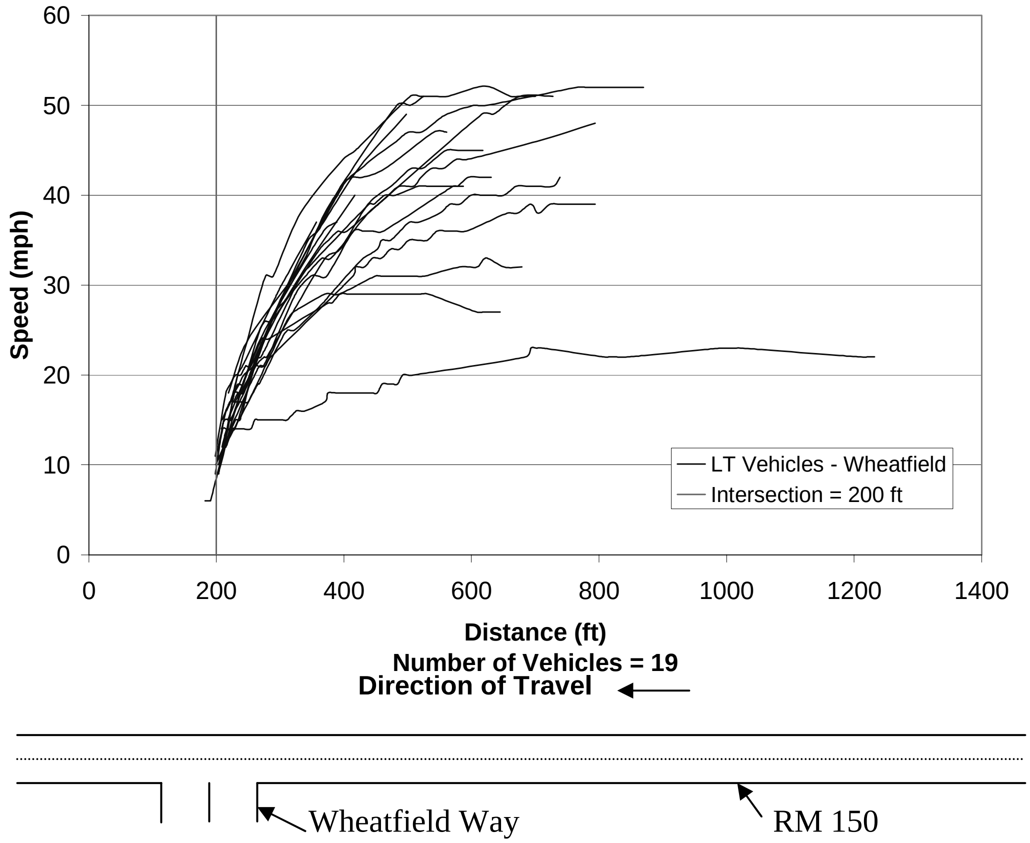
(1 mph = 1.61 km/h, 1 ft = 0.305 m)

Figure 3-17. Speed Profile for Left Turns at Site 3 (High Meadows).



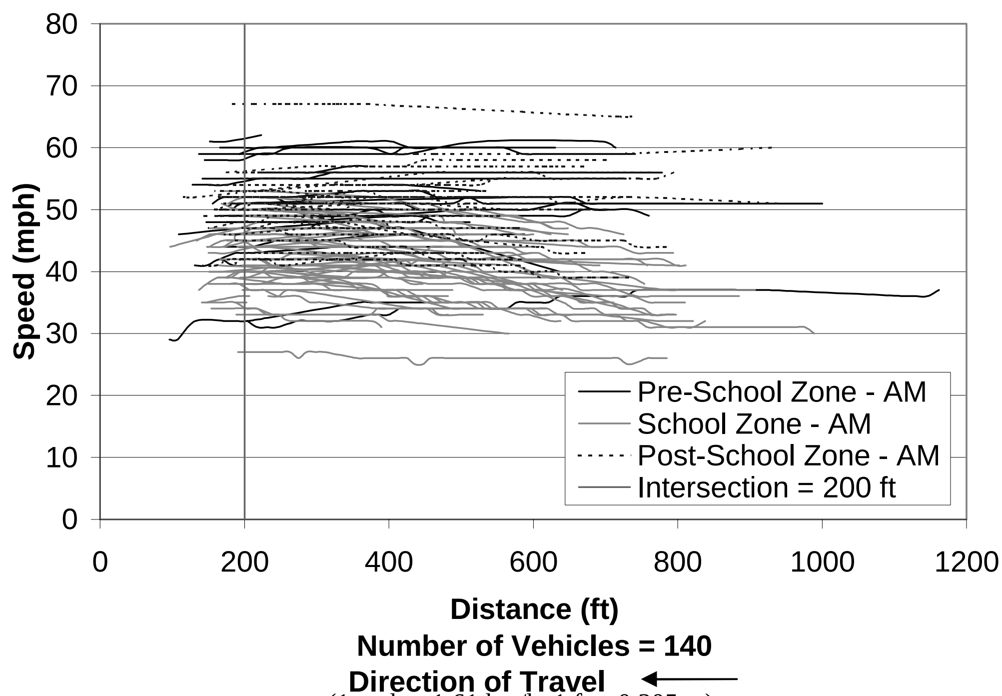
(1 mph = 1.61 km/h, 1 ft = 0.305 m)

Figure 3-18. Speed Profile for Straight Through Unimpeded at Site 3 (High Meadows).



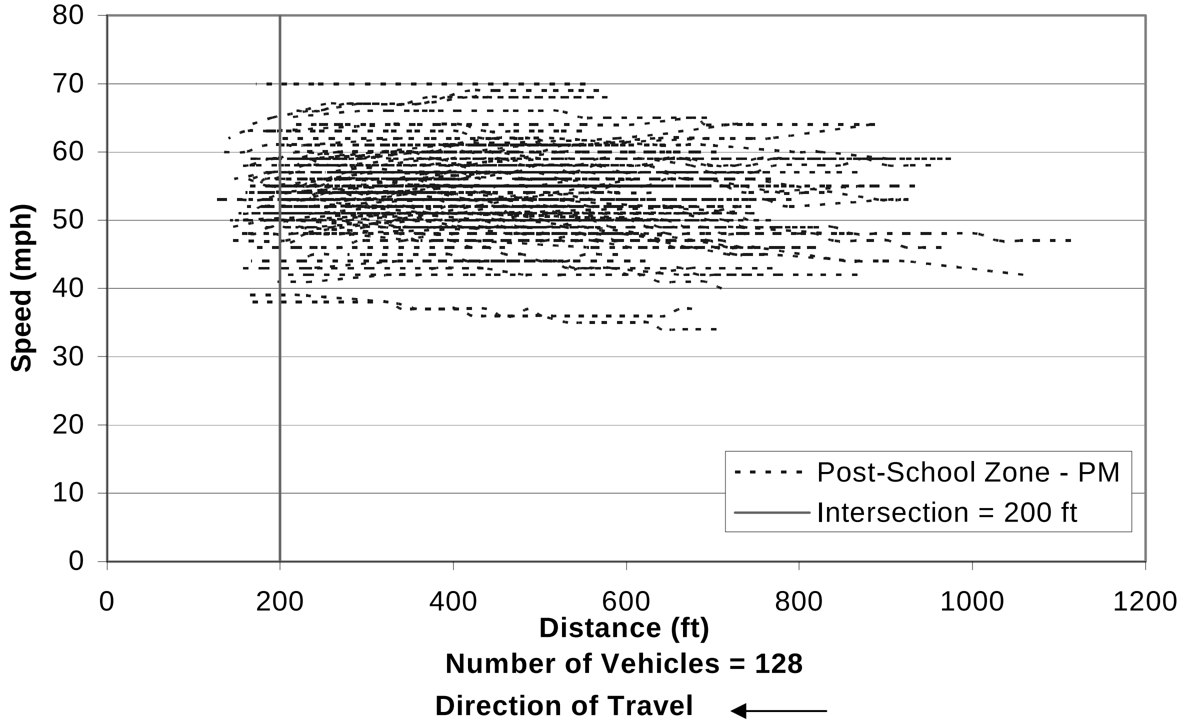
(1 mph = 1.61 km/h, 1 ft = 0.305 m)

Figure 3-19. Speed Profiles for Left Turns at Site 4 (10/03 - 10/04/02).



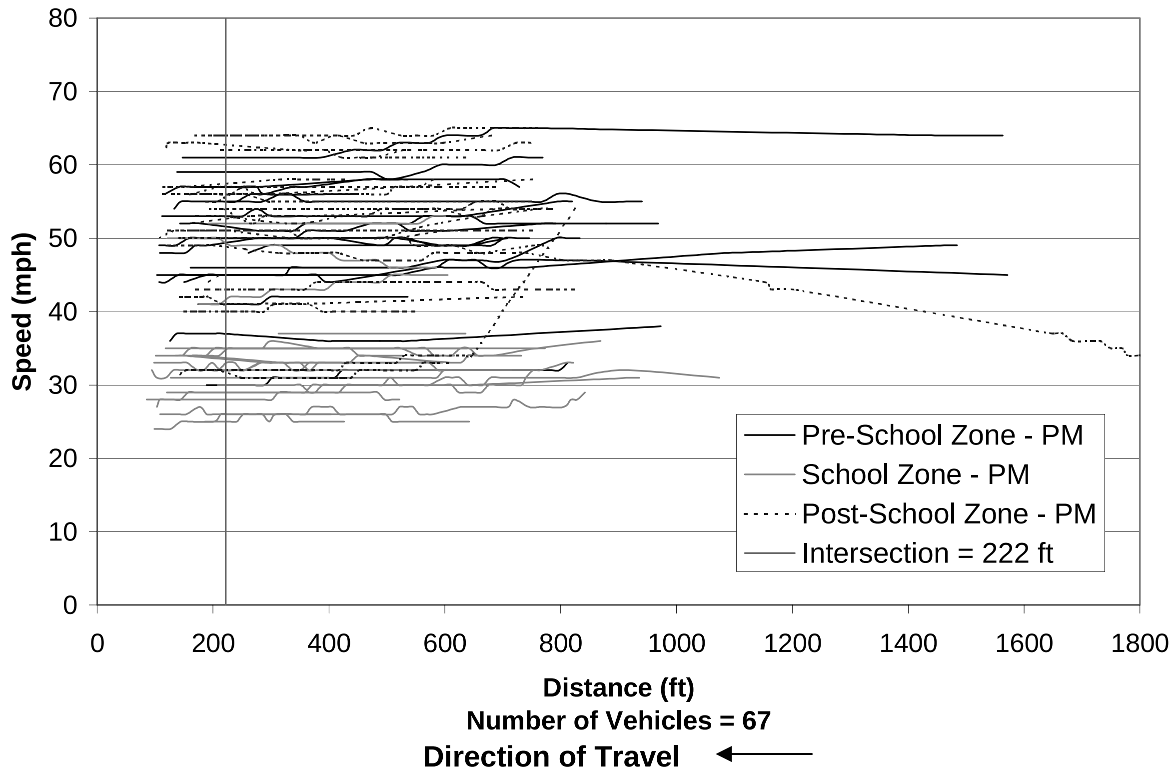
(1 mph = 1.61 km/h, 1 ft = 0.305 m)

Figure 3-20. Speed Profile for Straight Through Unimpeded at Site 4 (10/04/02 AM).



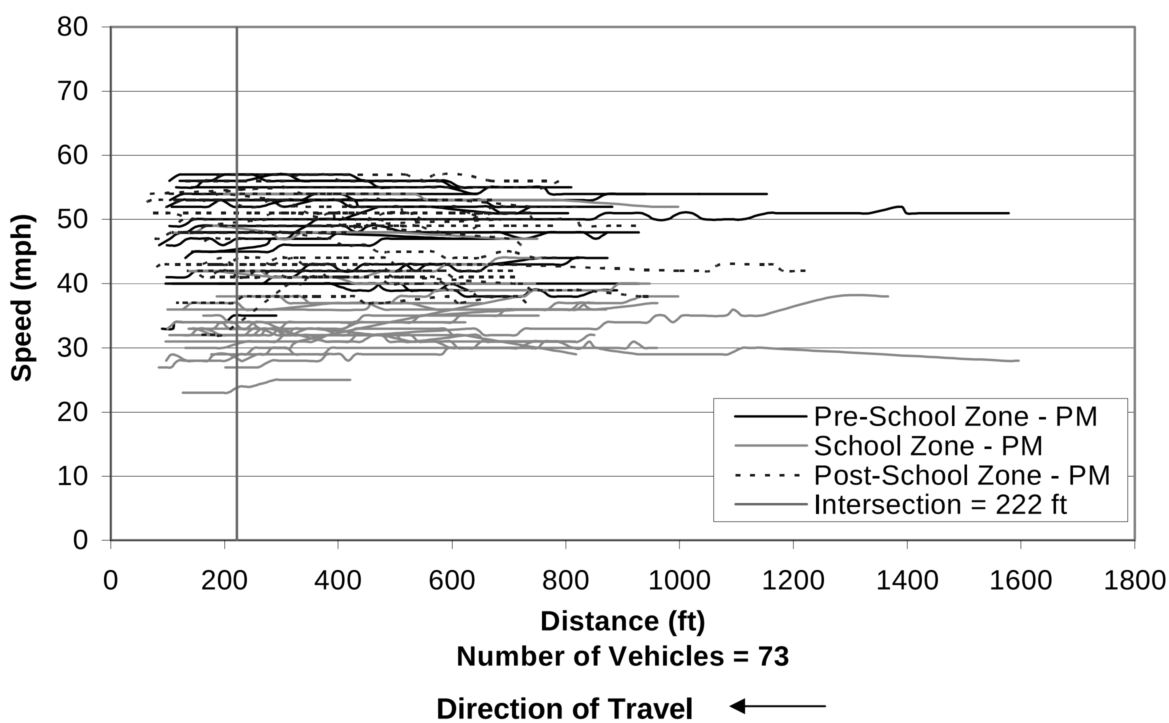
(1 mph = 1.61 km/h, 1 ft = 0.305 m)

Figure 3-21. Speed Profile for Straight Through Unimpeded at Site 4 (10/03/02 PM).



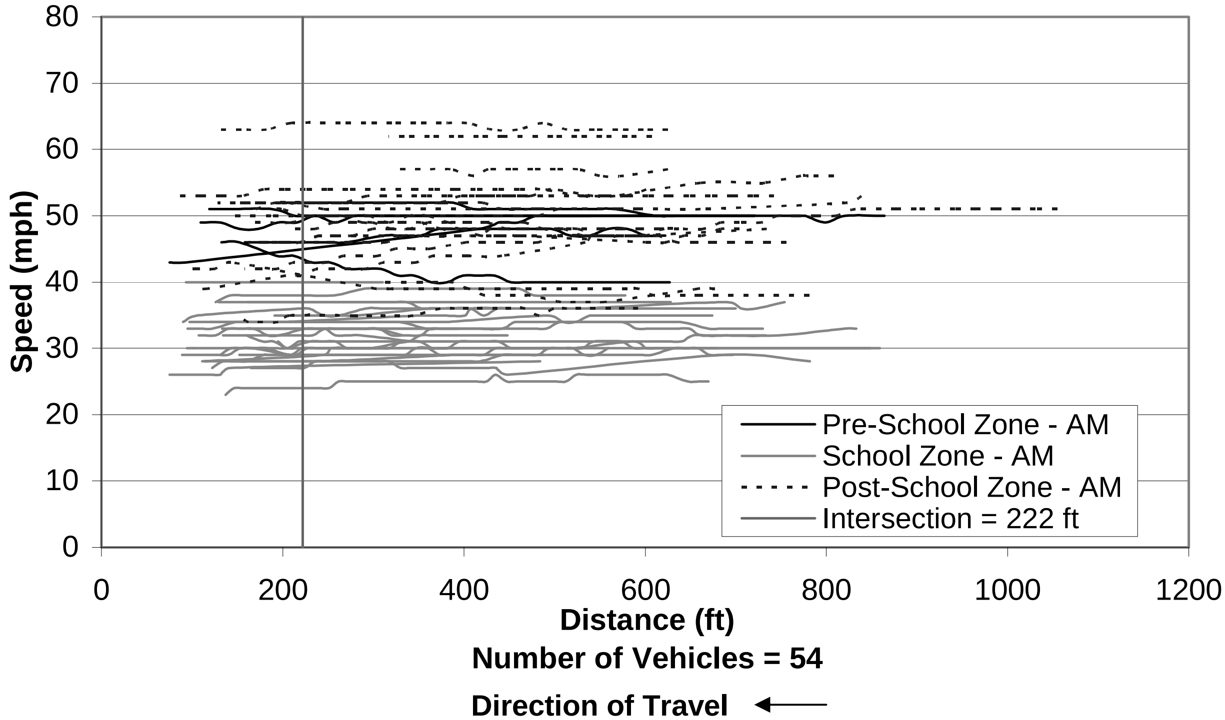
(1 mph = 1.61 km/h, 1 ft = 0.305 m)

Figure 3-22. Speed Profile for Straight Through Unimpeded at Site 5 (9/30/02 PM).



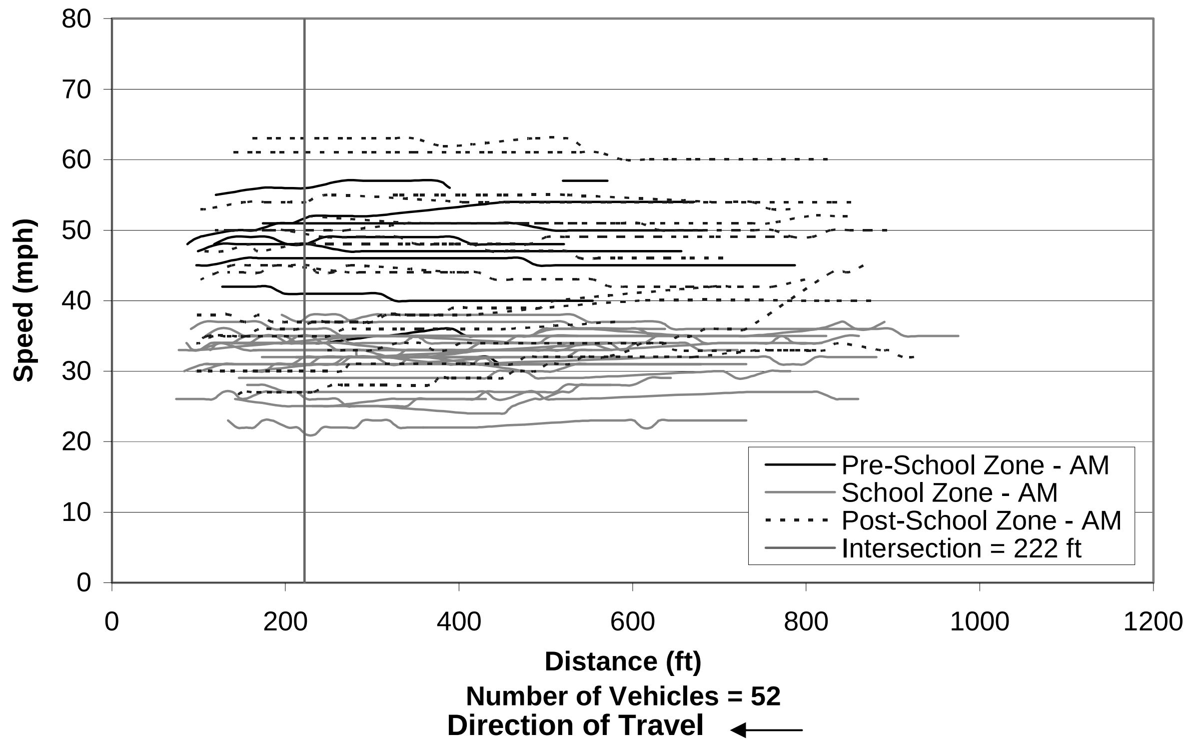
(1 mph = 1.61 km/h, 1 ft = 0.305 m)

Figure 3-23. Speed Profile for Straight Through Unimpeded at Site 5 (10/01/02 PM).



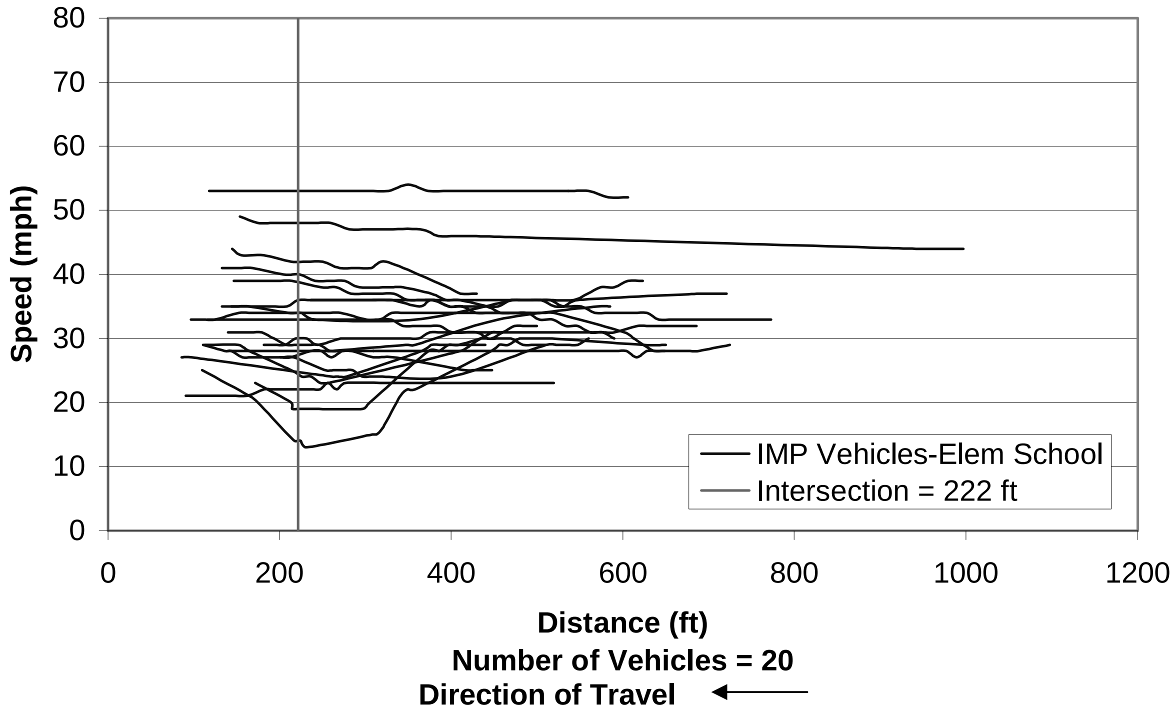
(1 mph = 1.61 km/h, 1 ft = 0.305 m)

Figure 3-24. Speed Profile for Straight Through Unimpeded at Site 5 (10/01/02 AM).



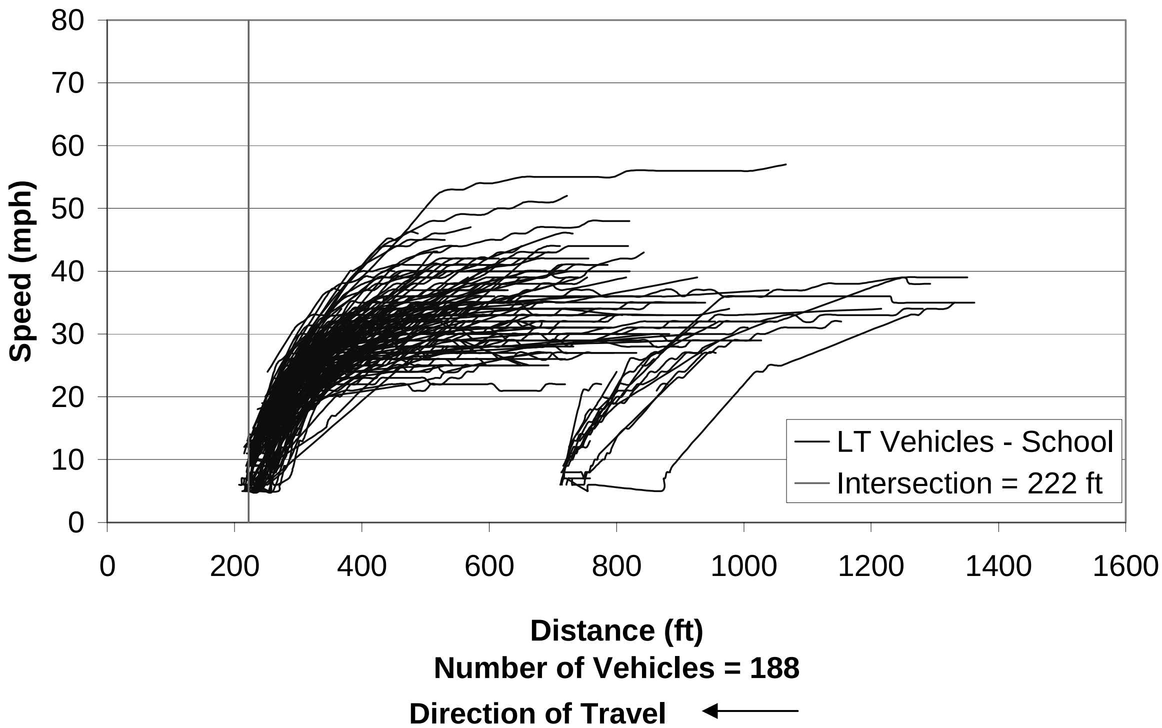
(1 mph = 1.61 km/h, 1 ft = 0.305 m)

Figure 3-25. Speed Profile for Straight Through Unimpeded at Site 5 (10/02/02 AM).



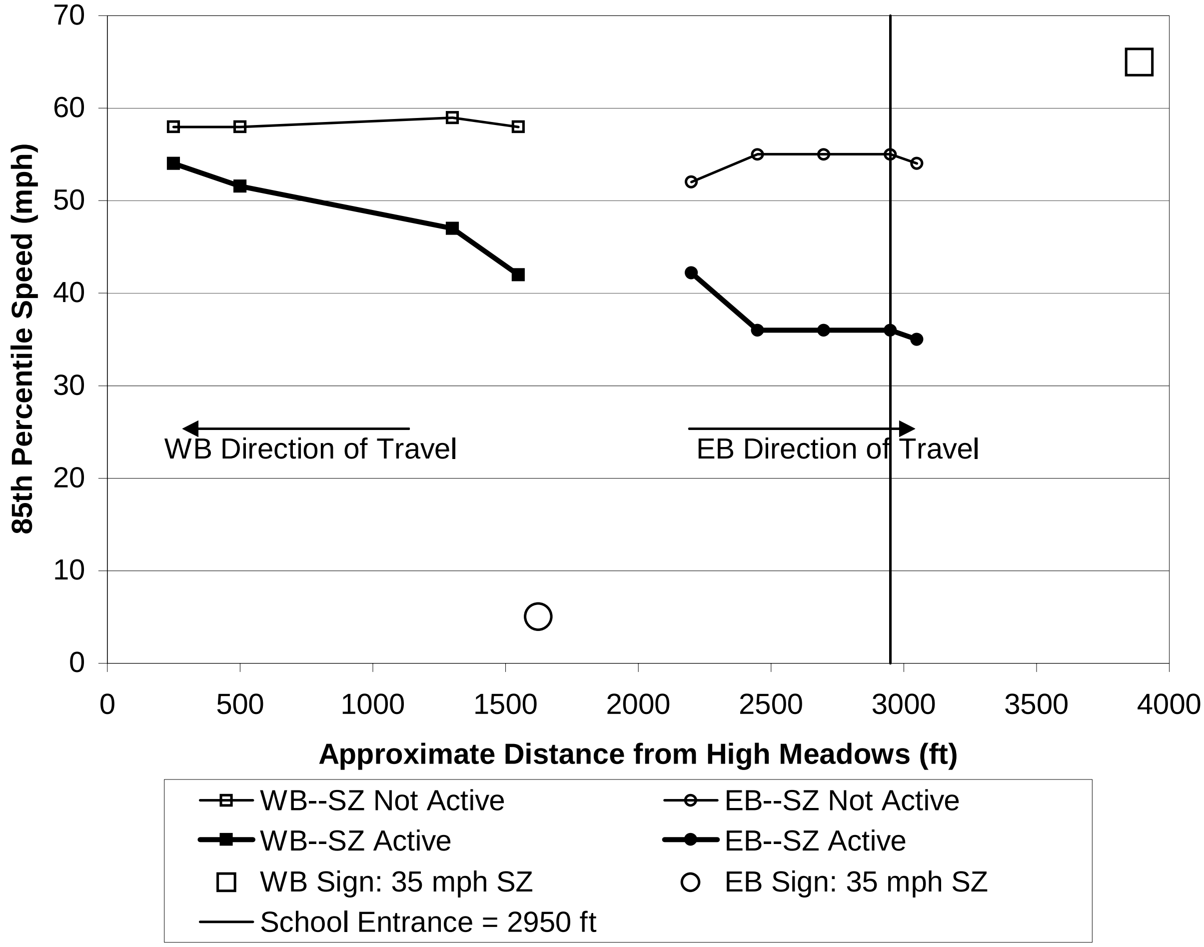
(1 mph = 1.61 km/h, 1 ft = 0.305 m)

Figure 3-26. Speed Profile for Straight Through Movement Affected by Vehicles Turning onto Major at Site 5 (9/30 - 10/02/02).



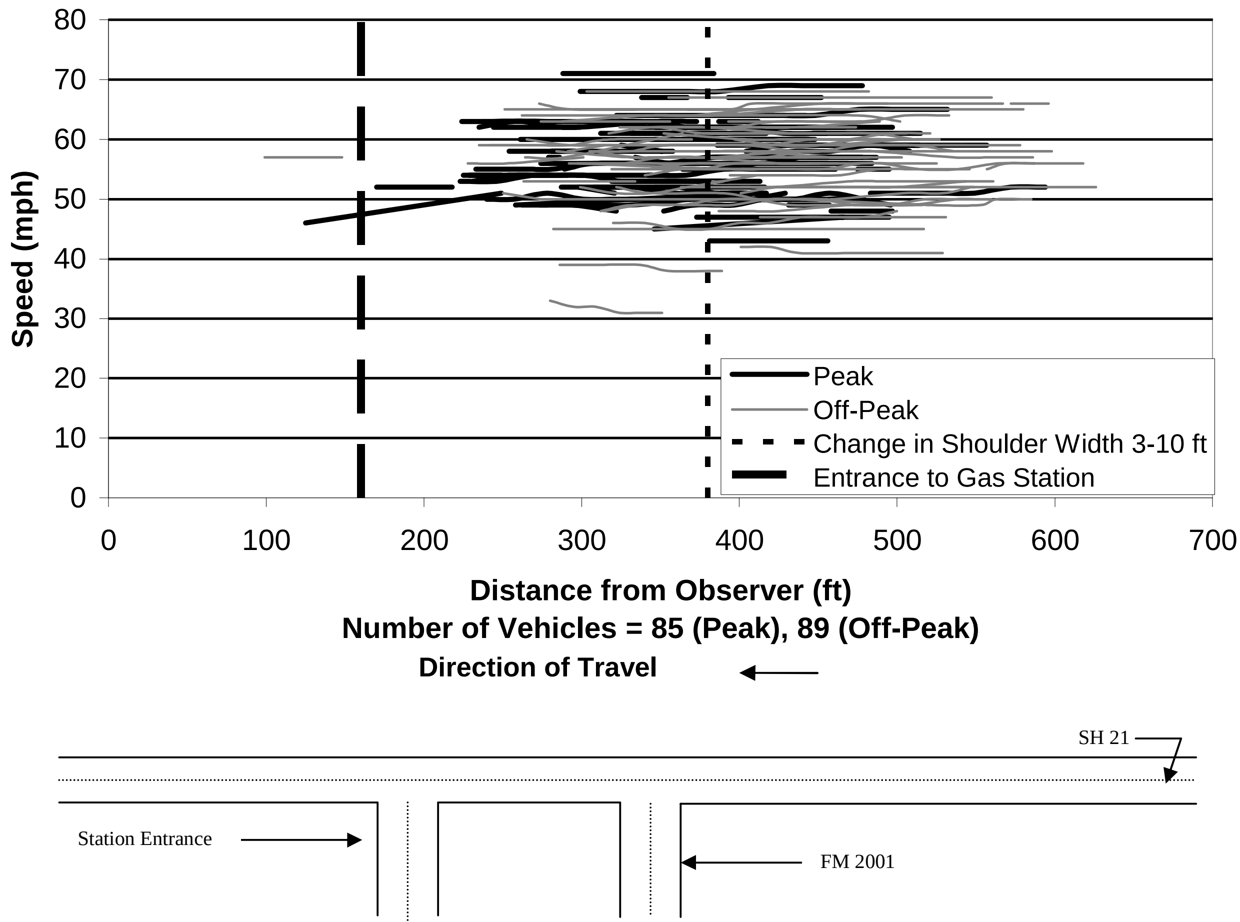
(1 mph = 1.61 km/h, 1 ft = 0.305 m)

Figure 3-27. Speed Profile for Left Turns at Site 5 (9/30 - 10/02/02).



(1 mph = 1.61 km/h, 1 ft = 0.305 m)

Figure 3-28. Speed Profile along RM 150 When School Zone is Active and Not Active.

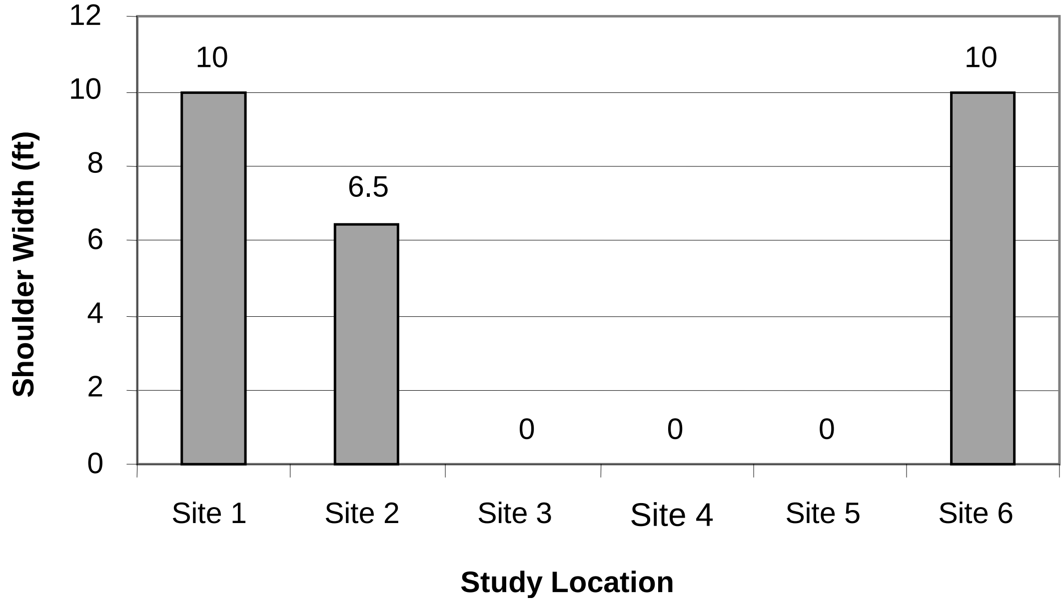


(1 mph = 1.61 km/h, 1 ft = 0.305 m)

Figure 3-29. Speed Profile for Straight Through Unimpeded (Peak and Off-Peak Period) at Site 6.

CONCLUSIONS

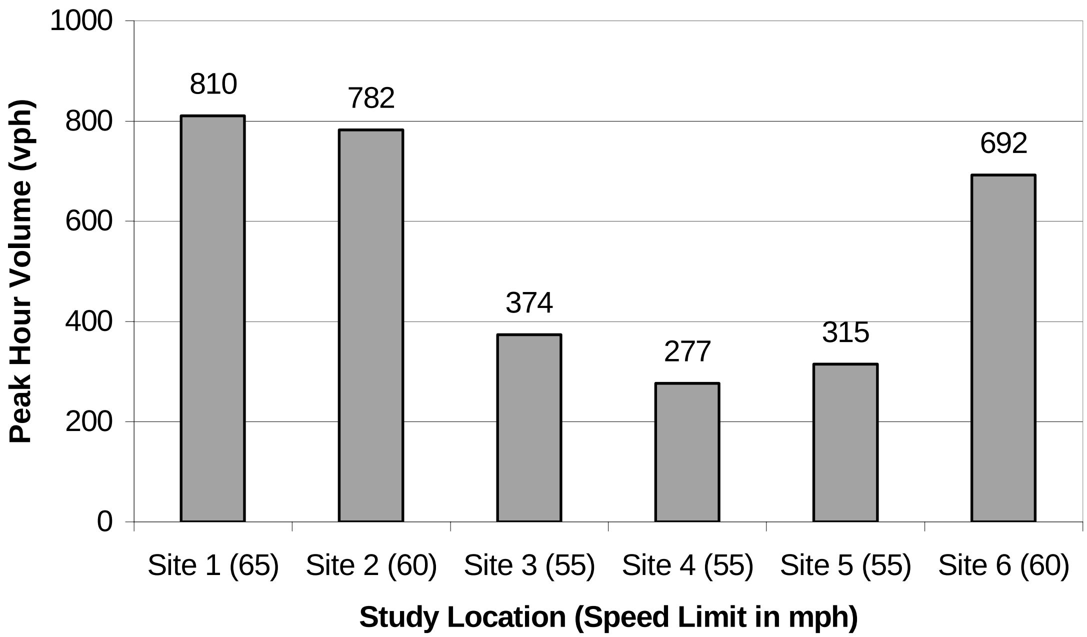
For each of the six sites, driver behavior at and approaching the site was observed. The sites had lane widths between 11 and 12 ft (3.4 and 3.7 m), shoulder widths that varied between 0 and 10 ft (0 and 3.1 m) (see Figure 3-31), and a level grade through the intersection. Peak hour volumes varied from 277 to 810 vph (see Figure 3-30) with posted speed limits ranging between 55 and 65 mph (89 and 105 km/h).



***Shoulder width widened from 3 to 10 ft (0.9 to 3.1 m) just prior to site.**

(1 ft = 0.305 m)

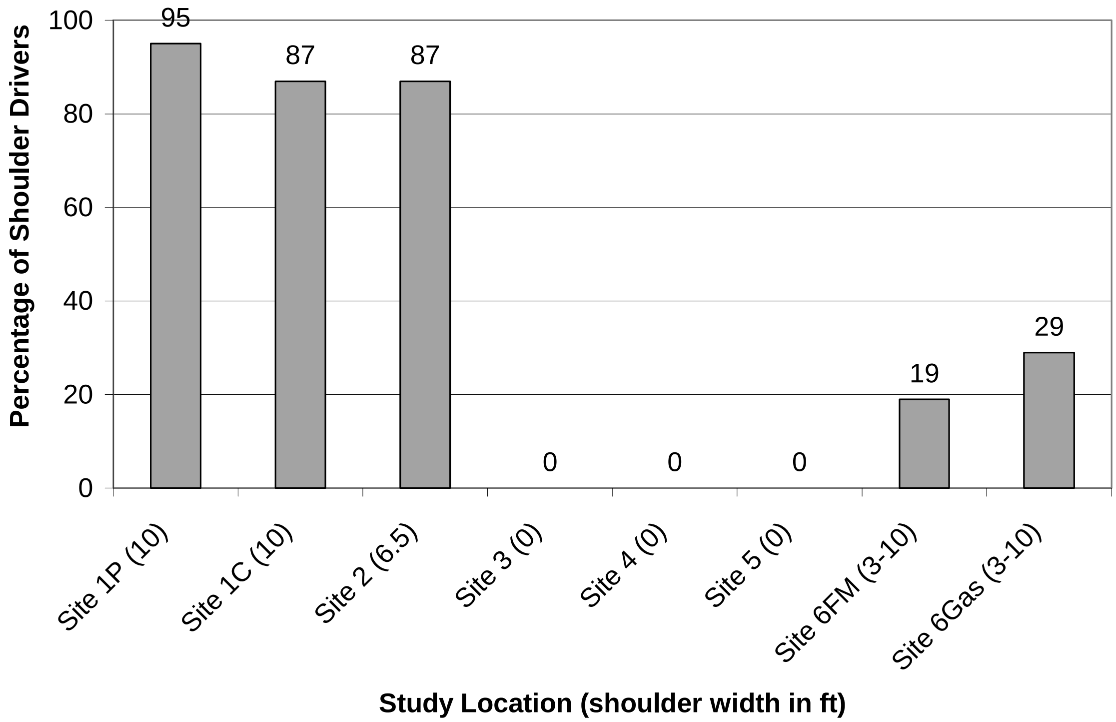
Figure 3-30. Differences in Shoulder Widths at Study Locations.



(1 mph = 1.61 km/h, 1 mi = 1.61 km)

Figure 3-31. Differences in Site Peak Hour Volumes.

The number of vehicles impeded by left-turning vehicles along with their reaction was recorded. The reaction was a function of the amount of shoulder width available to the driver. When a wide level shoulder was provided, a large percentage of the drivers, up to 95 percent, drove on the shoulder. At the site where the shoulder was retrofitted using available materials and widened from 3 to 10 ft (0.9 to 3.1 m) just prior to the intersection, only 19 to 29 percent of the drivers used the shoulder. At the site with minimum paved shoulder, none of the recorded drivers used the shoulder (although the number of drivers in this situation was low, on the order of 1 to 3 vph, compared to the other sites). Figure 3-32 shows the percentage of shoulder drivers at each site.

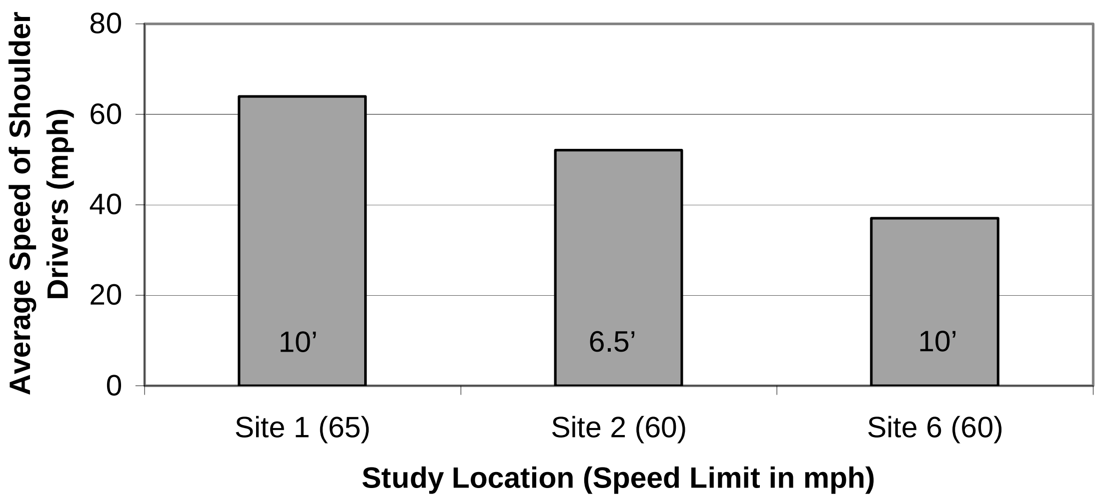


(1 ft = 0.305 m)

Figure 3-32. Percentages of Shoulder Drivers at Each Study Location.

Shoulder width and type also appears to influence the speeds at which the movements are performed. Previous tables presented the average speeds as well as other statistics for through and shoulder movements. At sites 1 and 2, higher speeds were recorded. The average 85th percentile speed for shoulder drivers at site 1 was 64 mph (103 km/h) with a range of speeds between 22 and 67 mph (35 and 108 km/h). At site 2, only one car was measured on the shoulder and its speed was 52 mph (84 km/h). At site 6, a lower speed was used by the one recorded shoulder driver: 37 mph (60 km/h). Figure 3-33 shows the comparison of the 85th percentile speeds of shoulder drivers.

Driving on the shoulder to bypass a left-turning vehicle is an efficient use of the roadway system in many areas. However, there are situations when this behavior generates concerns regarding the safety of those involved, for example, when other nearby driveways affect driver performance at the subject intersection. Multiple lane changes resulting from drivers reacting to entering and exiting vehicles creates complex interactions that are occurring at high speeds. At site 1, a high-volume driveway to a physician office is about 500 ft (153 m) upstream of the subject intersection. While this distance appears large, especially considering general recommendations for driveway spacing, the operations at the subject intersection (Copperfield Drive) were influenced by the behaviors at the Physicians Center driveway. Several cars moved onto the shoulder at the Physicians Center to bypass a left-turning vehicle and then returned to the through lane and slowed to make the left turn at the Copperfield intersection. In essence, drivers made two lane changes within a 500 ft distance at an initial speed of over 60 mph (97 km/h) ending with a left-turn maneuver where the driver had to judge the availability of gaps in the opposing traffic stream. Other vehicles entered the shoulder upstream of the Physicians Center driveway and continued on the shoulder until passing other left-turning vehicles at Copperfield Drive. In very few cases did a vehicle slow and wait behind a left-turning vehicle at either intersection at site 1.



(1 mph = 1.61 km/h, 1 ft = 0.305 m)

Figure 3-33. Comparison of Average Driving Speeds of Shoulder Drivers.

An observation from the behavior at site 1 is the need to consider how to handle left-turn treatments in a developing corridor. This consideration needs to factor the issue of operating speed into the decision. As a rural two-lane highway, the operating speeds are on the order of 60 to 70 mph (97 to 113 km/h). When fully developed, the operating speed for the corridor will probably be more on the order of 30 to 40 mph (48 to 64 km/h). If possible, the left-turning treatment selected should consider both conditions. Can the treatment used when the roadway is operating at 60 mph (97 km/h) be appropriate or easily changed to the treatment preferred for the 40 mph (64 km/h) condition?

More importantly for this situation, at what point should a left-turn treatment be considered for the developing two-lane highway. As a corridor develops and more driveways are added to the high-speed two-lane highway (and thus more left-turning traffic), the use of the shoulder as a bypass lane will also increase. This increases the likelihood for more lane changes as drivers attempt to avoid delay behind a left-turning vehicle and the likelihood for more conflicts as a greater number of turning movements are introduced into the corridor.

CHAPTER 4

LEFT-TURN DRIVER BEHAVIOR ON RURAL FOUR-LANE AND THREE-LANE HIGHWAYS

The anticipated conversion of a four-lane section to a three-lane section provided the unique opportunity to gather insight into driver behavior for these types of cross sections. The site under evaluation is an intersection of a major rural arterial with a rural collector. There are significant traffic volumes on the arterial, which makes turning left onto the collector difficult during certain periods of time. Conditions at the intersection were such that TxDOT decided to install a continuous two-way left-turn lane (TWLTL) along the section and a left-turn bay at the intersection to provide storage for left-turning vehicles.

STUDY LOCATION

The study site is the intersection of SH 71 and Pedernales Canyon Trail (PCT), which is highlighted in Figure 4-1.

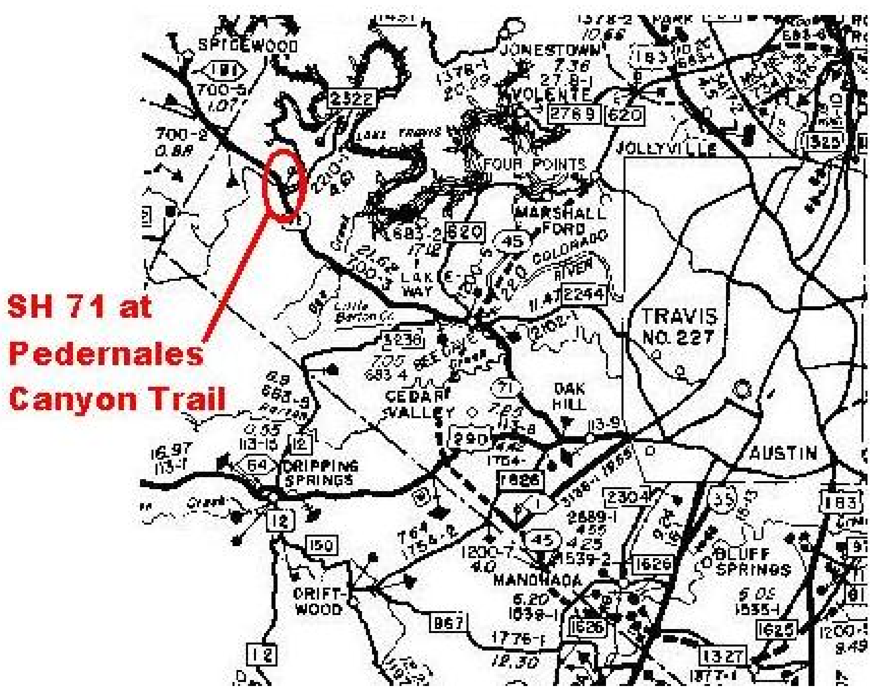


Figure 4-1. Location of Study Site.

This intersection is 0.6 mi (1 km) southeast of the intersection of SH 71 and FM 2322 and is approximately 20 mi (32 km) west of Austin. The intersection is in a depressed area of SH 71; there is a downgrade of approximately 3 percent on the westbound approach and a downgrade of approximately 2 percent on the eastbound approach. The posted speed limit on SH 71 at the study site is 65 mph (105 km/h).

DATA COLLECTION METHODOLOGY

Data were collected during July and August 2002 at this site by two methods: laser gun and video trailer. Laser guns collected speed profiles on selected vehicles traveling through the study site, while a video trailer recorded a visual record of all traffic at the study site. Laser and video data were collected in two periods, before and after the conversion to three lanes. The data collection periods are shown in Table 4-1 below. The times of day vary between the before and after periods because of technical problems with the equipment.

Table 4-1. Data Collection Periods.

Before Improvements				After Improvements			
July 18, 2002		July 19, 2002		August 27, 2002		August 28, 2002	
Laser	Video	Laser	Video	Laser	Video	Laser	Video
8:00-10:00 AM	8:00 AM – 12:00 PM	7:30-9:00 AM	12:00-9:00 AM	8:00-11:00 AM	8:00 AM – 3:00 PM	8:00 AM – 12:00 PM	
2:00-7:00 PM				4:00-7:00 PM		1:30-2:30 PM	

To collect data with the laser guns, two observers positioned themselves in a pickup truck off of the roadway on SH 71. The truck was parked 36 ft (11.0 m) from the edgeline and approximately 850 ft (259 ft) east of the intersection with PCT. Each observer had a laser gun, connected to a laptop computer, which recorded the speed and distance measurements of each reading taken by the laser gun. Observers collected speed profiles for 532 vehicles in the before period and 545 vehicles in the after period. Speed profiles were for a variety of vehicle types and movements in order to gain a sample of the traffic characteristics at the study site.

Video data were collected through the use of a trailer with a video camera mounted on an elevated telescoping arm, connected to a videocassette recorder. This trailer was also positioned off of the roadway on SH 71, approximately 40 ft (12.2 m) from the edgeline and 830 ft (253 m) east of the intersection with PCT. Observers periodically monitored the video trailer to confirm proper operation and to replace used videotapes.

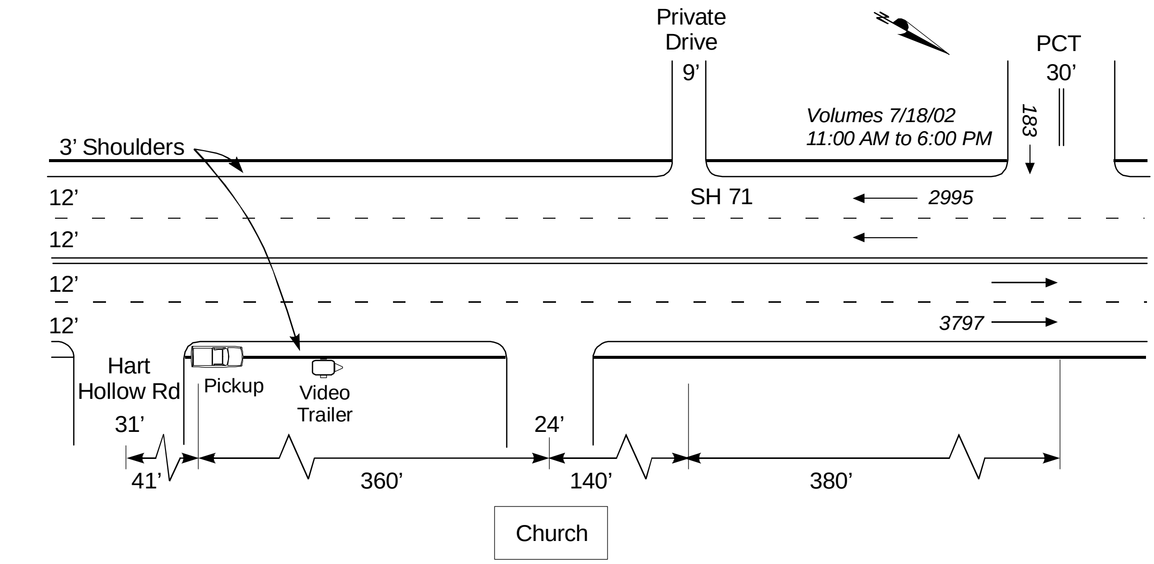
BEFORE AND AFTER COMPARISON

Site Condition

Prior to improvements, SH 71 consisted of four 12 ft lanes and 3 ft shoulders, for a total pavement width of 54 ft (16.5 m). PCT is a two-lane roadway with no shoulders and a total pavement width of 30 ft (9.2 m). Figure 4-2 shows a diagram of the study site area, including selected dimensions and distances. The numbers shown with arrows at the intersection at PCT are the number of entering vehicles for each leg of the intersection during a volume count from 11:00 AM to 6:00 PM on July 18, 2002.

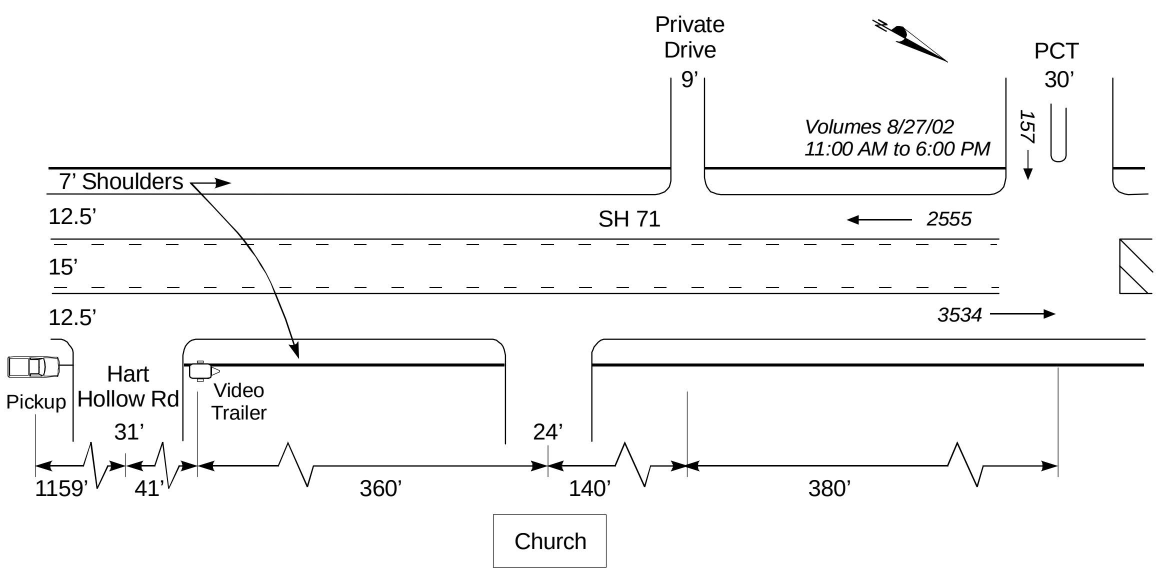
After improvements, SH 71 was re-striped to have two 12.5 ft (3.8 m) lanes with 7 ft (2.1 m) shoulders and a 15 ft (4.8 m) continuous TWLTL for a total pavement width remaining at 54 ft (16.5 m). The TWLTL begins at PCT and continues eastbound for approximately 1.5 mi

(2.4 km). West of PCT is the transition to the previously existing four-lane configuration. PCT was unchanged. Figure 4-3 shows a diagram of the study site area after improvements, including selected dimensions and distances. The numbers shown with arrows at the intersection at PCT are the number of entering vehicles for each leg of the intersection during a volume count from 11:00 AM to 6:00 PM on August 27, 2002.



(1 ft = 0.305 m)

Figure 4-2. Diagram of Before Conditions.



(1 ft = 0.305 m)

Figure 4-3. Diagram of After Conditions.

Figures 4-4 through 4-7 represent conditions before improvements were made. A changeable message sign was placed at the site to assist in informing drivers of the change. Figures 4-8 and 4-9 show the messages used on the signs. Figures 4-10 through 4-18 show conditions after SH 71 was re-striped.

Figure 4-4. Looking West toward PCT Intersection (about halfway into picture on left side).

Figure 4-5. Looking East (toward Austin) from the PCT Intersection.

Figure 4-6. PCT Looking toward SH 71.

Figure 4-7. PCT from SH 71.

Figure 4-8. Changeable Message Sign and Sign Showing Changes to Roadway.

Figure 4-9. Changeable Message Sign and Sign Showing Changes to Roadway.

Figure 4-10. Lane Reduction Sign, Looking East toward Austin.

Figure 4-11. Eastbound with PCT on the Right (second drive).

Figure 4-12. Looking EB as Vehicle Turns Left onto PCT Using TWLTL.

Figure 4-13. At PCT Intersection Facing EB.

Figure 4-14. Do Not Pass Sign, Looking East, Video Trailer on Left.

Figure 4-15. EB toward Austin Showing Grade.

Figure 4-16. Looking West, PCT on the Left (approx where vehicles are in picture).

Figure 4-17. Looking West toward PCT, from Church Parking Lot Entrance.

Figure 4-18. End of TWLTL at PCT.

Volumes

Tables 4-2 and 4-3 provide the traffic volumes for afternoon traffic at the intersection in the before and after periods.

Table 4-2. Volumes at Study Site in the Before Period.

	SH 71 WB Traffic		SH 71 EB Traffic		PCT Traffic	
Time Range (July 18, 2002)	Through	Left at PCT	Through	Right at PCT	Turn Left (EB)	Turn Right (WB)
11:00 AM-12:00 PM	332	11	400	7	4	28
12:00-1:00 PM	407	14	446	6	10	20
1:00-2:00 PM	473	25	405	10	13	15
2:00-3:00 PM	447	22	368	8	7	14
3:00-4:00 PM	552	16	434	10	7	11
4:00-5:00 PM	711	12	454	10	8	15
5:00-6:00 PM	766	39	431	6	15	16
TOTAL	**3688**	**139**	**2938**	**57**	**64**	**119**

Table 4-3. Volumes at Study Site in the After Period.

	SH 71 WB Traffic		SH 71 EB Traffic		PCT Traffic	
Time Range (Aug 27, 2002)	Through	Left at PCT	Through	Right at PCT	Turn Left (EB)	Turn Right (WB)
11:00 AM-12:00 PM	309	18	347	8	5	16
12:00-1:00 PM	346	11	340	8	8	16
1:00-2:00 PM	371	17	368	4	4	10
2:00-3:00 PM	406	21	330	7	10	23
3:00-4:00 PM	500	30	378	6	6	12
4:00-5:00 PM	689	29	343	6	6	13
5:00-6:00 PM	747	40	399	11	10	18
TOTAL	**3368**	**166**	**2505**	**50**	**49**	**108**

A comparison of Tables 4-2 and 4-3 reveals that volumes in the after period were slightly lower than those in the before period. However, the traffic patterns were similar, with westbound through traffic increasing steadily throughout the afternoon, eastbound traffic remaining steady, and approximately twice as many vehicles turning westbound from PCT as eastbound. In addition, it is important to note that even though volumes were generally lower in the after period by about 8.7 percent, there was nearly a 20 percent increase in the number of westbound vehicles turning left in the TWLTL configuration compared to the before period.

Maneuvers

The target vehicles in the collection of speed data were reviewed for the type and distribution of maneuvers. The possible maneuvers executed by the vehicles at the study site were categorized in two ways: by operations (free flow, impeded, and left turn) and by movement. Movement codes used to categorize the data are listed in Table 4-4.

Categorization by operational group was a much simpler method of looking at traffic patterns because of the small number of groups. The observer collecting the speed profile data assigned the vehicle's group label while in the field, based on whether the vehicle turned at the intersection, went straight through the intersection under free-flow conditions, or was impeded. The distribution of maneuvers by operational groups is shown in Table 4-5; this is the distribution for all target vehicles whose speed profiles were recorded during speed data collection.

Table 4-4. Movement Codes for Vehicles at the Study Site.

Action Code Before	*Action Comment*	*Action Code After*
101	Straight through	101
102	Left turn at PCT	102
103	Right turn at church	103
104	Left turn at private drive	104
105	U-turn	N/A
106	Right turn out of church	N/A
107	Left turn out of private drive	N/A
Impeded Code Before	*Action Comment*	*Impeded Code After*
201	Not impeded	200
202	Impeded by EB traffic	202
203	Queued	203
204	Impeded by car turning right at church	201
N/A	Car from EB was in TWLTL and subject vehicle could not use TWLTL at PCT	204
205	Impeded by slower WB through traffic	205
206	Impeded by slower WB traffic turning left	206
Free-Flow Code Before	*Action Comment*	*Free-Flow Code After*
301	Left lane to right lane to avoid turning car	N/A
302	Right lane to left lane to avoid turning car	N/A
303	Right lane to left lane (not avoidance)	N/A
304	Left lane to right lane (not avoidance)	N/A
305	Changed lanes to avoid turning car, then changed back	N/A

Table 4-5. Distribution of Maneuvers by Operational Group.

Group	Number Before	Number After	Difference
Free Flow	290	343	53
Impeded	111	129	18
Left Turn	131	74	-57
TOTAL	**532**	**546**	**14**

Categorization by movement code was more complex, since there were multiple combinations of codes possible for each vehicle. A movement code is made up of two parts, a 100-level action code, along with a supplementary 200- or 300-level free-flow or impeded code. The movement code was assigned to each vehicle by reviewing the movements of each target vehicle on the video. In this process, the other nearby intersections and driveways were also considered, as well as lane-change maneuvers in the before period. Thus, this categorization of vehicles was more precise than the operational groupings, which only took into account the movements at the study site. Table 4-6 provides the distribution of maneuvers by movement code; codes that appear on the same line in both periods are comparable.

Table 4-6. Distribution of Maneuvers by Movement Code.

Before Period		After Period	
Movement Code	Number	Movement Code	Number
101-201	284	101-200	339
101-205	37	101-205	126
101-206	8	101-206	2
101-301	12		
101-303	16		
101-304	9		
101-305	24		
102-201	77	102-200	32
102-202	46	102-202	38
102-203	3	102-203	2
104-202	2	104-202	2
107-202	1		
None assigned	13	None assigned	5
TOTAL	**532**	**TOTAL**	**546**

In viewing the values presented in Tables 4-5 and 4-6, one can see that while the overall number of vehicles in each period was similar, the movement patterns were somewhat different. In Table 4-5, there were 53 more vehicles assigned to the free-flow group, 18 more in the impeded group, and 57 fewer vehicles in the left-turn group in the after period than the before period. Even though there were fewer left turns, there were far more through vehicles that proceeded through the intersection largely unimpeded, based on the observation of the observers collecting the speed profile data.

A review of Table 4-6 provides a more thorough breakdown of the subject vehicles and their movements. In both periods, approximately the same number and percentage of total vehicles passed through the intersection unimpeded in the same lane (code 101-201 before or 101-200 after). Also, similar to Table 4-5, the drop in left-turning vehicles from the before period to the after period is again evident, with 125 before vehicles with 102 and 104 codes and 74 vehicles in the after period.

The most significant change in vehicle movements is shown in the remaining codes in Table 4-6, that is, the codes that are exclusive to only one period. In the before period, there were another 50 vehicles (with 101-30X codes) that passed through the intersection but changed lanes. In the after period, there were 128 vehicles (codes 101-205 and 101-206) that passed through the intersection but were somewhat impeded by the traffic in front of them. This change in movements makes sense, as there is no passing lane in the after period that drivers could use to pass slower or turning vehicles.

Speed Characteristics

There are a variety of metrics available on which to base the speed characteristics at this site. Distance, time of day, and maneuver are all valid qualifiers for describing the profiles and patterns that exist. The broadest description is a comparison of speed to distance, classified by maneuver. Several graphs of speeds vs. distance from both periods for the most common maneuvers along with graphs of the deceleration profiles for left-turning vehicles were created. Tables 4-7 and 4-8 provide key speed statistics for regular intervals relative to the intersection at PCT for selected through maneuvers.

Table 4-7. Speed Statistics at Regular Intervals for Free-Flowing Through Vehicles.

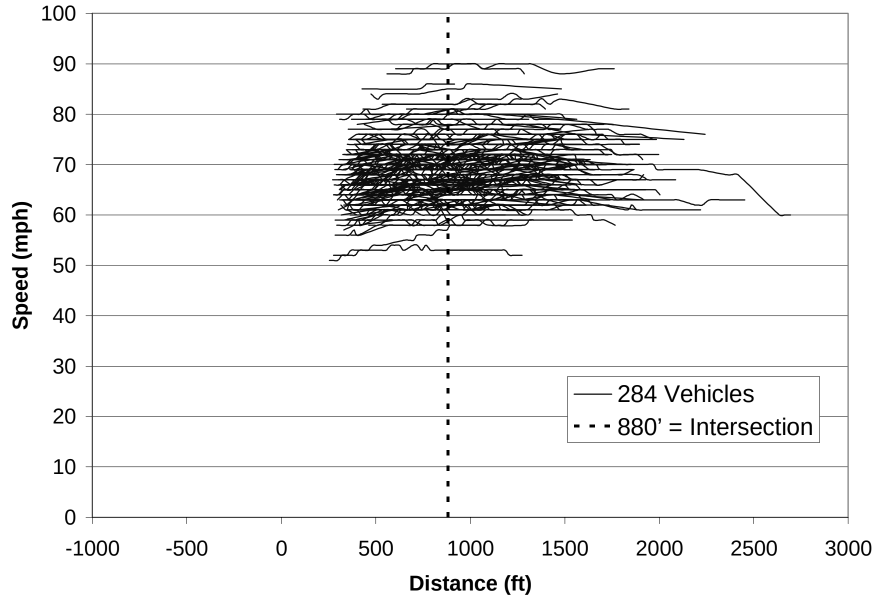
Distance (ft) from Intersection	# Readings		Mean Speed (mph)		85th Percentile Speed (mph)		Standard Deviation (mph)	
	Before	After	Before	After	Before	After	Before	After
-500	260	365	66.2	63.5	71	69	5.13	5.50
-250	428	281	68.4	64.4	73	70	5.40	5.85
0	394	181	68.9	64.1	73	69	5.33	5.49
+250	301	55	69.1	65.6	74	71	5.48	5.07
+500	172	4	68.9	65.0	74	67	5.07	4.00
1 ft = 0.305 m, 1 mph = 1.61 km/h								

Table 4-8. Speed Statistics at Regular Intervals for Impeded Through Vehicles.

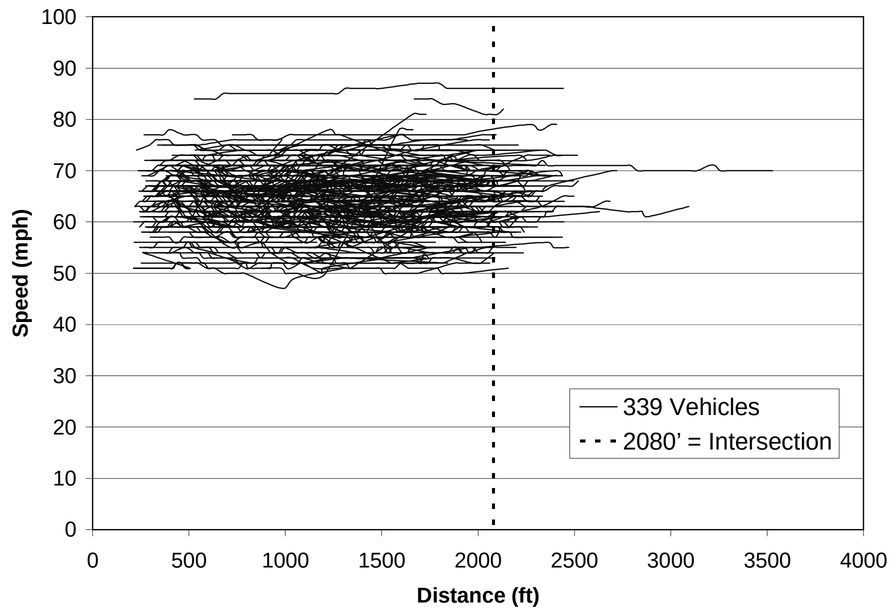
Distance (ft) from Intersection	# Readings		Mean Speed (mph)		85th Percentile Speed (mph)		Standard Deviation (mph)	
	Before	After	Before	After	Before	After	Before	After
-500	10	147	60.1	60.5	64	65	4.43	5.65
-250	48	107	63.2	62.0	68	67	6.51	5.71
0	52	54	63.7	63.6	70	69	6.42	4.70
+250	28	8	65.8	65.4	70	69	5.98	5.83
+500	4	0	67.3	N/A	75	N/A	9.43	N/A
1 ft = 0.305 m, 1 mph = 1.61 km/h								

Figure 4-19 reinforces the findings from Table 4-7. The after mean speed was approximately 3 to 4 mph (4.8 to 6.4 km/h) lower than the before mean speed for most of the section, as was the 85th percentile speed. A review of the profiles in Figures 4-19a and 4-19b indicates that the primary band of speed profiles for unimpeded through vehicles was lower in the after period than the before period. The primary band of speeds in the before period was from 60 to 80 mph (97 to 129 km/h). The band in the after period was from 50 to 75 mph (81 to 121 km/h). In the before period, the 50th and 85th percentile speeds were 68 and 73 mph (109 and 118 km/h), respectively. In the after period, the 50th and 85th percentile speeds were 64 and 70 mph (103 and 113), respectively. Considering that the posted speed limit on this section of the roadway is 65 mph (105 km/h), this reduction in speed adjusts the common driving behavior closer to the legal limit.

Figure 4-20 confirms the findings from Table 4-8. The number of speed readings is much higher in the after period, but the readings themselves are very similar between the two periods, generally within 1 mph (1.6 km/h).



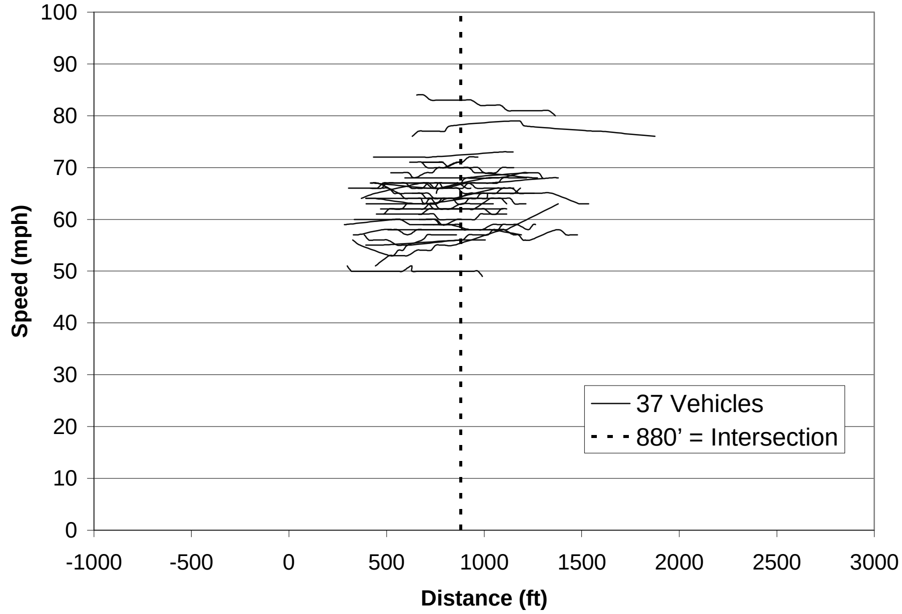
4-19a. Speed Profiles for Before Code 101-201.



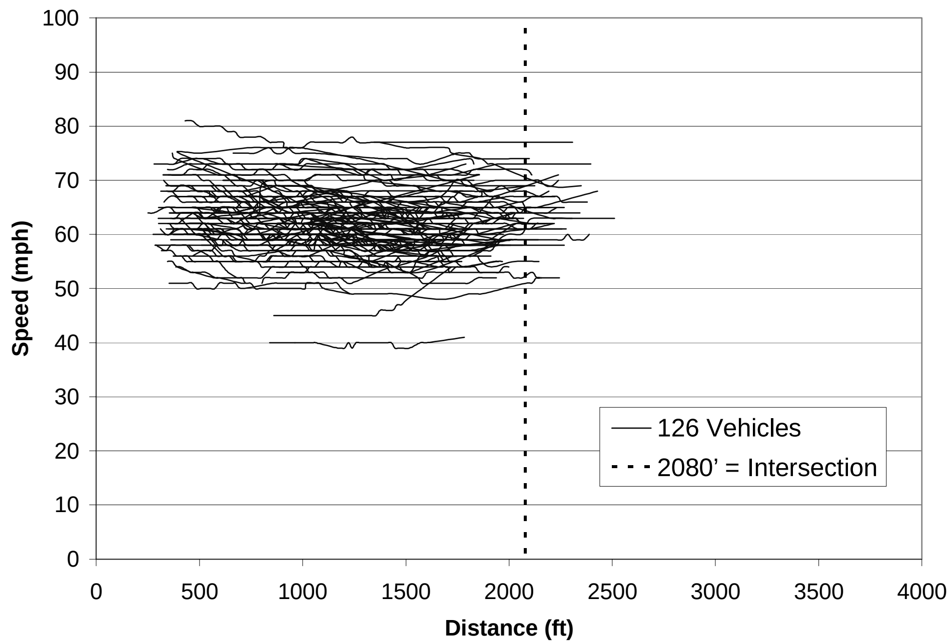
4-19b. Speed Profiles for After Code 101-200.

(1 mph = 1.61 km/h, 1 ft = 0.305 m)

Figure 4-19. Comparison of Speed Profiles for Unimpeded Westbound Through Vehicles.



4-20a. Speed Profiles for Before Code 101-205.



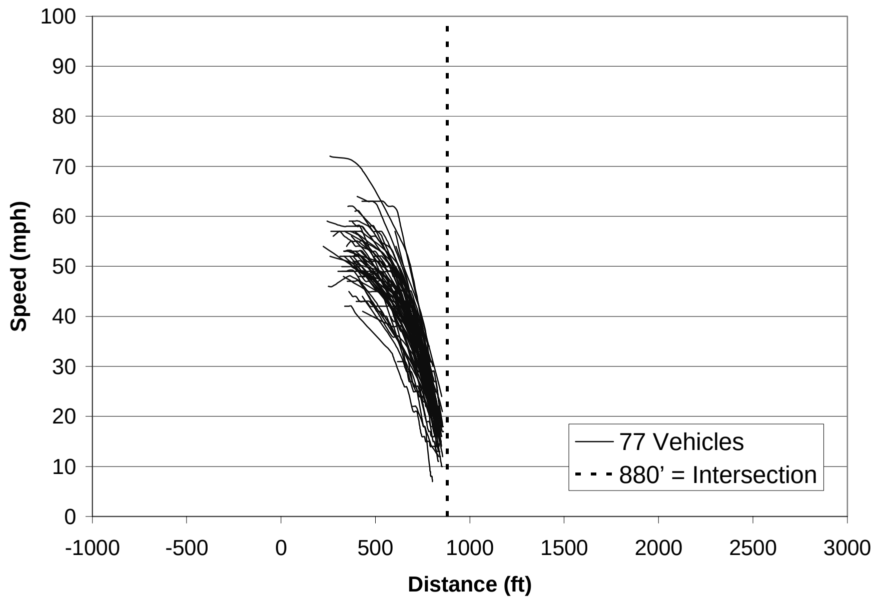
4-20b. Speed Profiles for After Code 101-205.

(1 mph = 1.61 km/h, 1 ft = 0.305 m)

Figure 4-20. Comparison of Speed Profiles for Westbound Through Vehicles Impeded by Slower Westbound Through Traffic.

With the removal of a through lane in each direction, the number of impeded through vehicles increased greatly. Because the option of passing a slower vehicle was no longer available, drivers of faster vehicles slowed down through the three-lane section. Despite the increase in the number of impeded vehicles, their behavior appears largely unchanged from the before period to the after period. Most vehicles traveled at speeds between 60 and 70 mph (97 and 113 km/h) in both periods, with a lesser number between 50 and 60 mph (80 and 97 km/h) making up the bulk of the remaining traffic. However, the vehicles on the extremes of the profiles in Figure 4-20 have lower speeds in the after period than the before period, with no vehicles sustaining speeds above 80 mph (129 km/h), and two vehicles below 50 mph (80 km/h).

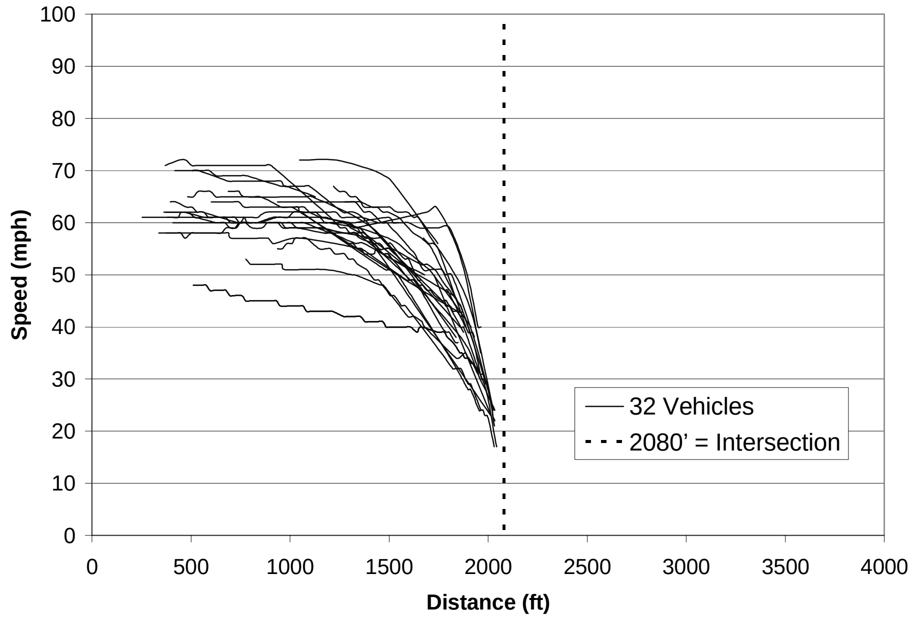
There were about half as many vehicles for unimpeded left-turning vehicles in the after period as in the before period (32 to 76). However, even with the fewer number of vehicles, a comparison of profiles in Figures 4-21a and 4-21b shows that unimpeded left-turning vehicles decelerated at a slower, more gradual rate in the after period than in the before period. Vehicles in the before period had a much higher rate of deceleration, and their speed profiles decreased more sharply than in the after period. Vehicles in the before period also decelerated in a tighter band than did vehicles in the after period.



4-21a. Speed Profiles for Before Code 102-201.

(1 mph = 1.61 km/h, 1 ft = 0.305 m)

Figure 4-21. Comparison of Speed Profiles for Unimpeded Westbound Left-Turning Vehicles.



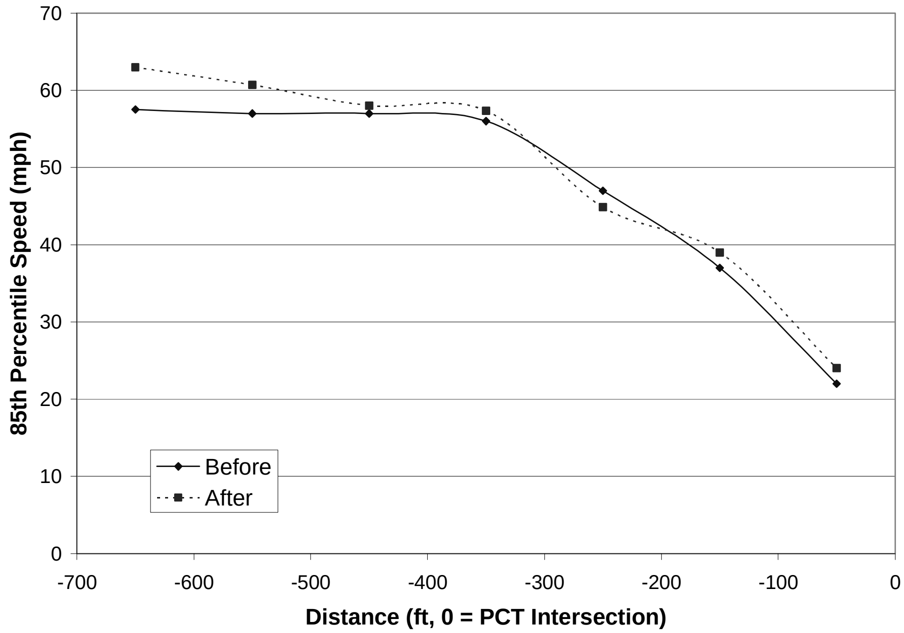
4-21b. Speed Profiles for After Code 102-200.

(1 mph = 1.61 km/h, 1 ft = 0.305 m)

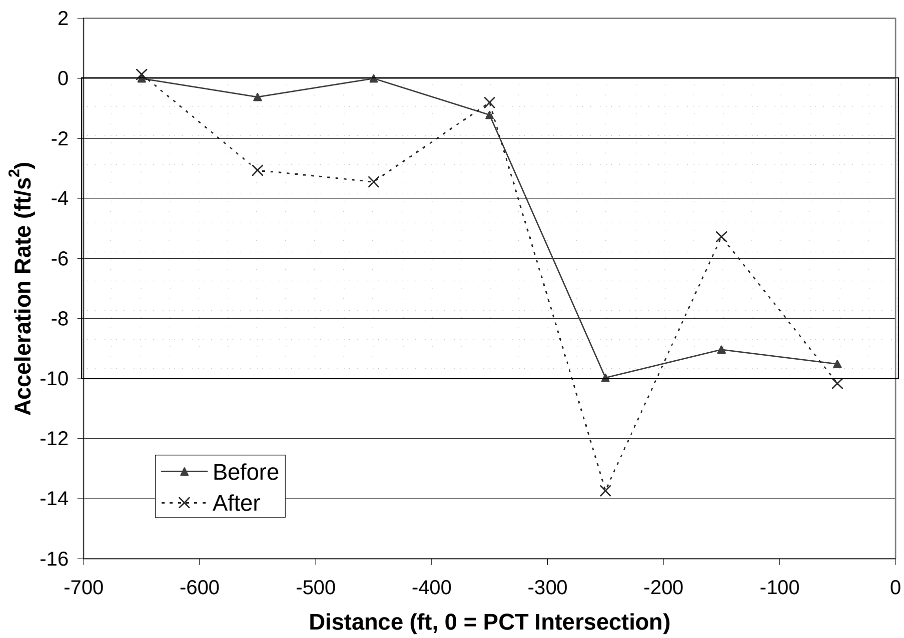
Figure 4-21. Comparison of Speed Profiles for Unimpeded Westbound Left-Turning Vehicles (continued).

Figure 4-22 shows the changes in 85th percentile speeds and corresponding deceleration rates (shown as negative acceleration) at 100 ft (30.5) increments approaching the intersection at PCT for left-turning vehicles. In general, vehicles in the after period had a slightly higher speed throughout the profile. One similarity between the two periods is that the deceleration appears to begin in earnest at about 300 ft (92 m) prior to the intersection, based on the downward spike of the deceleration profiles. This is consistent with the speed profiles shown in Figure 4-21. Even though the speed profiles are similar in shape, before vehicles decelerated at a more constant rate, with only one major change between -350 and -250 ft (-107 and -76 m).

The deceleration of after vehicles was more pronounced in this area, nearly -14 ft/s^2 (-4.3 m/s^2) This is the only point less than -10 ft/s^2 (-3.1 m/s^2) which is the value that the Institute of Transportation Engineers (ITE) *Traffic Engineering Handbook* (*28*) describes as the limit of "reasonably comfortable" deceleration. This range of "reasonably comfortable" deceleration is shaded in Figure 4-22b. While this appears to be a large departure from the remainder of the profile, the deceleration rate corresponds to a decrease in 85th percentile speed of 12.5 mph (57.4 to 44.9 mph) [20.1 km/h (92.4 to 72.3 km/h] in the 100 ft (30.5 m) interval. This compares to a rate of -9.9 ft/s^2 (-3.0 m/s^2) and a decrease of 9.0 mph (56.0 to 47.0 mph) [14.5 km/h (90.2 to 75.7 km/h)] in the before period over the same interval. Thus, while the after deceleration rate appears to be somewhat extreme as shown in Figure 4-22b, the actual magnitude of the speed change was similar.



4-22a. Speed Change Profiles.

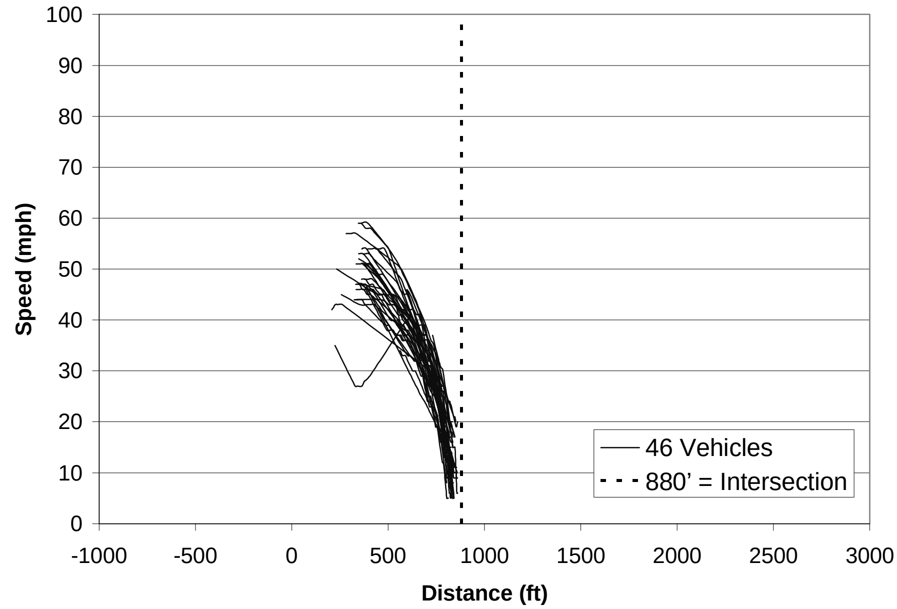


4-22b. Deceleration Change Profiles.

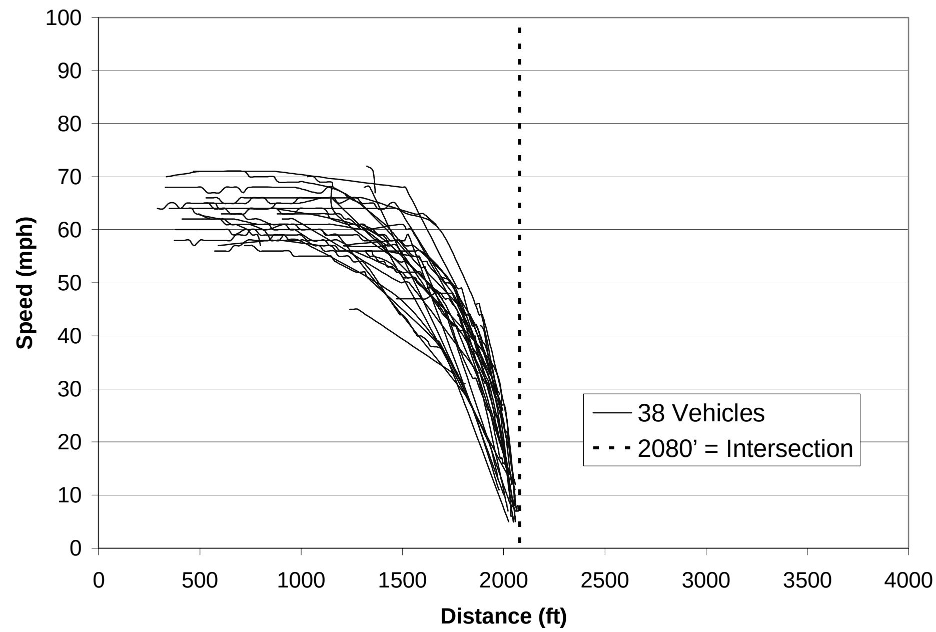
(1 mph = 1.61 km/h, 1 ft = 0.305 m, 1ft/s^2 = 0.05 m/s^2)

Figure 4-22. Comparison of Deceleration Profiles for Unimpeded Westbound Left-Turning Vehicles.

Figure 4-23 shows speed profiles similar to Figure 4-21 but for impeded left-turning vehicles. Similar to Figure 4-21, the vehicles in the before period decelerated in a tighter band than in the after period.



4-23a. Speed Profiles for Before Code 102-202.



4-23b. Speed Profiles for After Code 102-202.

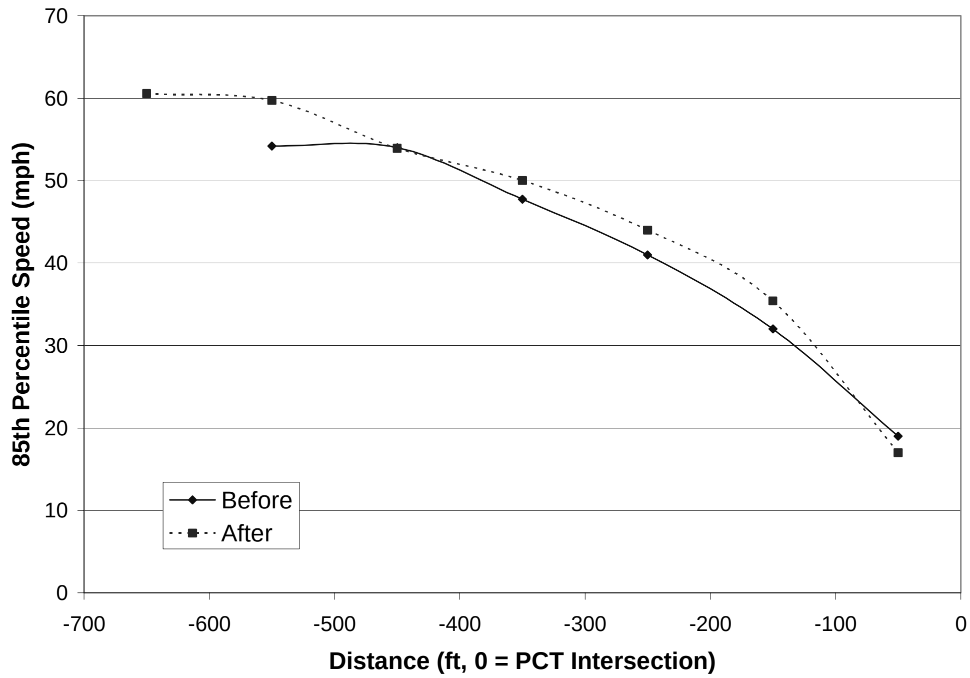
(1 mph = 1.61 km/h, 1 ft = 0.305 m, 1ft/s^2 = 0.05 m/s^2)

Figure 4-23. Comparison of Speed Profiles for Westbound Left-Turning Vehicles Impeded by Eastbound Traffic.

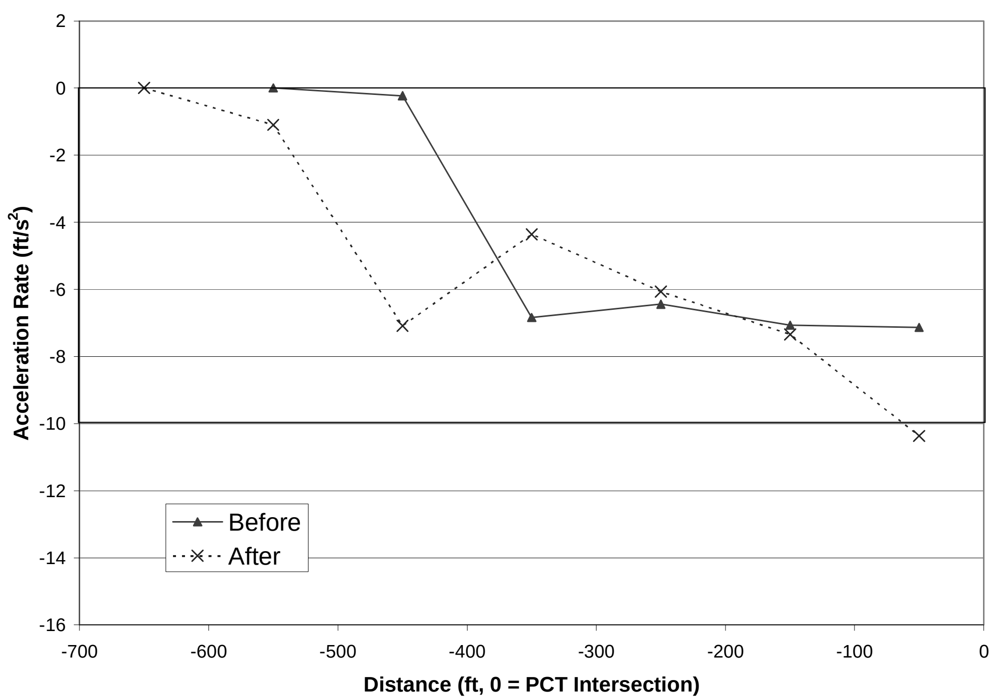
Figure 4-24 illustrates profiles for impeded left-turning vehicles, similar to Figure 4-22. In the before period, the vehicles began their profile at a lower speed than the after period and somewhat lower than unimpeded vehicles in the before period. As with Figure 4-22a, the after vehicles have a slightly higher 85th percentile speed for almost the entire length of the profile. Before vehicles in Figure 4-24 began to decelerate earlier and at a more constant rate, compared to Figure 4-22. In fact, the shape of the before deceleration curves in Figures 4-22b and 4-24b are almost identical, except that there is a shift of -100 ft (-31 m) in Figure 4-24b and the magnitude of the downward spike in Figure 4-24b is not as great. As in Figure 4-22b, the after profile appears more erratic, but when viewed in conjunction with the speed change profiles, the changes in deceleration are not as dramatic.

CONCLUSIONS

The reconfiguration of this segment of SH 71 to install a two-way left-turn lane at the intersection with Pedernales Canyon Trail reduces the potential for certain types of crashes involving left-turning vehicles by providing storage for those vehicles outside of the main through travel lanes. This allows left-turning vehicles to enter the TWLTL and decelerate at a more gradual rate, which also allows drivers to focus on potential conflicts with oncoming traffic rather than collisions with faster through traffic that may be approaching behind them. Furthermore, the reduction in the number of through lanes has affected the speed of the through vehicles. The number of impeded through vehicles increased noticeably after the improvement, and the overall speed of through vehicles decreased. While many through vehicles still travel at speeds higher than the posted speed limit of 65 mph (105 km/h), the number of vehicles exceeding that speed by more than 5 mph (8 km/h) has decreased considerably. This added effect may also improve the safety of this segment of roadway by reducing the differences in speed between vehicles.



4-24a. Speed Change Profiles.



4-24b. Deceleration Change Profiles.

(1 mph = 1.61 km/h, 1 ft = 0.305 m, $1 ft/s^2 = 0.05 m/s^2$)

Figure 4-24. Comparison of Deceleration Profiles for Westbound Left-Turning Vehicles Impeded by Eastbound Traffic.

CHAPTER 5

LEFT-TURN LANE INSTALLATION GUIDELINES

The addition of a left-turn lane can improve the operations and safety at an intersection as discussed in Chapter 2 of this report. Guidelines regarding when to include a left-turn in intersection design are plentiful. Some are based on minimizing conflicts in terms of the occurrence of a through vehicle arriving behind a turning vehicle, others are based on decreasing the amount of delay to through vehicles, and some are based on consideration of safety. Because of the quantity of methods, it is difficult to determine which method to use. For example, are certain techniques better for a rural versus an urban setting? Do the evaluations differ for number of lanes and for type of intersection? This chapter will review eight selected techniques and a number of criteria presented in state manuals. Some of the assumptions used in the techniques will be reviewed, and suggestions on changes to selected guidelines will be made. Figure 5-1 is an example of how a turning vehicle can affect traffic by creating queues and causing a vehicle to pass on the shoulder.

Figure 5-1. Example of a Left-Turning Vehicle.

Several of the techniques use common terms. Figure 5-2 graphically shows the following movements that are used to determine the need for a left-turn lane in several of the guidelines:

- Advancing Volume (V_A) – the total peak hourly volume of traffic on the major road approaching the intersection from the same direction as the left-turn movement under consideration.
- Left-Turn Volume (V_L) – the portion of the advancing volume that turn left at the intersection.
- Percent Left Turns (P_L) – the percentage of the advancing volume that turns left; equal to the left-turn volume divided by the advancing volume ($P_L = V_L \div V_A$).

- Straight Through Volume (V_S) – the portion of the advancing volume that travels straight through the intersection ($V_L + V_S = V_A$).
- Opposing Volume (V_O) – the total peak hourly volume of vehicles opposing the advancing volume.

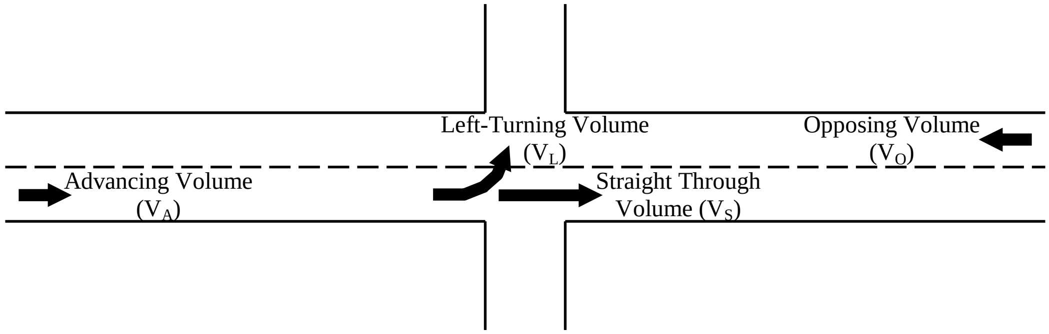


Figure 5-2. Volumes for Use in Left-Turn Lane Warrant Methods.

GUIDELINES REVIEWED

Researchers performed a review of the literature on many sources, including research reports, state and federal design manuals, and handbooks. Although many techniques are currently in use by various organizations to determine the need for left-turn lanes, several are either very similar or identical. Details are provided below on those methods that appeared to have distinctive results.

Harmelink

The oldest research found on evaluating the need for left-turn lanes at unsignalized intersections was that of M.D. Harmelink *(29)* in a paper that was published in 1967. His research provided the foundation for many current left-turn guidelines. Harmelink based his work on a queuing model in which arrival and service rates are assumed to follow negative exponential distributions. He states that the probability of a through vehicle arriving behind a stopped, left-turning vehicle should not exceed 0.02 for 40 mph (64 km/h), 0.015 for 50 mph (80 km/h), or 0.01 for 60 mph (96 km/h). He presented his criteria in the form of 18 graphs. To use his graphs, the advancing volume, opposing volume, operating speed, and left-turn percentage need to be known. Graphs for speeds of 40, 50, and 60 mph (64, 80, and 96 km/h) were given, as well as 5, 10, 15, 20, 30, and 40 percent left-turn volumes. An example graph of Harmelink's criteria for determining the need for left-turn lanes is shown in Figure 5-3.

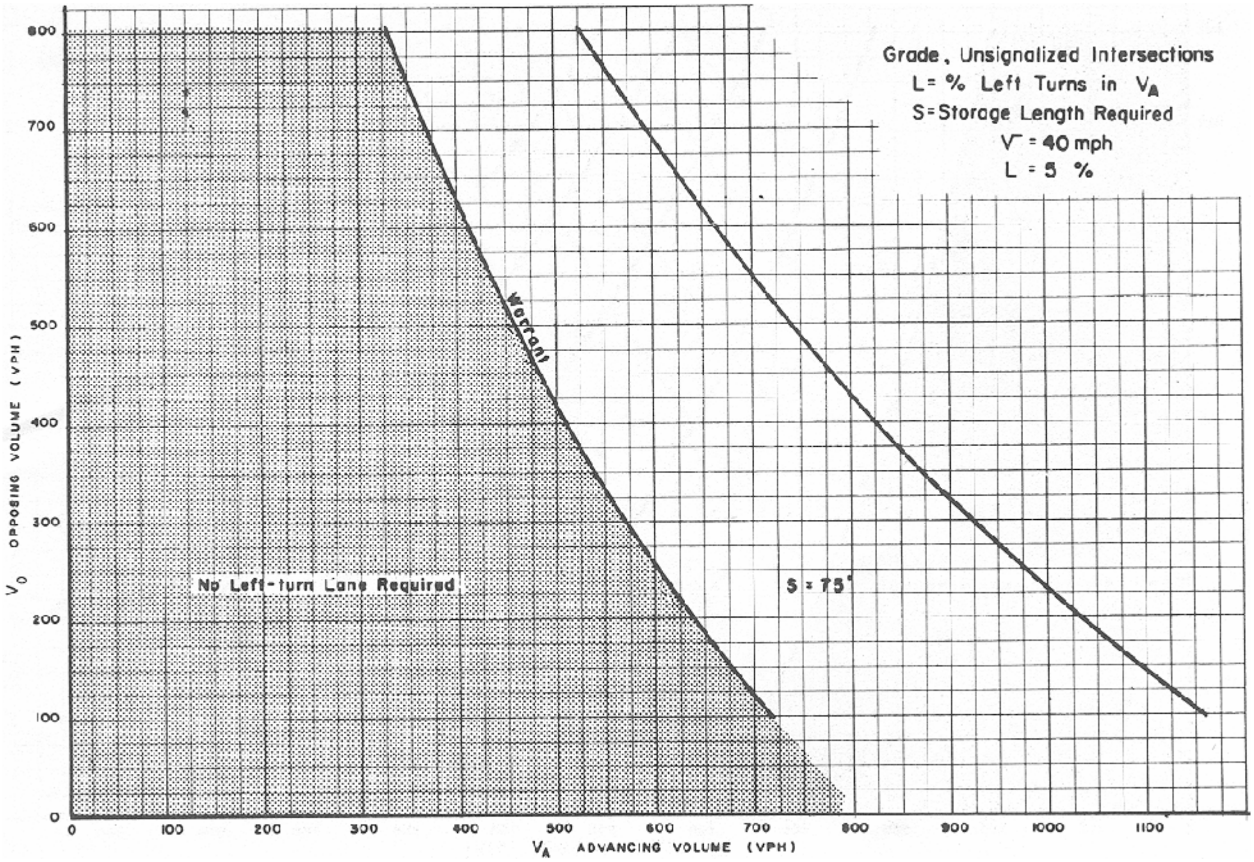


Figure 5-3. Harmelink – Left-Turn Lane Warrant Graph,
40 mph (64 km/h), 5% Left Turns, 1967 (*29*).

AASHTO

AASHTO's *Green Book* (*30*) and the TxDOT *Roadway Design Manual* (*31*) each contain a table for use in determining the need for a left-turn lane on two-lane highways (see Table 5-1). Similar tables were also present in the 1984 (*32*), 1990 (*33*), and 1994 (*34*) editions of the *Green Book*. The values in the table are based upon Harmelink's work.

Table 5-1. AASHTO *Green Book* Exhibit 9-75 and TxDOT *Roadway Design Manual* Table 3-11: Guide for Left-Turn Lanes on Two-Lane Highways (*30, 31*).

Opposing Volume (vph)	Advancing Volume (vph)			
	5% Left Turns	**10% Left Turns**	**20% Left Turns**	**30% Left Turns**
40 mph (60 km/h) operating speed				
800	330	240	180	160
600	410	305	225	200
400	510	380	275	245
200	640	470	350	305
100	720	515	390	340
50 mph (80 km/h) operating speed				
800	280	210	165	135
600	350	260	195	170
400	430	320	240	210
200	550	400	300	270
100	615	445	335	295
60 mph (100 km/h) operating speed				
800	230	170	125	115
600	290	210	160	140
400	365	270	200	175
200	450	330	250	215
100	505	370	275	240

NCHRP Report 279

In 1985, the Transportation Research Board published NCHRP Report 279, *Intersection Channelization Design Guide* (35). In that report, data from Harmelink's work were used to establish guidelines for determining the need for a left-turn lane. The guide provides the following advice for unsignalized intersections:

- Left-turn lanes should be considered at all median cross-overs on divided, high-speed highways.
- Left-turn lanes should be provided at all unstopped (i.e., through) approaches of primary, high-speed rural highway intersections with other arterials or collectors.
- Left-turn lanes are recommended at approaches to intersections for which the combination of through, left, and opposing volumes exceeds the warrants shown in Figure 5-4.
- Left-turn lanes on stopped or secondary approaches should be provided based on analysis of the capacity and operations of the unsignalized intersection. Considerations include minimizing delays to right turning or through vehicles and total approach capacity.

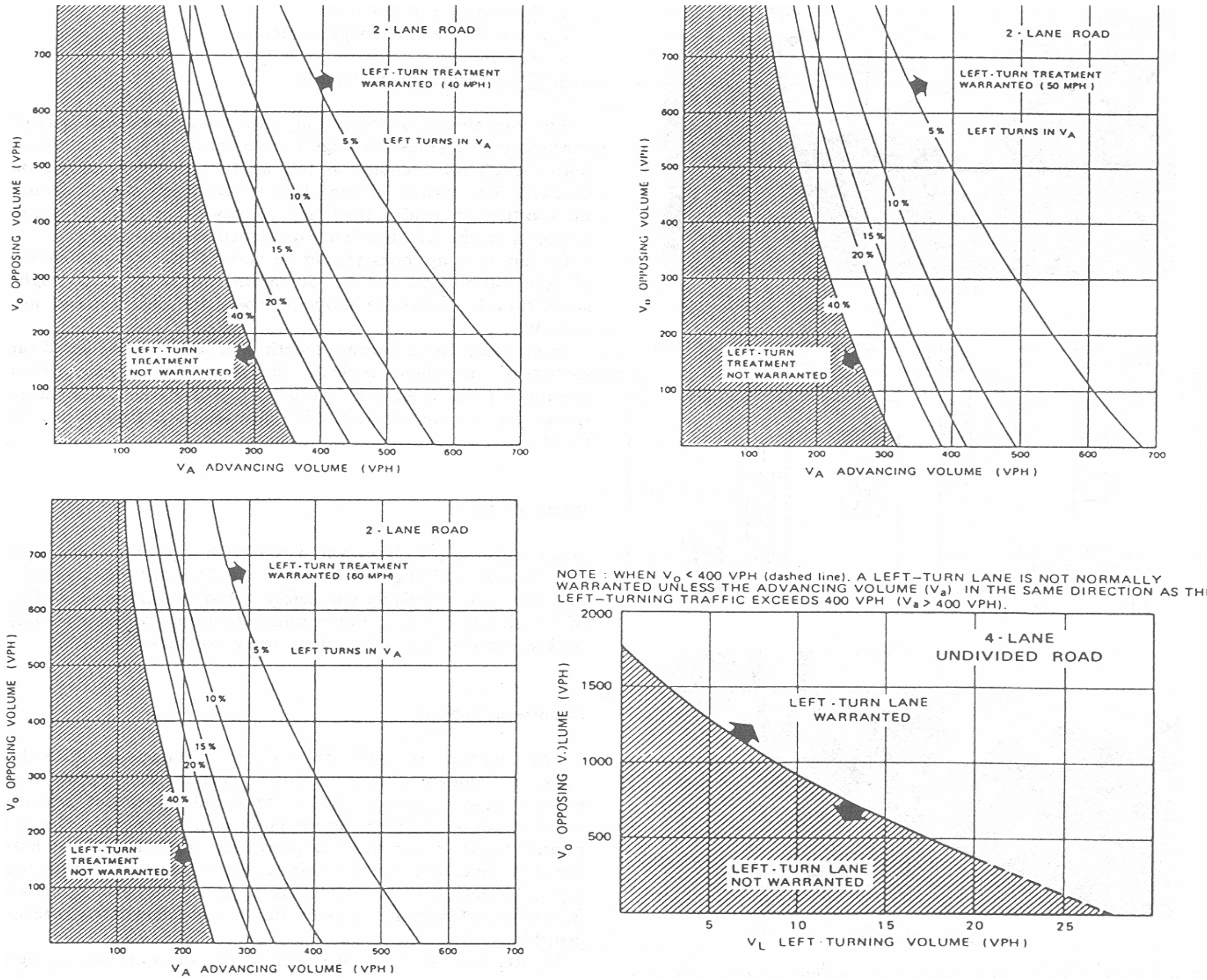


Figure 5-4. NCHRP 279 – Left-Turn Lane Guidelines, 1985 (*35*).
Oppenlander and Bianchi (ITE Technical Committee)

In the 1980s, ITE Technical Committee 4A-22 undertook the task of developing criteria for the provision of separate left-turn lanes at unsignalized and signalized intersections (*36*). The work performed by ITE Committee 4A-22 expanded the Harmelink model to include additional speeds (30 and 70 mph [48 and 113 km/h] roadways) and to include additional left-turn percentages. An example of one of the guideline graphs produced is shown in Figure 5-5.

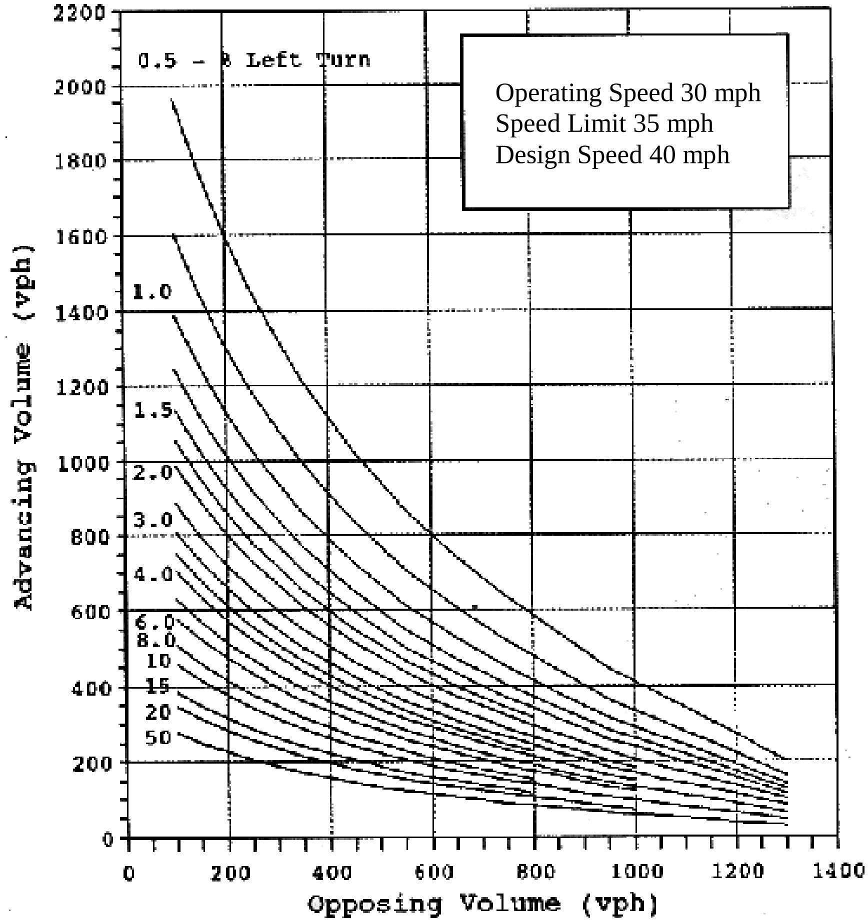


Figure 5-5. Oppenlander and Bianchi – Left-Turn Lane Guidelines; Unsignalized, Two-Lane, 30 mph (48 km/h) Operating Speed, 1990 (*36*).

NCHRP Report 348

Koepke provided two methods for determining the need for left-turn lanes in NCHRP Report 348 (*37*). The first method is shown in Figure 5-6 and Figure 5-7; however, Koepke states that in most cases, left-turn lanes should be provided where there are more than 12 left turns per peak hour. The second method presents the values included in the *Green Book* for determining whether a left-turn lane should be provided. They also stated that "left-turn lanes should be provided when delay caused by left-turning vehicles blocking through vehicles would become a problem." They emphasize the fact that separate left-turn lanes not only increase intersection capacity, but also increase vehicle safety.

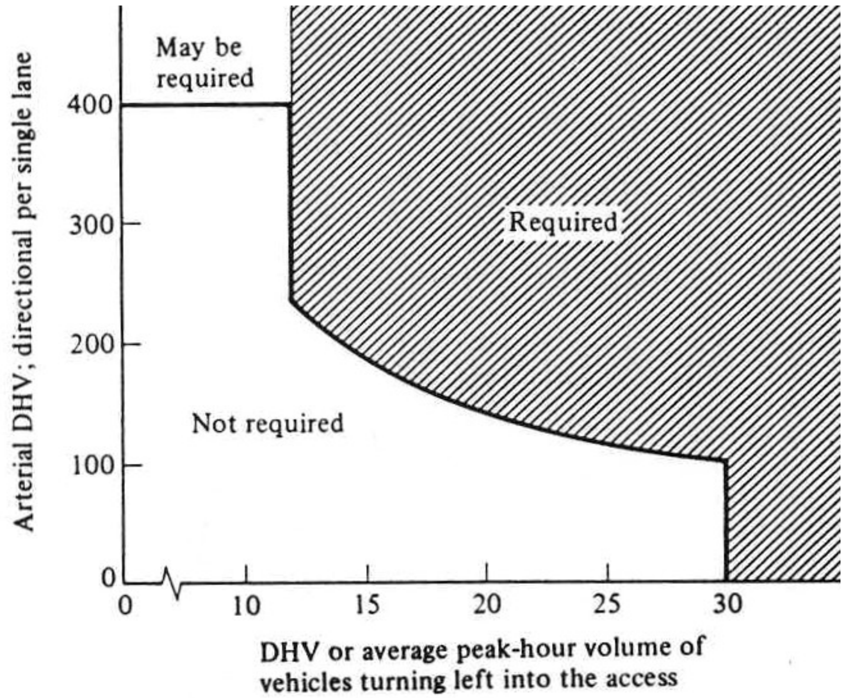


Figure 5-6. NCHRP Report 348 – Left-Turn Lane Guidelines for 30-35 mph (48-56 km/h), 1992 (*37*).

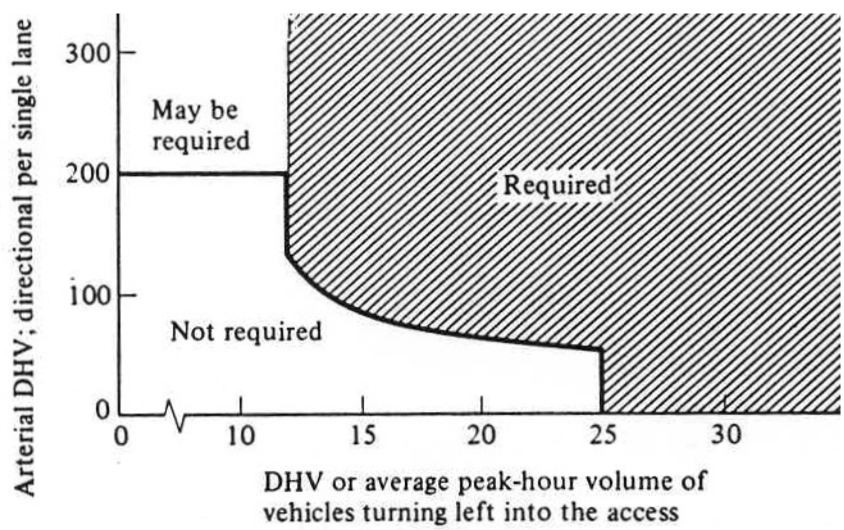


Figure 5-7. NCHRP Report 348 – Left-Turn Lane Guidelines for 40-45 mph (64-72 km/h), 1992 (*37*).

Modur et al.

Modur et al. examined the choice of median design and developed a set of guidelines for determining when to recommend left-turn lanes for arterial streets with speeds less than 45 mph (72 km/h) (*38*). They developed their guidelines using delay data generated from a simulation model. Table 5-2 shows the developed guidelines. The authors noted that sections with left-turn treatments are better than the sections with no treatments, and they recommended that left-turn treatments be used in sections with a disproportionately large number of crashes even though not warranted due to the operational criteria.

Table 5-2. Modur et al. – Left-Turn Lane Warrant Chart, 1990 *(38)*.

Opposing Traffic Volume per Lane per Hour	400-600				<200	Hourly Left-Turn Traffic Volume
	200-400					
	0-200					
	400-600				200-400	
	200-400					
	0-200					
	400-600				400-600	
	200-400					
	0-200					
		0-150	150-300	300-450		
Hourly Straight Through Traffic Volume per Lane						

Black boxes denote that a left-turn treatment is desirable, provided it can be accommodated within the available right-of-way and pavement width. Gray boxes mean that an operational left-turn treatment may be considered. Left-turn lane or raised median is satisfactory based on individual site considerations. White boxes signify that no left-turn treatment is required based on operational considerations.

Hawley and Stover

Hawley and Stover also used delay to generate guidelines on when to install a left-turn lane on four-lane undivided arterials *(39)*. They considered the delay to through vehicles and investigated under what volumes turning vehicles would seriously impact through traffic. They then evaluated the proposed guidelines with a conflict analysis based on the probability of two vehicles arriving at the intersection at the same time to assess the safety aspects of the guidelines. They selected a probability of 0.01 as the maximum likelihood of a conflict. The philosophy of the new guidelines focuses on recommending a left-turn lane above a set directional volume rather than a set turn volume. Figure 5-8 is a graph of the recommended curves for left-turn volumes in vehicles per hour (VPH) and directional volumes in vehicles per hour per lane (VPHPL).

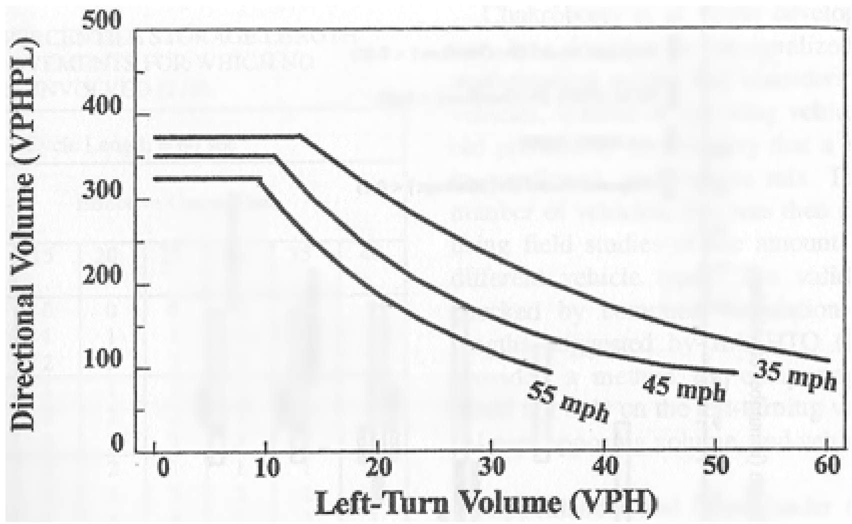


Figure 5-8. Hawley and Stover – Left-Turn Lane Guidelines for Four-Lane Undivided Arterial Street with Nonplatoon Flow, 1996 *(39)*.

NCHRP Report 457

In 2001, Bonneson and Fontaine *(40)* in NCHRP Report 457 discussed determining when to consider a left-turn lane. They sited work by Neuman *(35)* (which was based on the Harmelink model) and re-created the Harmelink model as an interactive spreadsheet available on the Internet as Figure 2-5 in the NCHRP report at http://trb.org/trb/publications/nchrp/esg.pdf.

State Manuals

Several state manuals also include information on when to consider a left-turn lane. The Texas Department of Transportation *Roadway Design Manual* (31) contains the same table of criteria as the values included in the *Green Book* (30) for determining the need for a left-turn lane. The Mississippi Department of Transportation's *Roadway Design Manual* (41) recommendations for the inclusion of left-turn lanes use graphs similar to those presented in NCHRP Report 279 (which is based on Harmelink's work).

Chapter 5 of the New York State Department of Transportation's *Highway Design Manual* (42) refers readers to the AASHTO *Green Book* for traffic volume criteria to consider in determining the need for left-turn lanes. It also includes discussion on the potential to reduce crashes with the installation of a left-turn lane and states that sight distance on the major road is another factor that can create a need for an exclusive left-turn lane.

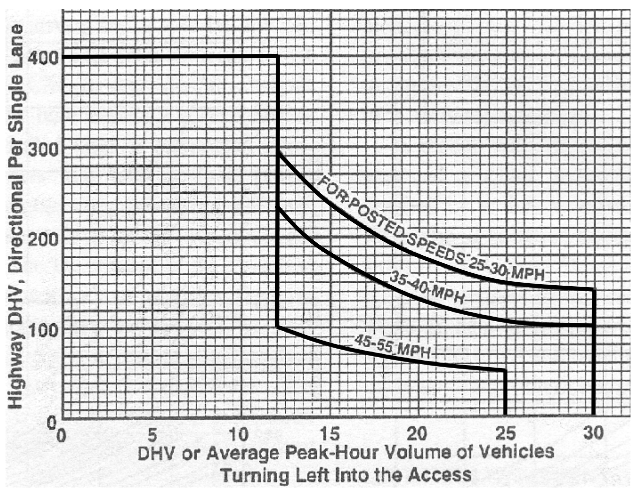
In the *Project Development Manual* (43), the Missouri Department of Transportation (MoDOT) considers left-turn lanes to be necessary where the number of left-turning vehicles is 100 vph or more during the peak hour. Two-lane left turns are necessary when volumes exceed 300 vph. MoDOT also states that the AASHTO *Green Book* should be used as a guide and that left-turn lanes should be considered at intersections where traffic volumes do not warrant but poor visibility or crash records indicate a need.

In its *Location and Design Manual* (44), the Ohio Department of Transportation recommends that left-turn lane installation when left-turn design volumes:

- exceed 20 percent of total directional approach design volumes or
- exceed 100 vph in peak periods.

In Utah, left-turn movements are the only deciding factors for determining the need for an exclusive left-turn lane in rural areas. The *Roadway Design Manual of Instruction* (45) states that in rural areas where there are 25 or more left-turn movements for the main highway in the peak hour, left-turn lanes should be considered. The need for left-turn lanes at signalized intersections is determined by capacity analysis and the acceptable level of service designated for the facility.

The Idaho Transportation Department (ITD) performed a study on left-turn lanes with regard to speed, volume, sight distance, passing opportunity, number of anticipated turning movements, and crash history (46). From that study, ITD determined that the need for left-turn lanes should be established by considering the advancing volume, left-turn volume, and the operating speed, as shown in Figure 5-9.



(1 mi = 1.61 km/h)

Figure 5-9. Idaho Transportation Department – Left-Turn Lane Warrant Graph, 1994 (*46*).

COMPARISON OF METHODS

Several methods use the Harmelink model as the basis of their guidelines, such as the guidelines in the AASHTO *Green Book* and in several NCHRP reports. These guidelines require the knowledge of volumes on each major approach along with the left-turning volume or percentage.

Some of the guidelines, especially a sample of those in state manuals, are based on design hour volumes. The use of design hour volume or left-turn design hour volume lends itself to easier use in a planning stage since volumes on individual major road approaches are not required. The identified procedures can also be subdivided by number of lanes and by speed (which could imply an urban and rural categorization).

Tables 5-3 and 5-4 summarize selected techniques from the literature and state manuals, respectively. Figures 5-9 and 5-10 show a comparison of the methods for a 40 mph (64 km/h) operating speed and a 55 mph (89 km/h) operating speed, respectively, for a two-lane roadway with 10 percent left turns. Figure 5-11 shows a comparison of the methods for four-lane roadways.

Table 5-3. Summary of Selected Left-Turn Lane Literature Guidelines.

Method	AASHTO 2001 (*30*), 1994 (*34*), 1990 (*33*), 1984 (*32*)	NCHRP 279, 1985 (*35*), NCHRP 457, 2001 (*40*)	Oppenlander and Bianchi, 1990 (*36*)	Modur et al., 1990 (*38*)	NCHRP 348, 1992 (*37*)	Hawley and Stover, 1996 (*39*)
Roadway Type	Two-lane	Two-way stop controlled	Two-lane unsignalized	Urban (roadways less than 45 mph [70 km/h])	Any unsignalized	Four-lane undivided
Developed with consideration of:	Minimize conflict	Minimize conflict	Minimize conflict and safety	Delay	Not specified	Delay
Key Feature	Based on Harmelink's 1967 study. Developed table of values for various speeds and left turn percentages.	Based on Harmelink's 1967 study. NCHRP 457 includes a spreadsheet to perform calculations.	Used Harmelink's model and expanded to additional speed ranges. Also added consideration of crashes.	Used simulation to determine guidelines.	Would recommend for lower left-turn volumes than other methods.	Used results from simulation to determine value.
Crashes	"…safety considerations are sufficient to warrant them."	States that there are benefits in crash reduction when left-turn lane is added.	Crashes by approach that would involve a left-turning vehicle: 4 per year at unsignalized and 5 per year at signalized.	"Sections with left-turn treatments are always better than the sections with no treatment."	"Separate turning lane…promote the safety of all traffic."	Guidelines checked against maximum probability of conflict of 0.01. Recommends that the designer consider potential crashes.

Table 5-4. Summary of State Methods.

State	Primary Method	Also Include Consideration of
Texas	*Green Book*	
Mississippi	Harmelink	Crashes and sight distance
New York	*Green Book*	Crashes and sight distance
Missouri	Left turn exceeds 100 vph *Green Book*	Crashes and sight distance
Ohio	Left turn exceeds 100 vph in peak period or 20% of total directional approach design volumes	
Utah	Left turn exceeds 25 vph	
Idaho	Unique graphs, in many cases 12 to 25 design hour volume of left turns	Crashes (4 per year on an existing approach)

When compared (as shown in Figure 5-10 and Figure 5-11), the methods presented in the AASHTO *Green Book* and the NCHRP Reports 279 and 457 overlap, as expected. Not expected was the difference between the AASHTO methods and the numbers provided by Oppenlander and Bianchi. These lines should also have overlapped since they were based on the same methodology. The reason for the difference is not apparent. Other methods available for two-lane highways that use criteria similar to those of AASHTO recommend left-turn lanes at lower volumes (see the curve for IDT on Figure 5-10 as an example). The different methods available for four-lane highways show greater diversity for when a left-turn lane would be recommended (see Figure 5-12).

Several state methods only use a left-turn requirement; hence, those methods are not shown in Figure 5-11 and Figure 5-12 because advancing or opposing volume requirements are not included. They are shown in Figure 5-13 and Figure 5-14 with left-turn volume on the y axis for 40 and 55 mph (64 and 89 km/h), respectively. The left-turn lane should be considered when the volume plots above or to the right of a curve. In most cases, the methods that use only left-turn volumes recommend a left-turn lane at lower volumes than the AASHTO *Green Book* method.

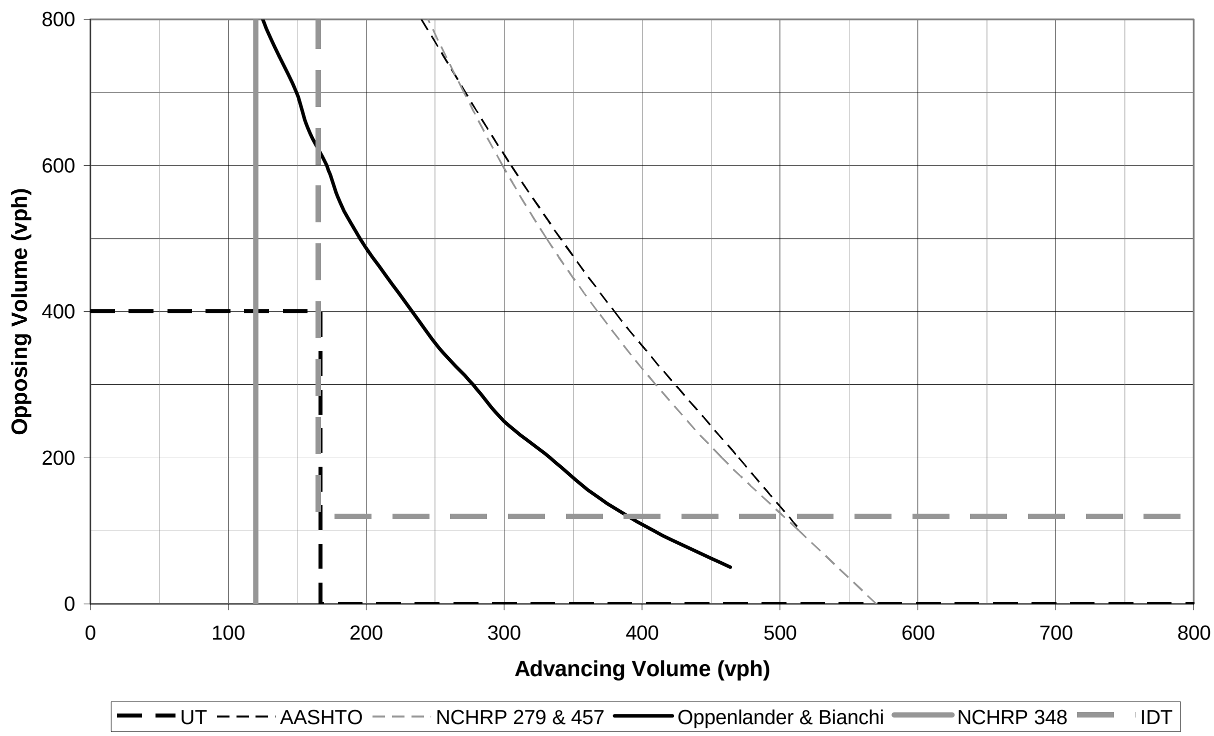


Figure 5-10. Comparison of Left-Turn Lane Installation Guidelines for 10 Percent Left Turns, 40 mph (64 km/h), Two-Lane Highways.

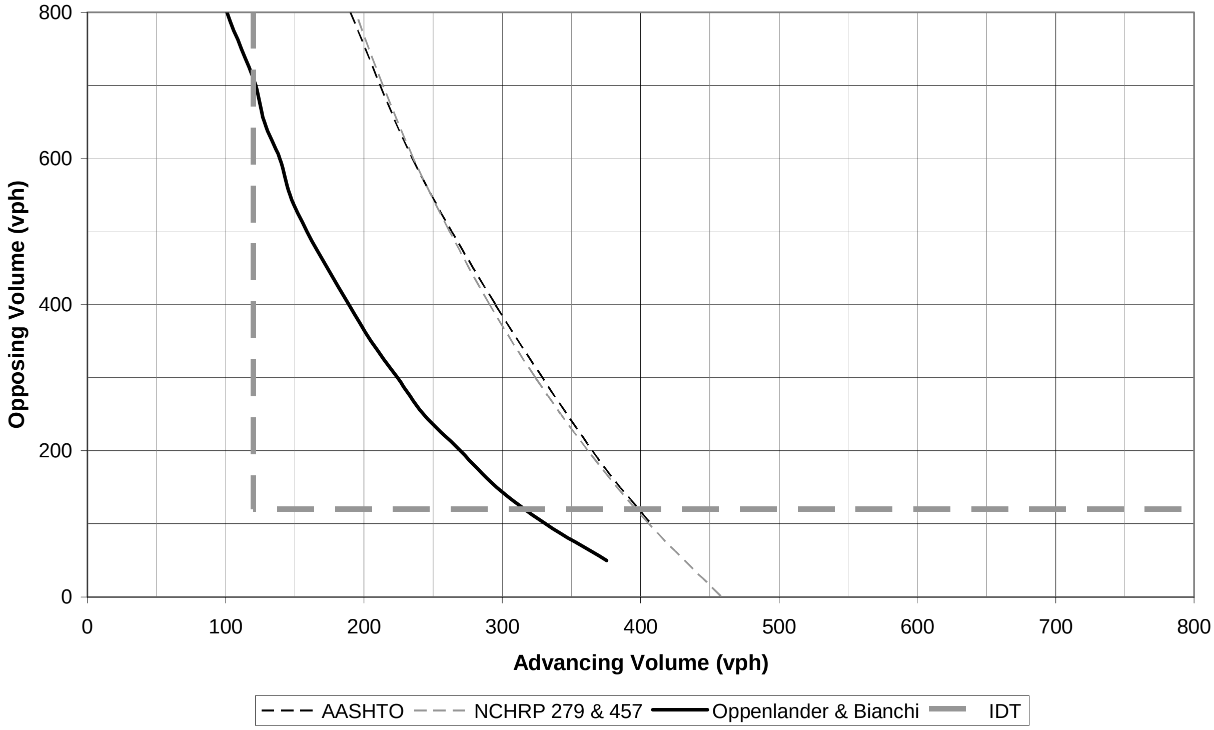


Figure 5-11. Comparison of Left-Turn Lane Installation Guidelines for 10 Percent Left Turns, 55 mph (89 km/h), Two-Lane Highways.

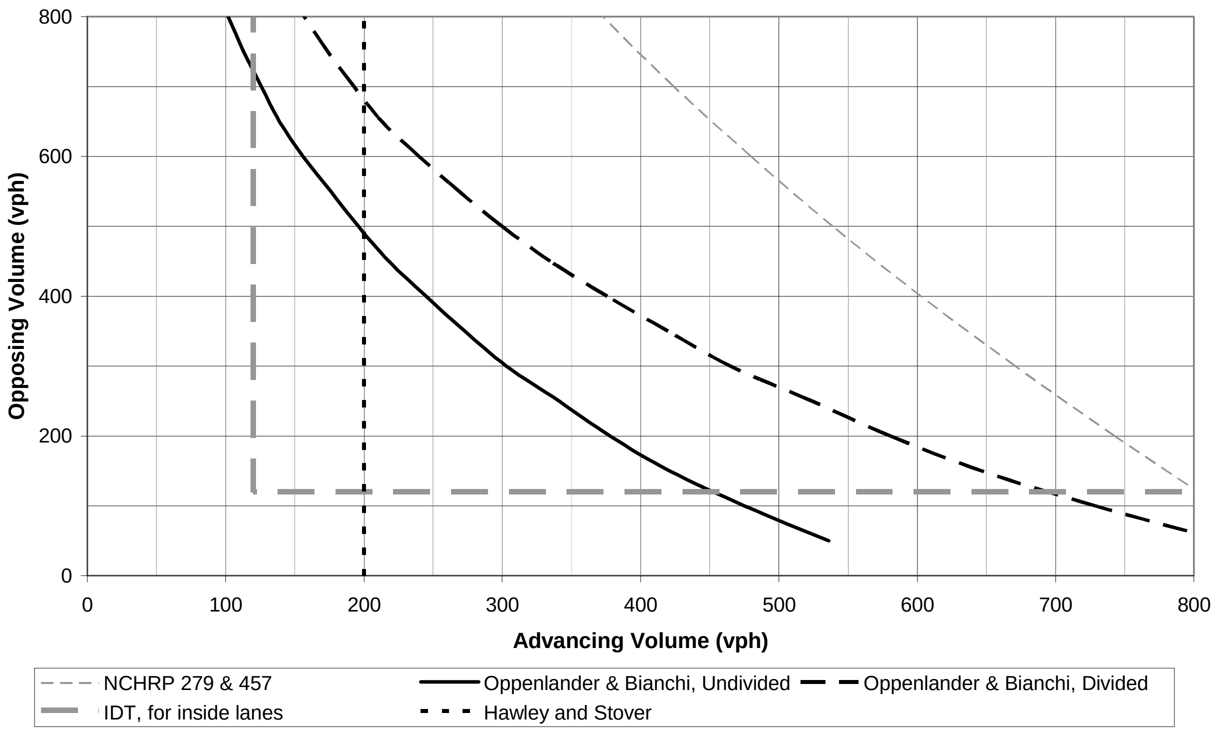


Figure 5-12. Comparison of Left-Turn Lane Installation Guidelines for 10 Percent Left Turns, 55 mph (89 km/h), Four-Lane Highways.

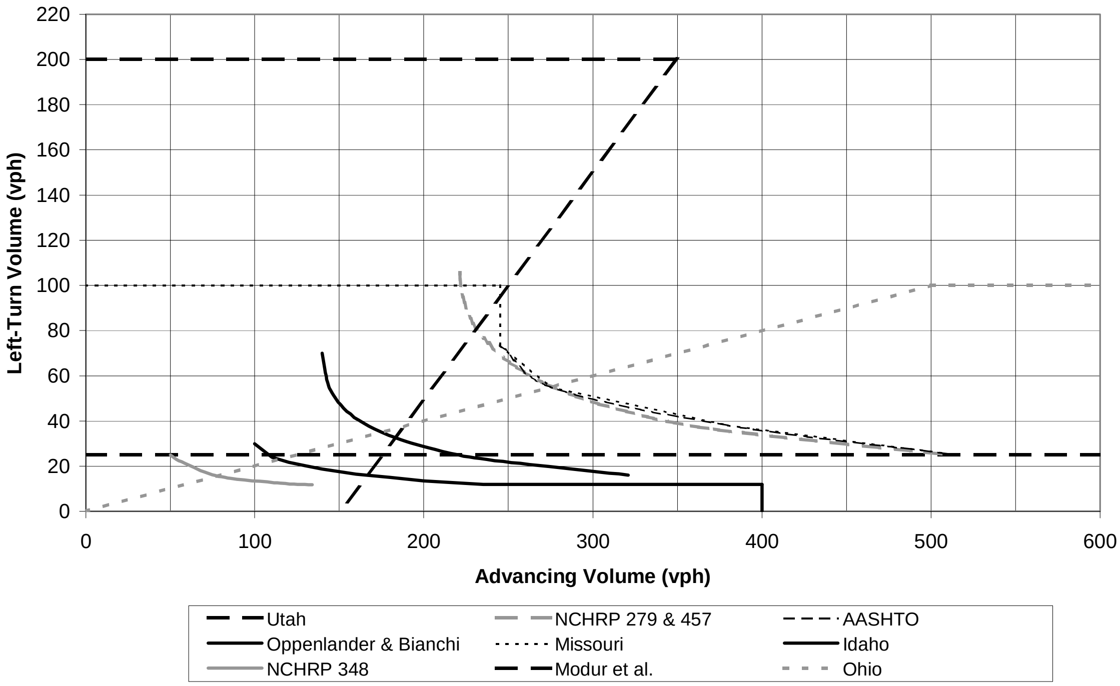


Figure 5-13. Comparison of Left-Turn Lane Installation Guidelines for Opposing Volume of 400 vph, 40 mph (64 km/h), Two-Lane Highways.

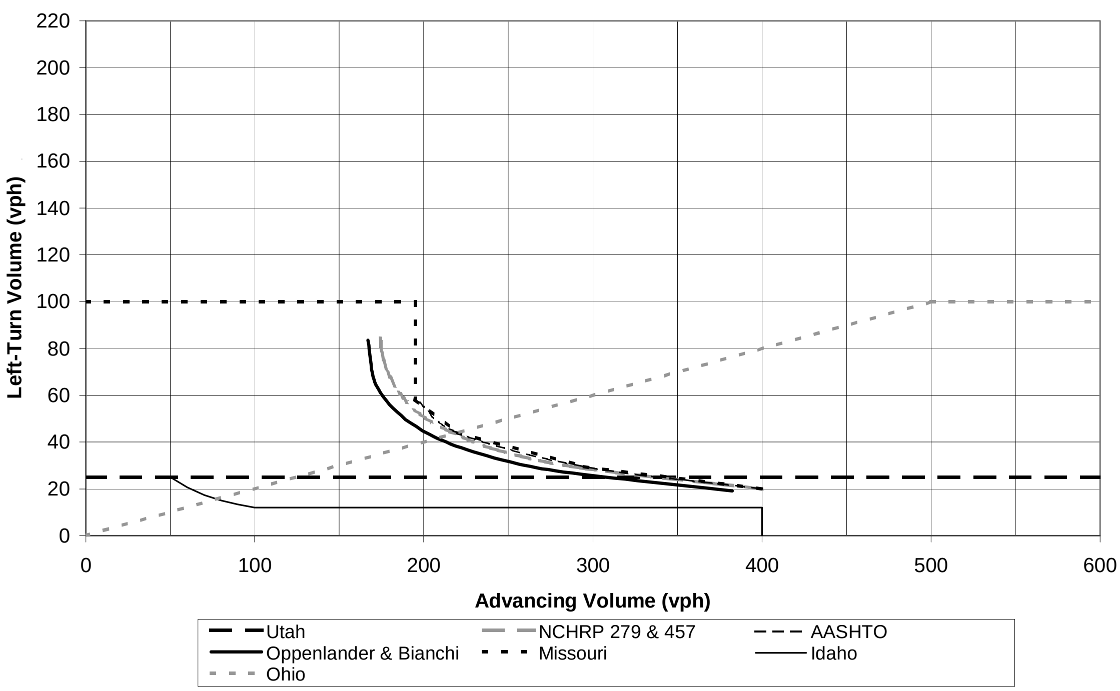


Figure 5-14. Comparison of Left-Turn Lane Installation Guidelines for Opposing Volume of 400 vph, 55 mph (89 km/h), Two-Lane Highways.

UPDATING OF ASSUMPTIONS

In many design manuals, a procedure based on the Harmelink model is the accepted approach. The guidelines developed by Harmelink include the following assumptions:

- The probability of a through vehicle arriving behind a stopped left-turning vehicle should not exceed 0.02 for 40 mph (64 km/h), 0.015 for 50 mph (80 km/h), or 0.010 for 60 mph (96 km/h).
- Arrival-time and service-time distributions are negative exponential.
- Average time required for making a left turn is 3.0 sec for two-lane highways and 4.0 sec for four-lane highways as determined from field studies.
- Critical headway in the opposing traffic stream for a left-turn maneuver is 5.0 sec on two-lane highways and 6.0 sec on four-lane highway as determined from field studies.
- Average time required for a left-turning vehicle to clear or "exit" from the advancing lane is 1.9 sec as determined from field studies.

Probability of Through Vehicle Arrival

Harmelink's assumption for avoiding the arrival of a vehicle behind a left-turning vehicle was scaled to the speed of the facility. He had a lower probability for the higher speed roadways. This assumption should be reflected in calculations for other operating speeds, such as using 0.025 for 30 mph (48 km/h) and 0.005 for 70 mph (113 km/h).

Critical Headway

Several recent research projects have determined the critical gap for use in intersection sight distance calculations and unsignalized intersection capacity analysis. As reported by Harwood et al. (*47*), Kyte et al. recommended a critical gap value of 4.2 sec for left turns from the major road by passenger cars for inclusion in the unsignalized intersection analysis procedures of the *Highway Capacity Manual* (*48*). A heavy-vehicle adjustment of 1.0 sec for two-lane highways and 2.0 sec for multilane highways was also recommended.

A study in Pennsylvania by Miscky and Mason recorded critical gap data for two intersections. They used logistic regression and found critical gaps for a 50 percent probability of acceptance of 4.6 and 5.3 sec (*49*). They also found 85th percentile gaps of 5.5 and 5.9 sec at the two intersections.

It is reasonable that design policies should be more conservative than operational criteria such as the *Highway Capacity Manual.* Using a higher critical gap value, such as the value accepted by 85 percent of the drivers rather than the gap accepted by only 50 percent of the drivers, would result in a more conservative, design-oriented approach. With that philosophy the authors of the 1996 intersection sight distance guidelines recommended a 5.5 sec gap value for use in intersection sight distance (*47*). This gap value should be increased to 6.5 sec for single-unit trucks and 7.5 sec for combination trucks, and an additional 0.5 sec for cars and 0.7 sec for trucks should be added when crossing an additional opposing lane.

Harmelink's assumption of 5.0 sec for the critical gap value is near the values identified in more recent research. If a more conservative gap value for use in design is desired, then the critical gap value should be increased to 5.5 or 6.0 sec. If heavy trucks are a concern at the site, then a higher critical gap should be considered, generally on the order of a 0.5 to 1.0 sec increase in the value assumed for passenger cars.

Time to Make a Left Turn

The 1994 AASHTO *Green Book* (*34*) included information on the amount of time to accelerate and clear an intersection. Assuming minimum travel path and crossing one lane, Miscky and Mason calculated that a left-turning vehicle would travel approximately 47 ft (14.3 m) to clear the opposing lane. Using Figure IX-33 in the 1994 *Green Book*, the estimated time is 4.3 sec for a passenger car accelerating from a stop (which would be the more critical situation). Miscky and Mason also found the travel time for left-turning vehicles at two intersections in Pennsylvania. The mean values were 4.0 and 4.3 sec, while the 85th percentile values were 4.6 and 5.1 sec at the two intersections. The authors noted that the longer turning time at the one intersection was caused by vehicles starting farther back than what is assumed in the theoretical model.

Researchers recorded vehicles turning left at two rural T-intersections in Texas. At the first site, the major roadway had 11 ft (3.4 m) lanes and no shoulders with a 55 mph (89 km/h) speed limit. The site is near San Marcos on RM 150 at an elementary school (see Chapter 3 for additional discussion on the site's characteristics). The data were collected between 7 and 9 am or 2 and 4 pm on two consecutive days. A total of 307 vehicles were recorded making left turns during the 8-hour period, with 71 beginning the turn from a stopped position. The times when a left-turning vehicle began the turn, completely cleared the advancing lane, and completely cleared the opposing lane were recorded. For the vehicles beginning the turn from a stopped position, 85 percent cleared the intersection in 4.1 sec (see Figure 5-15).

At the second rural T-intersection site in Texas, the major roadway had 12 ft (3.7 m) lanes and 10 ft (3.0 m) shoulders with a 65 mph (105 km/h) speed limit. The site is located in Bryan; see Chapter 3 for additional discussion on the characteristics of the intersection (see site 1). Figure 5-16 shows a left turn at the intersection along with vehicles reacting to the turn by driving on the shoulder or slowing behind the turning vehicle. For the 163 vehicles making a left turn from a complete stop, 85 percent completely cleared the intersection in 3.9 sec (see Figure 5-17). In his field studies, Harmelink found that only 3.0 sec was needed to make the left turn. Using the 1994 AASHTO *Green Book* results in a value of 4.3 sec, recent research at two Pennsylvania intersections found 5.1 sec (85th percentile value) for the time to cross the opposing lane, and data from Texas intersections found 4.1 and 3.9 sec. While the more recent research was performed at only four locations, it appears that Harmelink's assumption of 3.0 sec is low.

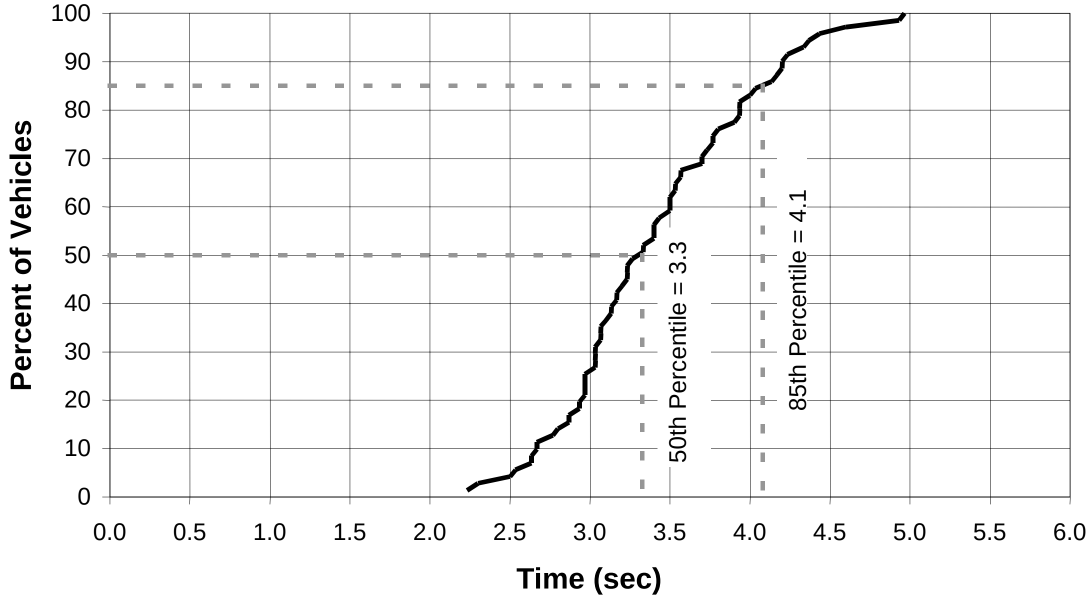


Figure 5-15. Percentile Values for Time to Clear the Opposing Lane When Making a Left Turn at San Marcos, Texas, Site.

Figure 5-16. Bryan, Texas, Site.

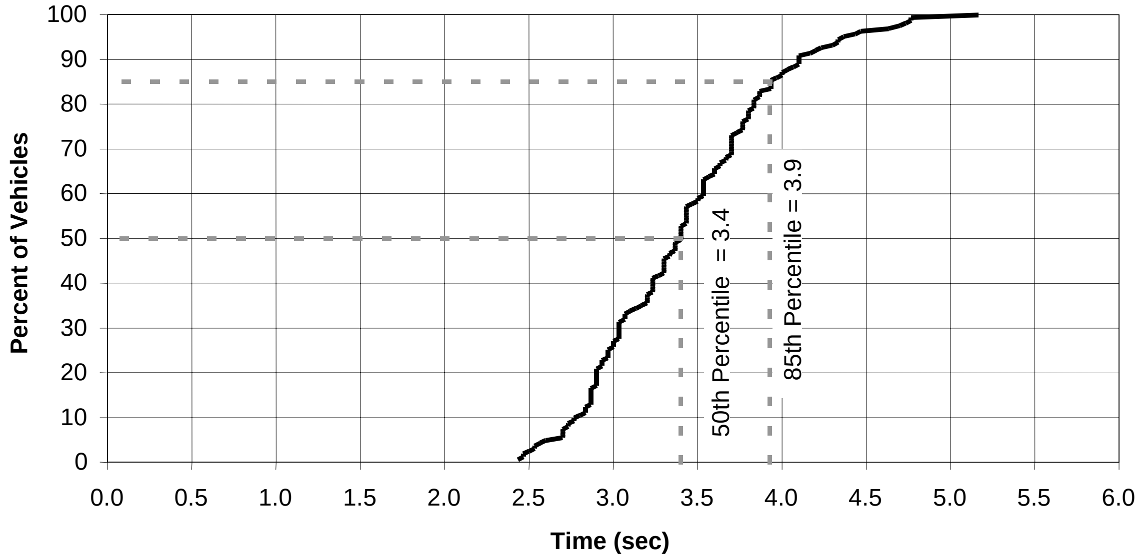


Figure 5-17. Percentile Values for Time to Clear the Opposing Lane When Making a Left Turn at Bryan, Texas, Site.

TIME REQUIRED TO CLEAR

Data on the amount of time to clear the advancing lane that is more recent than the value reported by Harmelink were not found except for the data available from the two rural Texas T-intersections discussed above. For the vehicles beginning the turn from a stopped position, 85 percent used 3.2 sec (based on 71 vehicles) and 2.7 sec (based on 163 vehicles) to clear the lane as shown in Figure 5-18 and Figure 5-19.

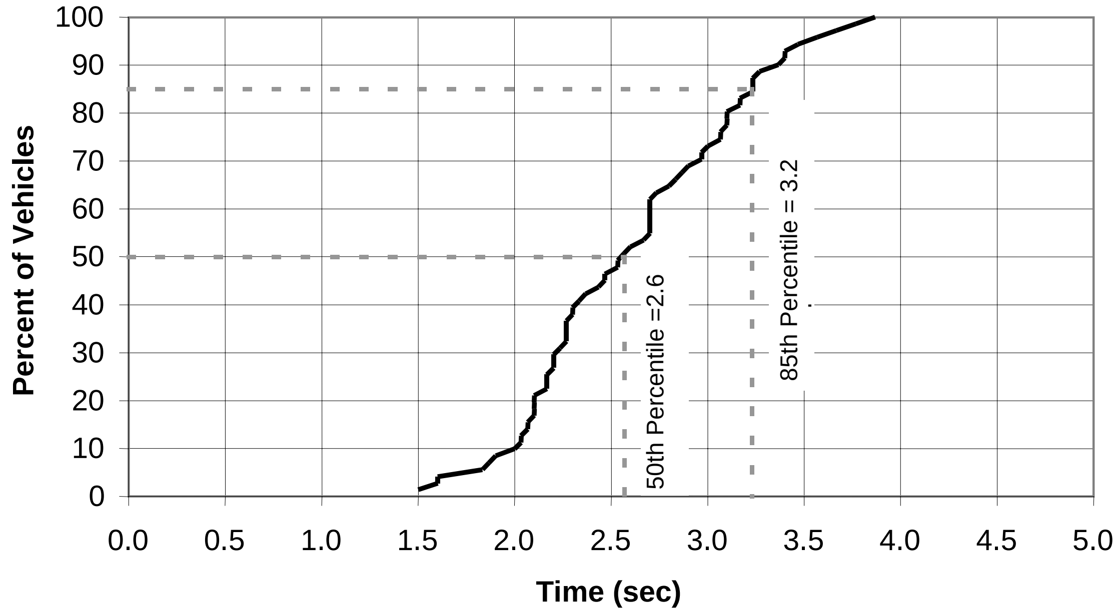


Figure 5-18. Percentile Values for Time to Clear the Advancing Lane When Making a Left Turn at San Marcos, Texas, Site.

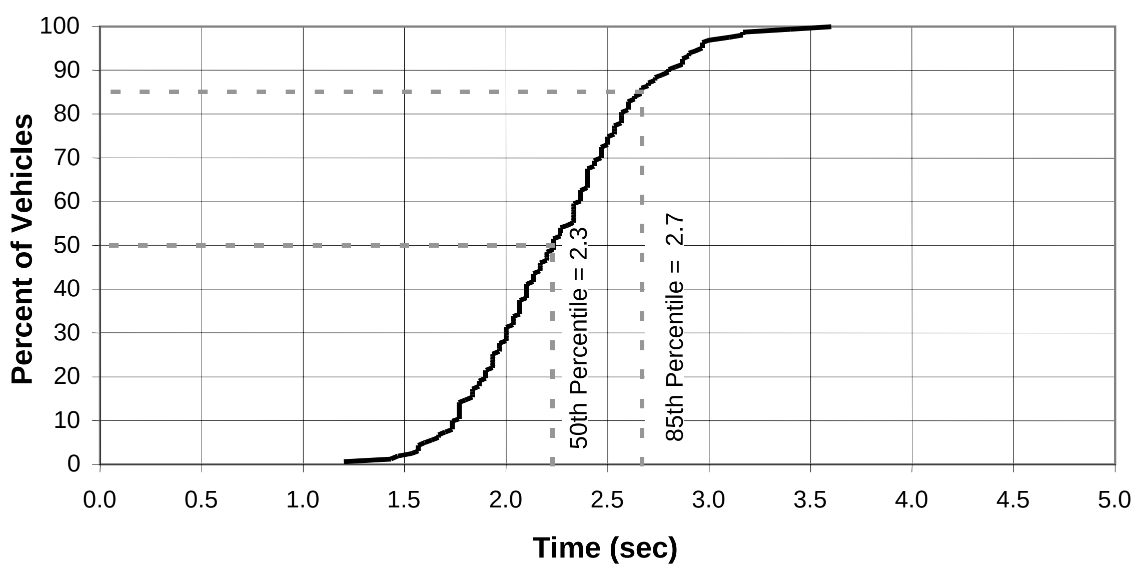


Figure 5-19. Percentile Values for Time to Clear the Advancing Lane When Making a Left Turn at Bryan, Texas, Site.

COMPARISON USING NEW ASSUMPTIONS

Using more recent findings, suggested assumptions for use in the Harmelink model are:

- Critical headway for a left-turn maneuver is 5.5 sec.
- Time to complete the left turn and clear the opposing lane is 4.3 sec.
- Time to clear the advancing lane is 3.2 sec.

Figure 5-20 illustrates the change in the curves when the above assumptions are used.

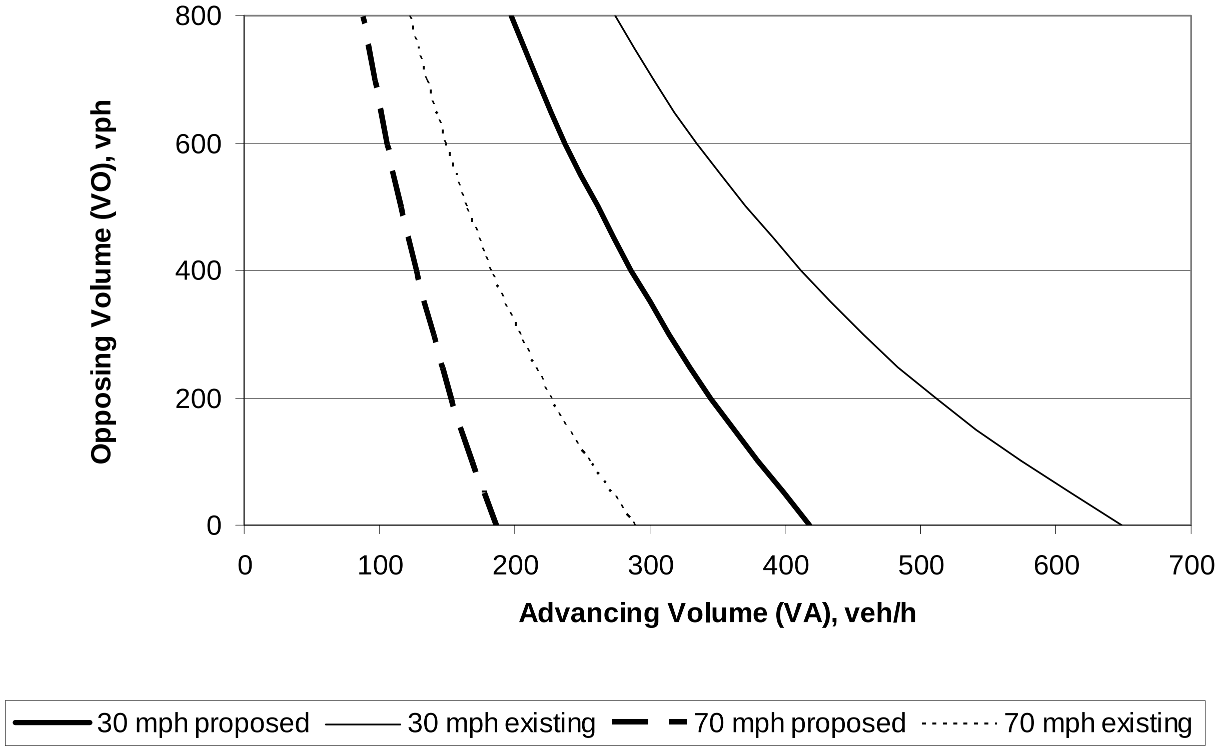


Figure 5-20. Comparison of Existing to Proposed Guidelines (Example Uses 10 Percent Left Turns).

CONCLUSIONS

Several methods are available for determining when to include a left-turn lane in the design at an intersection. Methods based on delay typically do not recommend a left-turn lane at lower left or through volumes when compared to methods based on conflict avoidance or safety. Because of the high benefits for crash reductions provided by left-turn lanes, a method that results in a recommendation at lower volumes would be preferred. The Harmelink model is a widely accepted approach based on conflict avoidance. The procedure first proposed by Harmelink in 1967 includes assumptions that may need revision. Findings from current research on the time to clear an intersection and on critical gaps suggest that Harmelink guidelines should be modified. Table 5-5 lists suggested guidelines for installing left-turn lanes for operating speeds of 30, 50, and 70 mph (50, 80, and 110 km/h).

If the Harmelink approach is preferred and the operating speed of interest (or number of lanes) is not provided in Table 5-5, the reader can use the interactive spreadsheet included as part of NCHRP 457 (http://trb.org/trb/publications/nchrp/esg.pdf). The assumptions need to be changed to match those at the intersection of interest or to reflect more recent research findings such as using a critical gap of 5.5 sec, a time to make left turn of 4.3 sec, and a time to clear of 3.2 sec.

Table 5-5. Guidelines for Installing Left-Turn Lanes on Two-Lane Highways.

Opposing Volume (vph)	Advancing Volume (vph) Percent Left Turns 10	20	40
30 mph (50 km/h)			
800	197	148	121
700	217	162	133
600	238	178	146
500	261	196	160
400	286	215	175
300	314	236	193
200	345	259	211
100	380	285	232
0	418	313	256
50 mph (80 km/h)			
800	153	115	94
700	168	126	103
600	184	138	113
500	202	152	124
400	222	166	136
300	244	183	149
200	268	201	164
100	294	221	180
0	323	243	198
70 mph (110 km/h)			
800	88	66	54
700	97	73	59
600	106	80	65
500	117	88	71
400	128	96	78
300	141	105	86
200	154	116	95
100	170	127	104
0	187	140	114
Example: For a 70 mph (110 km/h) two-lane highway with 10 percent left turns, a left-turn lane should be considered when the opposing volume is 200 vph and the advancing volume is more than or equal to 154 vph.			

CHAPTER 6

SIMULATION OF RURAL T-INTERSECTIONS

Researchers used simulation to obtain an appreciation of the effects of intersection geometry, turning volume, and total volume on rural highway operations.

MODEL SELECTION

The VISSIM v.3.61 microscopic traffic simulation model was selected for the ramp spacing and weaving simulation conducted for TxDOT Project 0-4278 (*50*). The ability to easily create an appropriate vehicle mix within the modeled traffic stream and the flexibility afforded by the model in generating vehicle routing behavior were of particular use to researchers. The fact that the VISSIM input file is an ASCII text file was also beneficial in that researchers were able to use a rapidly created simple text editor and make changes to the 36 input files used in the simulation effort.

EXPERIMENT DESIGN

The simulation performed as part of TxDOT Project 0-4278 had the following objectives:

- Quantify the effects of intersection configuration, turning volume, and total volume on rural T-intersection performance.
- Determine when to consider a left-turn lane (or shoulder bypass lane).

Speed was the primary measure of effectiveness used to evaluate the effect of the different intersection configurations, turning volumes, and total volumes found within the simulation scenarios. Queue length and delay were also reviewed.

Geometric Layout

Common to all of the simulations performed was the basic components of the rural highway and the rural collector. A dual-direction rural highway cross section with two lanes was used. A two-lane rural collector intersected the highway at a 90-degree angle, forming a T-intersection. Figure 6-1 shows the basic geometric outline used for the simulation. Figure 6-2 shows a modified version of the intersection, which includes a left-turn lane.

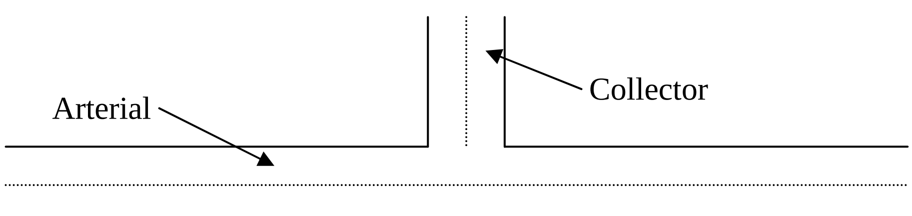


Figure 6-1. Basic T-Intersection.

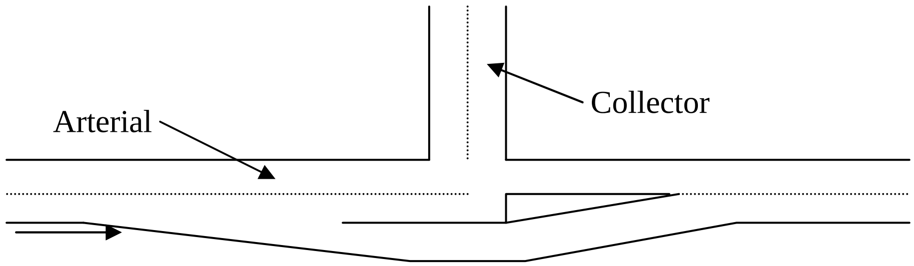


Figure 6-2. T-Intersection with Left-Turn Lane.

Variables

The key variables for the simulation scenarios for Project 0-4278 were:

- intersection configuration (presence of left-turn lane),
- proportion of impeded drivers who use shoulder as bypass lane,
- total initial traffic volume, and
- left-turning volume.

Four combinations of intersection configuration and driver characteristic were simulated:

- No left-turn lane, no shoulder use;
- No left-turn lane, 25 percent of impeded drivers used shoulder;
- No left-turn lane, 90 percent of impeded drivers used shoulder; and
- Left-turn lane.

Initial arterial volumes used were:

- 500,
- 1000, and
- 1500 vph.

The traffic was split with 60 percent eastbound (approaching) and 40 percent westbound (opposing).

Left and right turning percentages from the arterial to the collector were:

- 10 percent,
- 20 percent, and
- 30 percent of through vehicles.

Initial volume on the southbound collector was 50 percent of the initial arterial volume. Left and right turns from the collector to the arterial were evenly split.

A total of 36 unique simulations were run three times each and averaged. Each run simulated 1 hour of traffic. A speed limit of 70 mph was simulated by using a speed distribution of 65 to 77 mph.

VISSIM OUTPUT

Users of the VISSIM simulation model have the ability to specify the type, quantity, and aggregation of simulation output desired. For the Project 0-4278 simulation effort, speed and queue length were primary measures of performance within the system. Accordingly, queue measurement data collection markers were configured within VISSIM so that queue data could be collected for the eastbound rural highway. Data were also collected on arterial speeds at four points in the main through lane spaced 100 ft (31 m) apart upstream from the intersection.

The software recorded a speed for each vehicle that crossed a speed data collection point and then averaged speeds for each collection point. The four collection point averages were averaged, yielding an overall average speed. Figure 6-3 shows the location and type of data collection points. Speed data were always collected in the main through lane of the highway, not on the shoulder bypass or in the left-turn lane.

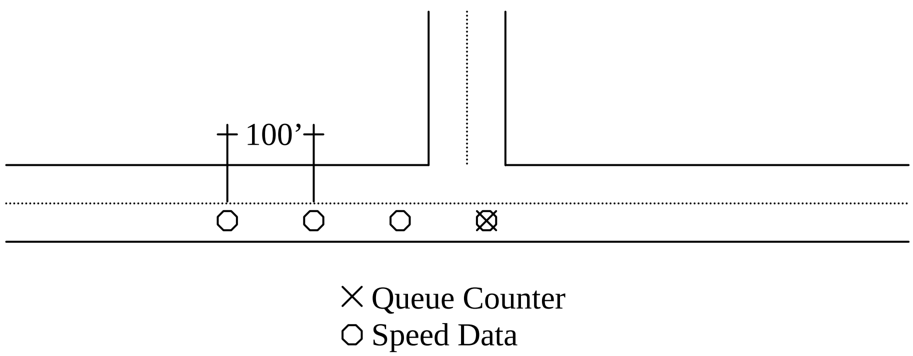


Figure 6-3. Data Collection Points.

Queue data were collected using the queue counter feature in VISSIM. VISSIM counts queues from the location of the queue counter on the link or connector upstream to the final vehicle that is in queue condition. A vehicle enters queue condition when its speed drops below 3.1 mph (5.0 km/h). It remains in queue condition until its speed exceeds 6.2 mph (10.0 km/h). VISSIM measures queue length in feet. To convert feet into cars, the *Highway Capacity Manual* includes a factor of 40 ft (12.2 m) per car *(48)*.

DEVELOPMENT OF GUIDELINES

Different measures were recorded from the simulation runs to evaluate the effects of the different intersection configurations, turning volumes, and total volumes. Average speed at four points at and upstream of the intersection along with queue length and delay were recorded.

Operating speed is being considered within this effort as the criterion to determine whether a left-turn lane should be installed at a rural intersection. The question is at what point is the change in operating speed a concern. Accepting left-turn vehicles having a small impact on the operating speed along a two-lane highway is reasonable; however, when is the impact too large? Should the criterion be that a left-turn lane is installed when the speeds drop more than 15 mph (24 km/h) below the posted speed or the anticipated operating speed? Should the criterion be more conservative and be only 5 mph (8km/h), or should it be 10 mph (16 km/h), which is the speed difference when a climbing lane is considered? How should the Texas driving behavior of passing the left-turn vehicle by driving on the shoulder be factored into consideration? As discussed in Chapter 3 of this report, the majority of drivers *will* pass the turning vehicle on the shoulder.

The simulation produced average speeds; however, discussing criteria in terms of posted speed, anticipated operating speed, or 85th percentile speed is more common. When selecting a posted speed limit for a roadway, the 85th percentile speed is generally used. A recent research project on predicting speeds on rural two-lane highways collected data on 70 mph (113 km/h) roadways (*51*). The data are listed in Table 6-1 for seven sites. The typical difference between the average speed and the 85th percentile speed for 70 mph (113 km/h) roadways is approximately 6.0 mph (10 km/h). Therefore, to adjust the speeds measured from the simulation to reflect the 85th percentile or posted speed limits, add 6.0 mph (10 km/h) to the speed provided.

While different values could be used, for this research the evaluations were performed using an average operating speed of 55 mph (89 km/h) as being the point when a left-turn lane should be considered. The 55 mph (89 km/h) assumption represent about a 10 mph (16 km/h) decrease from the highest average operating speeds measured in the simulation. When adjusted to an 85th percentile value, the 55 mph (89 km/h) threshold represents a 61 mph (98 km/h) operating speed on a 70 mph (113 km/h) roadway. Stated in another manner, when the 85th percentile operating speed drops to approximately 61 mph (98 km/h) on a two-lane highway with a posted speed limit of 70 mph (113 km/h), a left-turn lane should be considered.

Table 6-1. Speed Measurements for a Sample of 70 mph (113 km/h) Rural Two-Lane Highways.

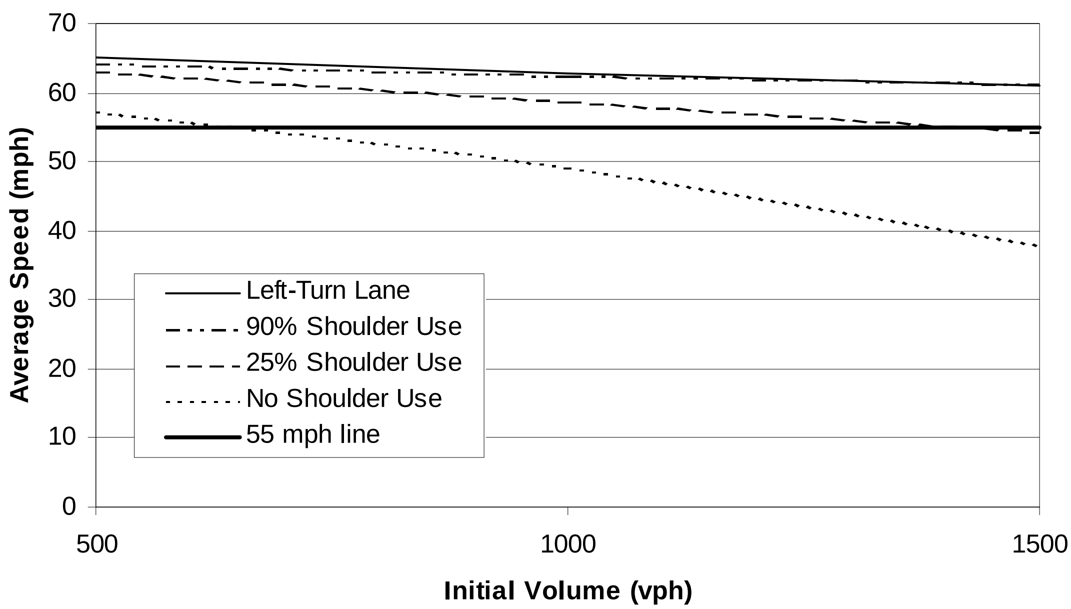
Site Number	Average Speed (mph)	85th Percentile Speed (mph)	Standard Deviation (mph)
1	70.1	75.6	7.5
2	62.0	71.5	7.2
3	67.0	73.6	4.9
4	62.5	68.8	5.0
5	68.0	73.0	5.9
6	68.0	73.0	5.3
7	70.0	74.0	4.9
Average	66.8	72.8	5.8
1 mph = 1.61 km/h			

FINDINGS

Researchers summarized the results from the various simulation runs into figures. Figures 6-4, 6-5, and 6-6 show average speeds for 10, 20, and 30 percent turning vehicles, respectively. To convert the data in Figures 6-4 to 6-6 to an 85th percentile speed, the curves should be shifted upward by 6.0 mph (10 km/h). Also included on the figures is a heavy black line at 55 mph (89 km/h). This line represents one condition of when to consider a left-turn lane.

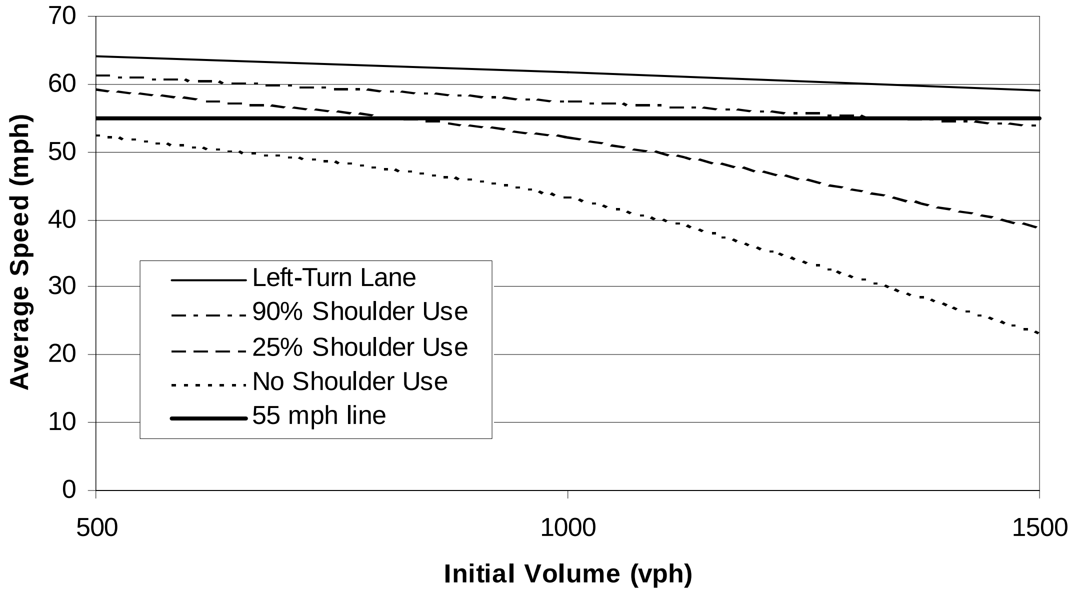
For the range of volumes simulated and with 10 percent of the vehicles turning, only the condition when none of the impeded vehicles used the shoulders resulted in the average speed measured dropping below 55 mph (89 km/h). When the major road volume is 1500 vph, the 25 percent shoulder use condition resulted in speeds just below 55 mph (89 km/h).

As discussed in Chapter 3, at the four sites investigated in this research nearly 90 percent of the approaching vehicles will use the shoulder to pass the turning vehicle. When 90 percent of the vehicles approaching a left-turn vehicle use the shoulder, 30 percent of the more than 1000 vehicles on the two-lane highway must be turning left before the average speed drops below 55 mph (89 km/h) (see Figure 6-6).



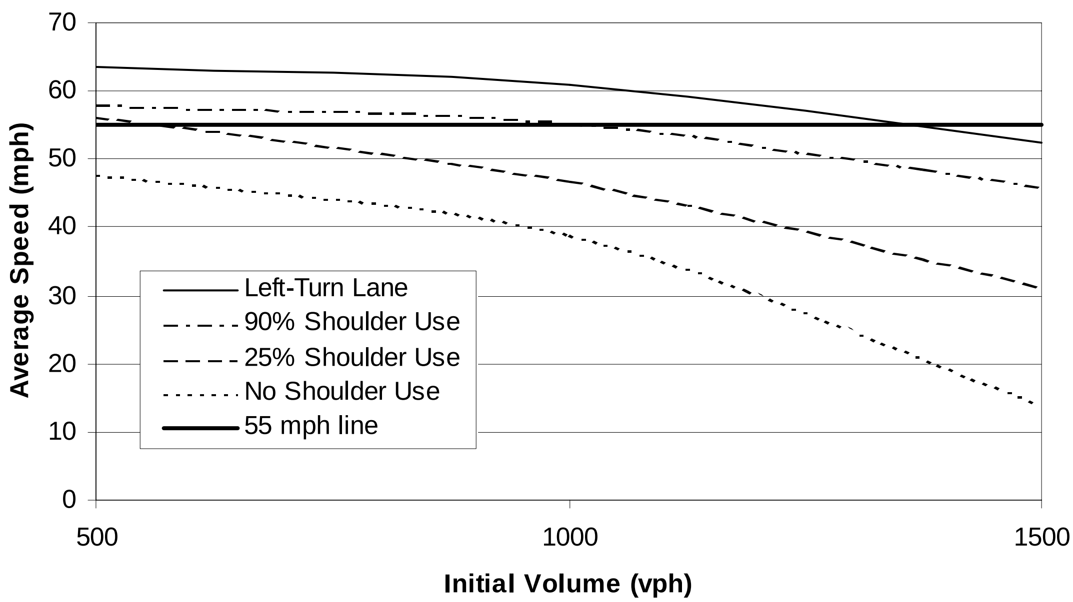
(1 mph = 1.61 km/h)

Figure 6-4. Average Speed with 10 Percent Turning.



(1 mph = 1.61 km/h)

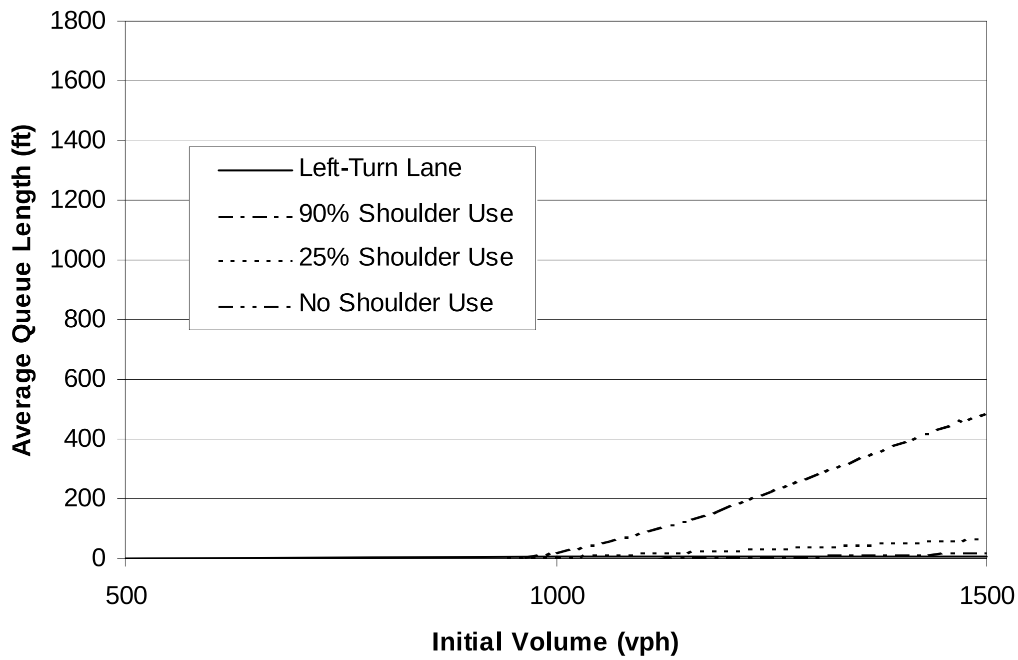
Figure 6-5. Average Speed with 20 Percent Turning.



(1 mph = 1.61 km/h)

Figure 6-6. Average Speed with 30 Percent Turning.

Queue length and delay values for the various combinations of volumes and shoulder usage were also collected. The findings for these variables had similar trends, as shown in the average speed figures. Figure 6-7 is an example of the findings for the queue data for the simulation runs with 20 percent of the traffic turning. Queue lengths are minimal for the left-turn lane and 90 percent shoulder use scenarios. When 25 percent of those approaching a left-turning vehicle used the shoulder, queue lengths up to 70 ft (21.4 m) developed. When none of the approaching through vehicles used the shoulder, the average queue length was approximately 500 ft (153 m) when the major roadway volume was 1500 vph.



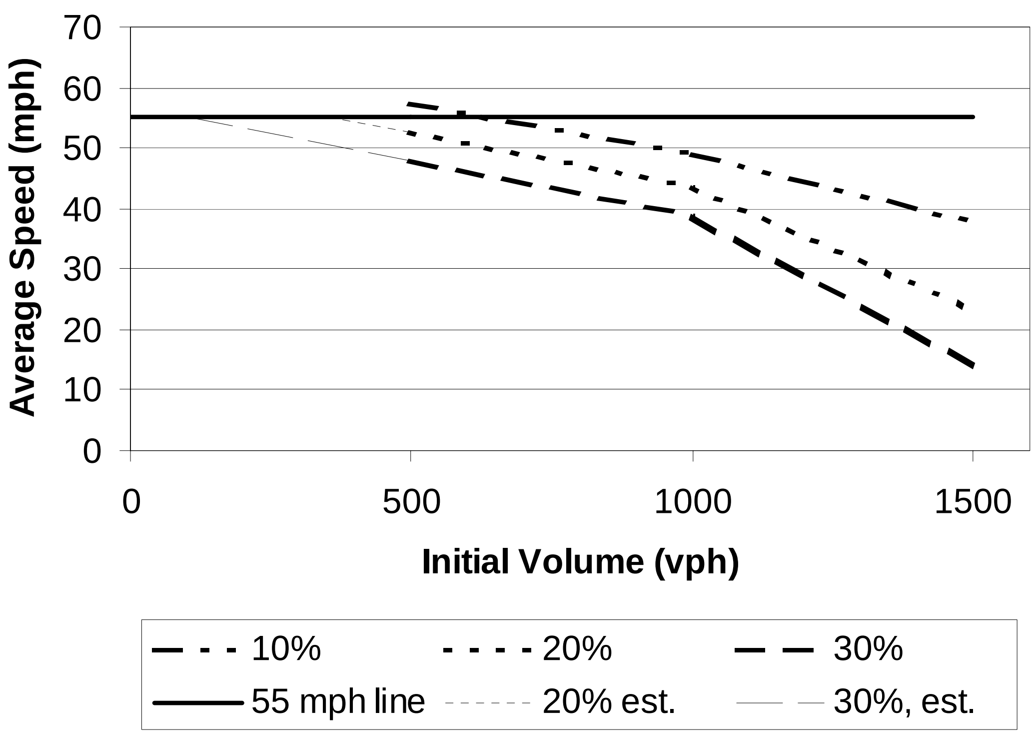
(1 ft = 0.305 m)

Figure 6-7. Average Queue Length with 20 Percent Turning.

DEVELOPED GUIDELINES

Some intersections are not equipped to accommodate the vehicles on the shoulder. This can occur for a number of reasons, including the presence of a curb, poor shoulder pavement, no shoulder pavement, local driver preference, and local laws. Another situation when an intersection cannot accommodate vehicles on the shoulder is in a work zone where the shoulder has been converted into a through lane. The use of edgeline rumble strips has grown in popularity. Their use on freeways and multilane highways has clearly demonstrated safety benefits in reducing crashes, especially run-off-road crashes. While the use of edgeline rumble strips on two-lane rural highways is currently limited in Texas, there is the potential for eventual widespread use. When edgeline rumble strips do become common, they could have major impacts on whether a driver will be willing to cross them in order to pass a turning vehicle. Because of these situations and the desire to design to conservative conditions, the developed left-turn guidelines should be based on no shoulder use rather than reflecting current operating practices of using the shoulder to bypass turning vehicles.

Figure 6-8 shows the curves for the various left-turn percentages when the shoulder is not used to bypass a left-turn vehicle. A speed of 55 mph (89 km/h) was selected within this research as the point when a left-turn lane should be considered. For the 10 percent turning scenario, when 600 vph are on the major roadway, the average speed on the two-lane highway is 55 mph (89 km/h). For the other two turning percentage scenarios, all speeds were less than 55 mph (89 km/h). Therefore, the curves were extrapolated to show where they would intersect the 55 mph (89 km/h) line. When 20 percent of the traffic is turning, the major road speed would be approximately 55 mph (89 km/h) when the initial volume is 350 vph. For 30 percent turning, the volume is only 100 vph; however, this result should be used with extreme caution given the extensive extrapolation needed to determine when the curve could interest the 55 mph (89 km/h) point. The values of 600 and 350 vph for 10 and 20 percent turning, respectively, could represent the volumes when a left-turn lane should be considered at a rural intersection. These values are based on operations (e.g., speed or delay), and consideration of conflict or safety should be made before they are selected as the guidelines.



(1 mph = 1.61 km/h)

Figure 6-8. Average Speed for No Left-Turn Lane and No Shoulder Usage Scenarios.

Chapter 5 discusses other methods used to consider when to install a left-turn lane. The most common method is based on a model developed by Harmelink. The Harmelink model uses the probability of an advancing vehicle arriving behind a stopped left-turning vehicle to determine when a left-turn lane should be considered. Figure 6-9 shows the simulation results on the same graph with plots of the *Green Book* guidelines and plots of the revised Harmelink model guidelines (see Chapter 5 for additional discussion). The simulation assumed that the lane approaching the left-turning vehicles had 60 percent of the initial volumes and 40 percent was in the lane opposing the left-turn vehicles. Therefore, the points in Figure 6-9 for the simulation data represent the 60/40 split. For both the 10 and 20 percent turning traffic scenarios, guidelines developed using the simulation data would result in left-turn lanes being recommended at higher volumes than using either the current *Green Book* guidelines or the guidelines developed from the revised Harmelink model.

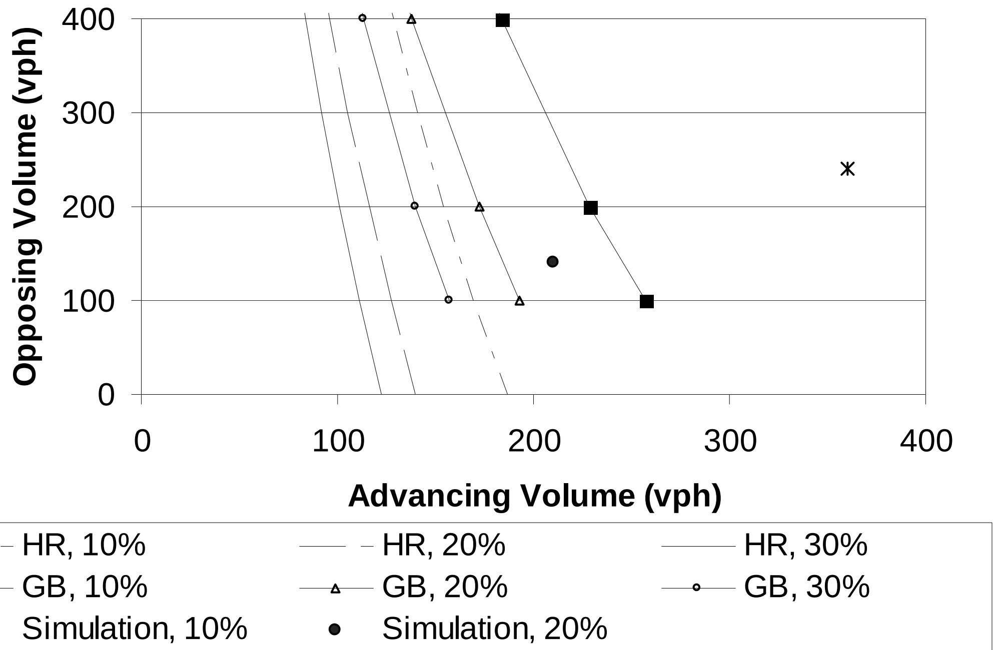


Figure 6-9. Comparison of Advancing Volume to Opposing Volume for *Green Book (GB)* Data, Revised Guidelines Using Harmelink Model (HR), and Simulation Findings.

Another method for looking at the data is to convert the findings to a comparison of left-turn volumes with major road volumes as shown in Figure 6-10. This approach would allow a simpler comparison with some of the other states, methods presented in Chapter 5 (see Figures 5-13 and 5-14 in Chapter 5). Several states use a minimum left-turn volume guideline. These guidelines range from 12 to 100 vph (with several also noting that the *Green Book* or Harmelink guidelines should be considered). In most cases the *Green Book* and the revised Harmelink models both suggest that left-turn lanes should be considered at left-turn volumes below 25 left-turn vehicles per hour. The revised Harmelink model produces values below 12 vph when the major road volume exceeds 600 vph.

Similar to Figure 6-9, Figure 6-10 shows that guidelines developed using the simulation data would not recommend left-turn lanes at the lower volume levels that either the current *Green Book* guidelines or the revised Harmelink model would. Therefore, the recommendation is to use the revised Harmelink model in setting guidelines.

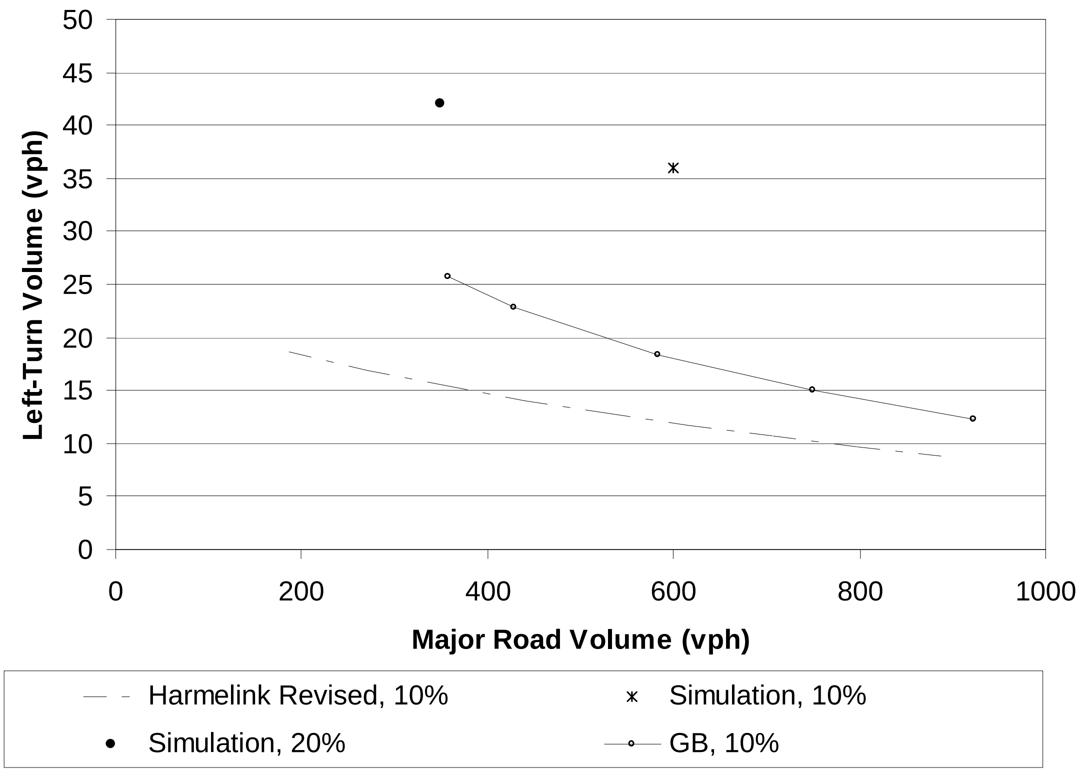


Figure 6-10. Comparison of Major Road Volume to Left-Turn Volume for *Green Book (GB)* Data, Revised Guidelines Using Harmelink Model, and Simulation Findings.

CHAPTER 7

BEFORE-AND-AFTER EVALUATION OF IN-LANE RUMBLE STRIPS

In order to gauge the effectiveness of in-lane (or transverse) rumble strips on driver speeds, rumble strips were installed on selected approaches to rural intersections. A summary of the before-and-after evaluation at each site follows.

SITE SELECTION

During the fall of 2002 and spring of 2003, TxDOT installed transverse rumble strips on 14 approaches at 10 intersections near Abilene, Dallas, and Gatesville. Sites for rumble strip installations near Abilene and Gatesville were selected by the TxDOT Districts. The Dallas installations were identified in previous crash studies that recommended their installation. Table 7-1 provides a brief description of each site.

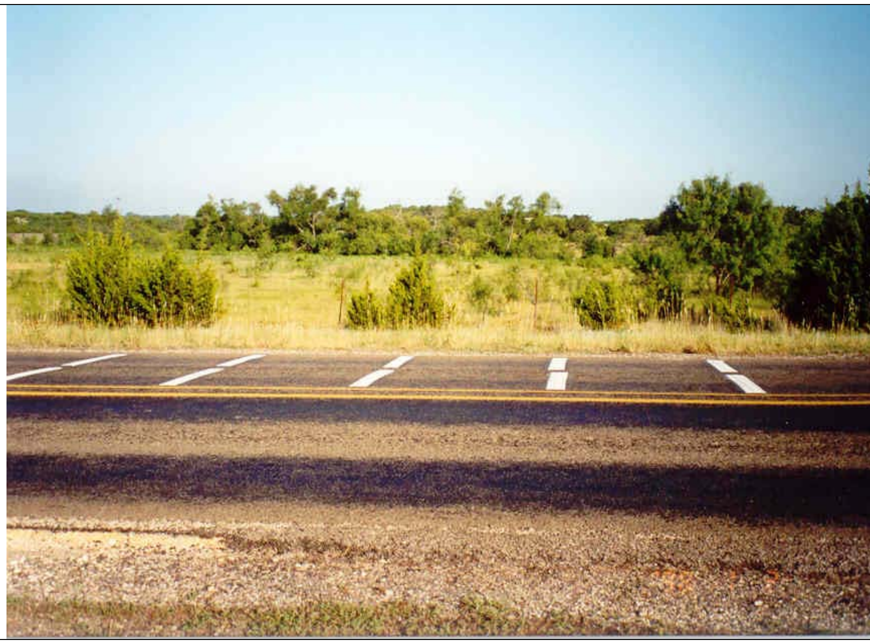
Table 7-1. Description of Transverse Rumble Strip Study Sites.

City	Subject Approach	Crossing Road	Intersection Type	Traffic Control[1]	Speed Limit (mph)[1]	Position of Rumble Strips	Distance from Stop Line to Rumble Strip Set (ft)
Abilene	FM 3326 SB	FM 1226	T	Stop	70	Staggered	848 / 1281
Abilene	FM 2702 WB	US-277	4-leg	Stop/PB[2]	70	Staggered	796 / 1205
Abilene	SH 92 EB	US-277	4-leg	Stop/PB	70	Staggered	532 / 952
Dallas	FM 2728 NB	FM 429	T	Stop	55	Staggered	766 / 1170
Dallas*	FM 1827 SB	US-380	T	Signal	45	Staggered	884 / 1293
Dallas	SH 78 WB	SH 160	T	Stop	65	Staggered	1100 / 1512
Dallas	FM 2514 EB	FM 1378	T	Stop	55	Staggered	850 / 1255
Dallas	FM 2933 NB	FM 545	4-leg	Stop	60	Staggered	878 / 1288
Dallas*	FM 2933 SB	FM 545	4-leg	Stop	60	Staggered	891 / 1296
Gatesville	FM 219 EB	US-281	4-leg	Stop/OB	60	Parallel	1240
Gatesville	FM 219 WB	US-281	4-leg	Stop/OB	60	Parallel	1125
Gatesville*	FM 929 SB	SH 36	4-leg	Stop/PB	55	Parallel	1930
Gatesville	FM 215 NB	SH 36	4-leg	Stop/PB/ OB	50	Parallel	875
Gatesville	FM 215 SB	SH 36	4-leg	Stop/PB/ OB	60	Parallel	878

[1] On the Subject Approach
[2] PB = Pole-Mounted Flashing Beacons; OB = Overhead Flashing Beacons
* Not included in analysis
1 mph = 1.6 km/h, 1 ft = 0.305 m

For all T-intersections, the subject approach was on the stem of the T. All of the subject approaches were two-lane roads.

Two patterns were used for the rumble strips: parallel and staggered. An example of parallel rumble strips is shown in Figure 7-1a, and staggered rumble strips are shown in Figure 7-1b.



a. Parallel rumble strips

b. Staggered rumble strips

Figure 7-1. Examples of Rumble Strip Applications.

DATA COLLECTION METHODOLOGY

Before data were collected when the researchers were notified of the planned installations, and after data were collected approximately 30 days or more after the rumble strip installation. The data collection effort included obtaining both the characteristics of the site and the speed data of vehicles at the site. Generally, one data collector or a team of two data collectors could collect data at two sites a day. The characteristics of the sites were either measured before or after the speed data were collected, or the information for two approaches of the same intersection were obtained at the same time.

Site Characteristics

The site characteristic data collected at each study site are listed in Table 7-2. Data focused on characteristics of the study approach between the stop line and the set of rumble strips located farthest upstream.

Table 7-2. Site Characteristics Data Collected.

General information • Date • City/county • Time of day • Route of subject approach • Data collector's initials • Weather • Intersecting route	**Field observations** • Roadside environment (within 2 ft [0.6 m] and 10 ft [3.0 m] of edge of roadway for approach and opposite directions) • Roadside development • Number of access points within 1 mile (1.6 km) of stop line • Shoulder width and type in each direction • Total pavement width, pavement type and pavement condition in each direction • Presence and width of median
Intersection approach information • Approach grade • Terrain • Sight distance • Number of vertical and horizontal curves within 1 mile (1.6 km) of stop line • Direction of travel on approach • Lane width in approach and opposite directions	**Miscellaneous checklist** • Shoot drive-through video • Take pictures • Draw typical cross-section • Draw diagram of intersection • Other notes on sight distance restrictions, unusual features, or unique characteristics
Traffic control devices • Presence of traffic signal, beacon, or Stop sign • Posted speed limit • Presence and value of advisory speeds • Type of centerline and edgeline markings • Distance from "Stop Ahead/Signal Ahead" sign to stop line • Distance from nearest rumble strip to stop line	**Rumble strip spacing on subject approach** • Distance from stop line to leading edge of first rumble strip • Parallel length of rumble strips • Distance between rumble strips • Offset between edgeline and rumble strips • Transverse width of rumble strips • Offset between rumble strips and centerline • Width of motorcycle path (distance between left and right rumble strips) • Offset between left and right rumble strips • Distance between sets of rumble strips

Each characteristic was measured and recorded in the field. A measuring wheel was used to obtain the width of each lane and the distances to various objects. Roadside development was recorded as residential, commercial, farm/pasture, prison, or trees/cliff/mountain. The number of access points was counted for both the study side of the roadway and the other side of the roadway. Roadside environment was determined for within 2 ft (0.6 m) and within 10 ft (3 m) of the roadway. One of five categories was selected for the section: clear with no fixed objects, yielding objects only, combination of yielding and isolated rigid objects, isolated rigid objects only, and many or continuous rigid objects.

Speed Data Collection

A laser gun connected to a laptop computer recorded speeds of subject vehicles. The use of laser guns in speed data collection has two major advantages over radar. First, laser guns can measure distance to a vehicle as well as the speed of that vehicle, while the radar guns only measure speed. To measure speed and distance, the gun releases hundreds of infrared light pulses every second. As each pulse is transmitted, a time is started. When the energy of the light pulse is received by the device, the time is stopped. Based on elapsed time, the distance is calculated using the known speed of light through the atmosphere. An algorithm is used to derive the speed of the target from a successive number of range calculations.

The second advantage of laser over radar is that the transmitted signal travels in a straight line, whereas the radar transmission is conically shaped. The narrower beam has at least two distinct advantages: it is harder to detect with conventional radar and laser detectors and it allows for more precise measurements of individual speeds. It has the capability of continuously tracking a vehicle’s speed through a section of the roadway.

Only free-flowing vehicles were desired as subjects. Vehicles that braked, turned, or exhibited any unusual behavior were not used (see “Data Reduction” below). Data were only collected during daylight hours, generally between 8:00 am and 5:00 pm. The data were collected only on weekdays. Texas Transportation Institute (TTI) developed a software program to transmit the speed, time, and distance from the laser gun to a laptop computer. The transfer of data occurs at a rate of approximately three times per second. The program also has the capability of adding a remark to each profile, describe the target vehicle, or comment on unusual behavior or circumstances.

The goal was to collect speed profiles for 125 vehicles on each study approach. However, on roadways with low volumes, it could take an entire day to collect the desired 125 vehicles. Therefore, 3 hours was set as a maximum time limit to collect speed data at each approach.

At some sites, the geometry of the approach prohibited data collection by one collector, so a team of two collectors was used. This allowed for a lengthy speed profile over vertical curves and around horizontal curves or roadside obstructions.

Data Reduction

The collected speed and distance data were transferred into a spreadsheet and examined for irregularities or errors. Any vehicles that had been tagged in the field for unusual behavior were removed from the file, as were vehicles that turned prior to the intersection or vehicles that were impeded by queues at the intersection or other traffic. For sites that had a team of collectors, vehicles were removed from the file if they did not have profiles recorded by both collectors.

For the remaining vehicles, the recorded speed was converted to an absolute value and the position of the vehicle for each reading was converted to a distance relative to the stop line on that approach. The separate profiles recorded by a team of data collectors were combined to produce one complete profile per vehicle. The resulting file produced an output similar to that shown in Table 7-3.

Table 7-3. Example of Speed Profile.

	Time	Speed	Distance
DAT	8:42:24	45	543
DAT	8:42:24	45	523
DAT	8:42:24	45	503
DAT	8:42:25	44	465
DAT	8:42:25	44	445
DAT	8:42:26	44	427
DAT	8:42:26	44	408
DAT	8:42:26	43	389
DAT	8:42:27	43	371
DAT	8:42:27	42	353
DAT	8:42:27	42	335
DAT	8:42:27	42	317
DAT	8:42:28	41	304
DAT	8:42:28	41	287
DAT	8:42:30	34	189
DAT	8:42:30	34	174
DAT	8:42:30	33	163
DAT	8:42:32	26	104
DAT	8:42:33	24	82
DAT	8:42:34	18	50
DAT	8:42:35	8	27
DAT	8:42:35	8	25
REM	gry car		

After reviewing the reduced data for each site, researchers determined that three sites had insufficient data to make a meaningful before-and-after comparison. These sites are noted in Table 7-1. For example, FM 2933 SB had very few free-flowing through vehicles in the after period; many of the vehicles on that approach were heavy trucks that turned into and out of a soil storage area on land adjacent to the roadway. As a result, the reduced population of usable vehicles was too small to compare with conditions before improvement. Therefore, the site was removed from further analysis.

SITES

There were three study site approaches at two intersections near the city of Abilene. The study sites include:

- FM 3326 SB at FM 1226, and
- FM 2702 WB and SH 92 EB at US-277.

There were six study site approaches at five intersections near the city of Dallas. The study sites include:

- FM 2728 NB at FM 429 in Kaufman County,
- FM 1827 SB at US-380 in Collin County,
- SH 78 WB at SH 160 in Collin County,
- FM 2514 EB at FM 1378 in Collin County, and
- FM 2933 NB and SB at FM 545 in Collin County.

There were five study site approaches at three intersections near the city of Gatesville. The study sites include:

- FM 219 EB and WB at US-281 (in Hico),
- FM 929 SB at SH 36, and
- FM 215 NB and FM 215 SB at SH 36.

After reviewing the data, three sites were removed from the analysis, as noted in “Data Reduction” above. Tables 7-4 through 7-14 illustrate conditions at each of the 11 remaining approaches before (left column) and after (right column) rumble strips were installed.

Table 7-4. Before and After Comparison of FM 3326 at FM 1226 near Abilene.

BEFORE	
 Stop Ahead sign.	
AFTER No changes were made at this site other than the addition of two sets of rumble strips.	
 Stop Ahead sign with rumble strips in view. Both sets are 200 ft (61 m) either side of Stop Ahead sign.	 No changes at the intersection.
 Rumble strips downstream of the Stop Ahead sign. Stagger starts with outside strip first. The distance between each strip is 4 ft, 6 inches (1.4 m). The total length of the set is 20 ft, 8 inches (6.3 m). Distance between inside and outside strip is approximately 10 inches (0.25 m). Both sets are similar in dimensions except for the leading strip.	 Rumble strips upstream of the Stop Ahead sign. Stagger starts with inside strip first in the direction of travel.

Table 7-5. Before and After Comparison of FM 2702 at US-277 near Abilene.

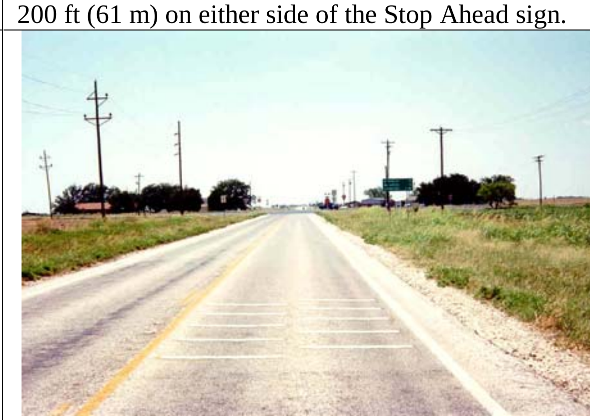
BEFORE	AFTER
 Stop Ahead sign with no beacons, 1027 ft (313 m) from stop bar.	 Stop Ahead sign with new beacons on top, 1003 ft (306 m) from stop bar. Rumble strips approximately 200 ft (61 m) on either side of the Stop Ahead sign.
 View of roadway without rumble strips.	 Rumble strips between the Stop and Stop Ahead signs. Distances between rumble strips are 4 ft, 6 inches (1.4 m). The total length of the strips is approximately 20 ft, 9 inches (6.3 m) for each set. The distance between the inside and outside strips is approximately 10 inches (0.25 m). Both sets were placed in similar fashion.
 Stop sign at intersection of US-277 with no beacons.	 Stop sign at intersection with beacons.

Table 7-6. Before and After Comparison of SH 92 at US-277 near Abilene.

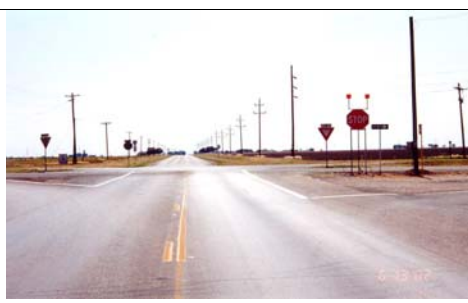
BEFORE	AFTER
 Highway intersection sign with no beacons.	 Highway intersection sign with beacons.
 Stop Ahead sign and “Stop Ahead” on pavement (buttons in pavement).	 The same Stop Ahead sign and the same pavement markings are still in place. Rumble strips 200 ft (61 m) on either side of the sign.
 Second set of Stop Ahead pavement markings.	 Distance between rumble strips is 4 ft, 6 inches (1.4 m). The total length of the strips is 20 ft, 8 inches (6.3 m). Both sets are similar in layout.
 Stop sign with beacons on top.	 Stop sign with new beacons on top of sign.

Table 7-7. Before and After Comparison of SH 78 at SH 160 near Dallas.

BEFORE	
 Aerial View of SH 78 @ SH 160.	
	

Table 7-7. Before and After Comparison of SH 78 at SH 160 near Dallas (continued).

AFTER This site looks the same except for the rumble strips that are placed on either side of the Stop Ahead sign.	
	
	
There are 2 sets of rumble strips, one on each side of the Stop Ahead sign. The set closest to the stop bar is 202 ft (62 m) from the sign. The other set is 200 ft (61 m) from the sign. The strips are staggered with the outside strip being the lead strip. The total length of each set is 8 ft, 8 inches (2.6 m). The distance between each strip is 1 ft, 8 inches (1.5 m). There is 2 ft (0.6 m) between the inside and outside strips.	

Table 7-8. Before and After Comparison of FM 2514 at FM 1378 near Dallas.

BEFORE	AFTER
 Aerial Photo of FM 2514 at FM 1378.	
 There are two Stop Ahead signs on the approach during the before data. They are 330 ft (101 m) and 905 ft (276 m) from the intersection.	 This Stop Ahead sign is 1055 ft (322 m) from the intersection. There is only one Stop Ahead sign during the after study.
	

Table 7-8. Before and After Comparison of FM 2514 at FM 1378 near Dallas (continued).

BEFORE	AFTER
 Stop Ahead sign.	 There are two sets of rumble strips. They are approximately 200 ft (61 m) on either side of the Stop Ahead sign. The total length of each set is 8 ft, 8 inches (2.6 m). The distance between each strip is 1 ft, 6 inches (0.46 m). There is 1 ft (0.3 m) between the outside and inside strips.
 The Stop sign has a two-pole mount during the before period.	 The Stop sign is on a single pole during the after period.

Table 7-9. Before and After Comparison of FM 2933 NB at FM 545 near Dallas.

BEFORE	AFTER
 Aerial Photo of FM 2933 NB at FM 545.	
 The Stop Ahead sign is on a short pole. This sign is 962 ft (293 m) from the intersection.	 The Stop Ahead sign is on a different pole. This sign is 1080 ft (329 m) from the intersection.
	

Table 7-9. Before and After Comparison of FM 2933 NB at FM 545 near Dallas (continued).

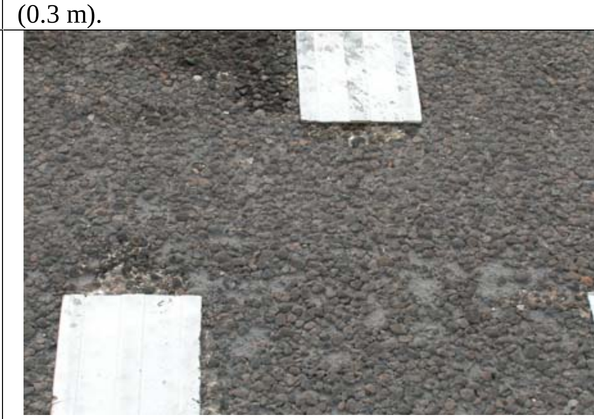
BEFORE	AFTER
	 Inside set of rumble strips.
	 There are two sets of rumble strips. Each one is approximately 200 ft (61 m) from the Stop Ahead sign. Each set is staggered with the outside strip leading. The total length of the set is 8 ft, 8 inches (2.6 m). Each strip is 1 ft, 6 inches (0.46 m) apart. The distance between the inside and outside strips is 1 ft (0.3 m).
All of the signs have been updated with new poles. It is hard to tell if they have changed position.	

Table 7-10. Before and After Comparison of FM 2728 at FM 429 near Dallas.

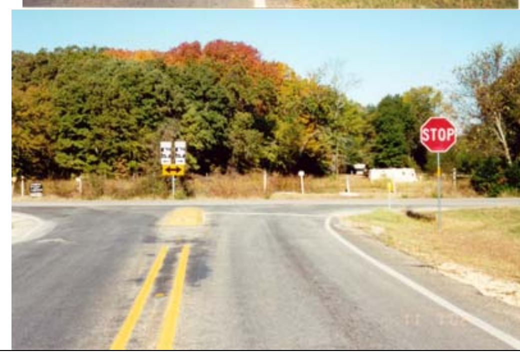
BEFORE	
 Aerial View of FM 2728 at FM 429.	 
AFTER	
	
	
There are no changes to the approach except for the addition of the rumble strips. The rumble strips are staggered with the inside being the lead. There is a set of strips on either side of the Stop Ahead sign. They are approximately 200 ft (61 m) on either side. The total length of each set is 10 ft, 6 inches (3.2 m). There is 2 ft (0.6 m) between each strip. The distance between the inside and outside strips is also 2 ft (0.6 m).	

Table 7-11. Before and After Comparison of FM 219 EB at US-281 near Gatesville.

BEFORE	AFTER
	
	
	
	There is one set of rumble strips. They are 200 ft (61 m) upstream of the Stop Ahead sign. They are parallel, not staggered. The total length of the set is 21 ft, 8 inches (6.6 m). The distance between each set is 4 ft, 4 inches (1.32 m) to 5 ft, 6 inches (1.67 m). There are no other changes to the approach except for the rumble strips. The rumble strips were put down with "tar" instead of the glue used elsewhere.

Table 7-12. Before and After Comparison of FM 219 WB at US-281 near Gatesville.

BEFORE	AFTER
	
	
	
	There are no changes to the approach except for the addition of the rumble strips. The rumble strips are 175 ft (53 m) from the Stop Ahead sign and are in advance of the sign. The total length of the set is 21 ft, 6 inches (6.5 m). The distance between each set is approximately 4 ft, 8 inches (1.4 m). The strips were put down using tar instead of glue.

Table 7-13. Before and After Comparison of FM 215 NB at SH 36 near Gatesville.

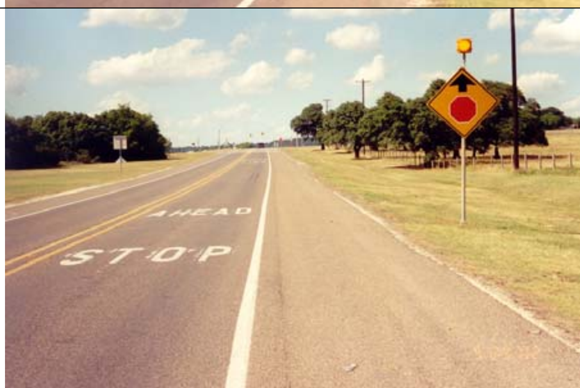
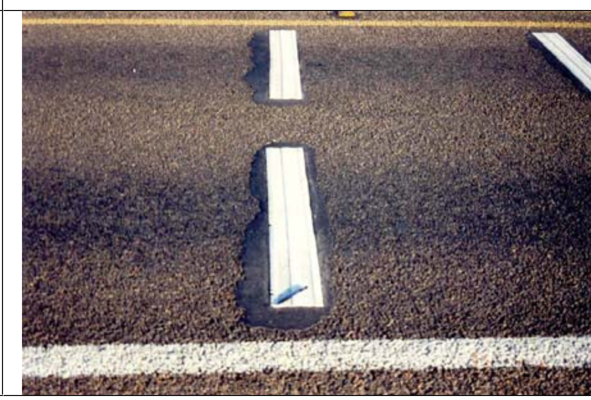
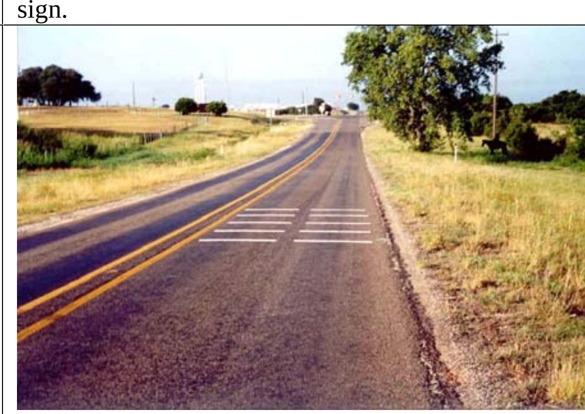
BEFORE	AFTER
	
	
	
Nothing changed at this site except for the rumble strips being added. They were placed between the Stop sign and the Stop Ahead sign. They are 135 ft (41 m) from the Stop Ahead sign. The distance between the strips range from 4 ft, 6 inches (1.37 m) to 4 ft, 10 inches (1.47 m). The total length of the set is 21 ft, 3 inches (6.47 m). Each pair is approximately 2 ft (0.6 m) apart. The rumble strips are not staggered. They are parallel with each other.	

Table 7-14. Before and After Comparison of FM 215 SB at SH 36 near Gatesville.

<table>
<tr><th>BEFORE</th><th>AFTER</th></tr>
<tr><td>

Stop Ahead sign with beacon and pavement markings.</td><td>

Same as before except the rumble strips in the foreground. Rumble strips are 117 ft (35.6 m) from the sign.</td></tr>
<tr><td>
</td><td>
Rumble strips are not staggered; they are in line with each other. The distance between the strips ranges from 4 ft, 6 inches (1.37 m) to 5 ft, 0 inches (1.52 m). The total length of the set is 21 ft, 9 inches (6.6 m).</td></tr>
<tr><td>

At the intersection, Stop sign with beacon, beacons over the intersection.</td><td>

Same as before period.</td></tr>
</table>

DISCUSSION OF RESULTS

Site-by-Site Comparisons

Analysis of the speed data produced site-by-site comparisons of the speed profiles before and after rumble strips were installed at each of the 11 study sites. These profiles consisted of speeds collected at regular intervals along each study site approach and allowed a direct assessment of the difference between speeds in the before and after periods. However, the magnitude of the difference between corresponding speeds in the two periods does not provide an indication of whether the difference is statistically significant. Therefore, a significance test was performed on the data from the 11 sites. While traditional analysis of speeds has focused on the 85th percentile speed, the test for statistical significance uses the mean (average) speed. The test statistic is based on the following formula:

$$t = \frac{\overline{X}_1 - \overline{X}_2}{\sqrt{\frac{s_1^2}{N_1} + \frac{s_2^2}{N_2}}}$$

where

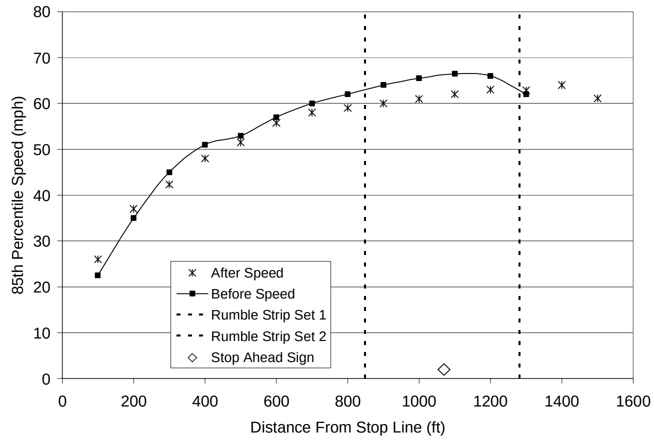
- t = statistic of the t distribution
- X_1 = mean of first sample (before period speeds)
- X_2 = mean of second sample (after period speeds)
- s_1 = standard deviation of first sample
- s_2 = standard deviation of second sample
- N_1 = number of observations in first sample
- N_2 = number of observations in second sample

The computed value of t is then compared with the critical value of t (t_c) for the sample size. The value of t_c is selected in accordance with a specified level of significance, usually 0.05, which corresponds to 95 percent confidence. If the computed value of t is greater than t_c, the difference between the two means is significant. For this test, the calculation of t was not performed when the difference between means was less than 1.0 mph (1.6 km/h), which was the error of the laser guns used in this study.

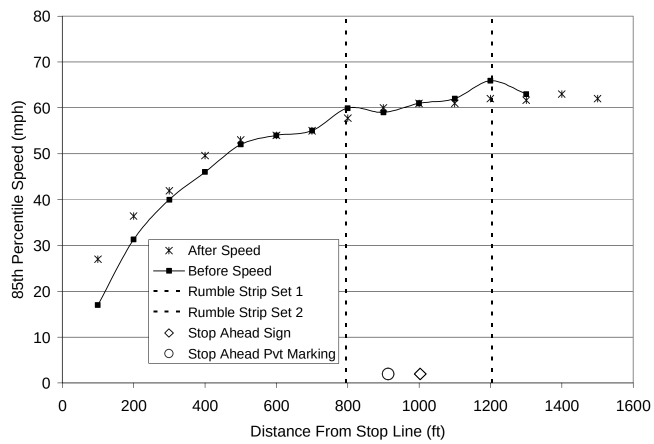
Using mean speeds at 100 ft (30.5 m) intervals from the stop line, the significance test was performed at a 0.05 level of significance for each individual site. Table 7-15 contains graphs of the 85th percentile speed profiles on 100 ft (30.5 m) intervals at each site, along with descriptions of noteworthy findings.

Table 7-15. Site-by-Site Comparison of Before and After Speed Profiles.

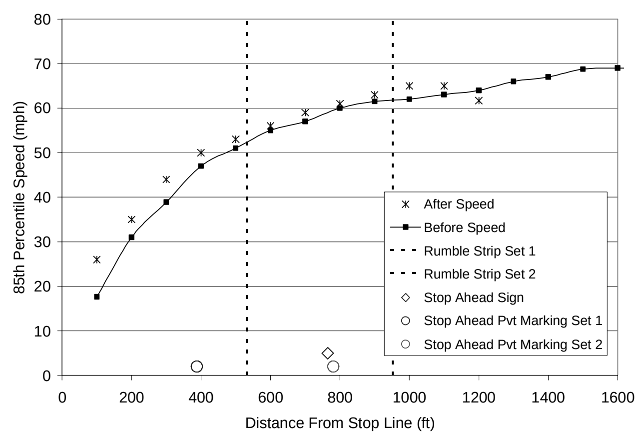
<table>
<tr><th>Findings</th><th>Graphs</th></tr>
<tr><td><u>FM 3326 at FM 1226</u>

Speeds in the after period were noticeably lower (4 mph) between the rumble strips. Inside 500 ft the two profiles are very similar. The difference in mean speed was found to be statistically significant only at 300 ft.</td><td>


</td></tr>
<tr><td><u>FM 2702 WB at US-277</u>

No considerable difference in speed between the rumble strips. After speeds are higher inside 500 ft, and the differential increases as the distance approaches zero. Differences in mean speeds were found to be statistically significant at 400 ft or less.</td><td>


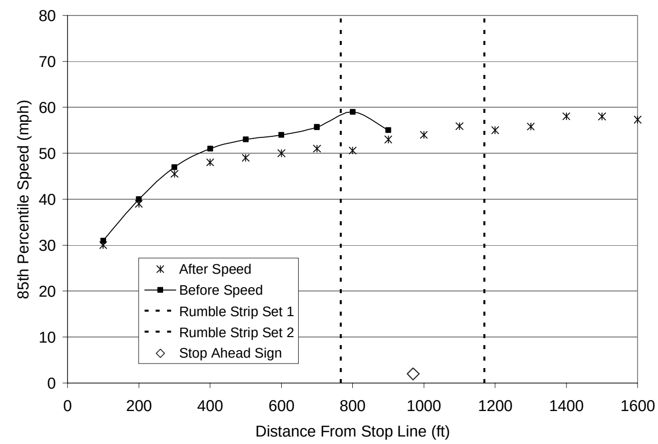
</td></tr>
<tr><td><u>SH 92 EB at US-277</u>

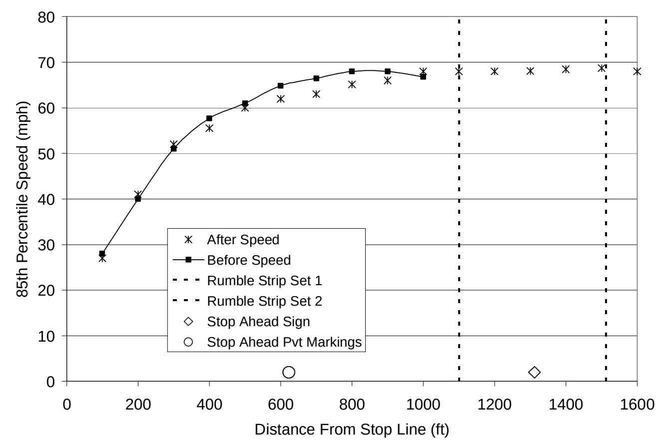
After speeds are slightly, but consistently, higher throughout the entire profile. Larger (and statistically significant) differences inside 500 ft.</td><td>


</td></tr>
</table>

Table 7-15. Site-by-Site Comparison of Before and After Speed Profiles (continued).

<table>
<tr><th>Findings</th><th>Graphs</th></tr>
<tr><td><u>FM 2728 at FM 429</u>

After speeds are substantially lower (8 mph) at the downstream rumble strip and somewhat lower throughout the remainder of the profile. The difference in mean speeds is significant only at 600-800 ft.</td><td>
</td></tr>
<tr><td><u>SH 78 at SH 160</u>

After speeds are nearly constant (68 mph) through the rumble strips and are slightly lower than before speeds downstream of the rumble strips. Only the 300 ft difference was significant.</td><td>
</td></tr>
<tr><td><u>FM 2514 at FM 1378</u>

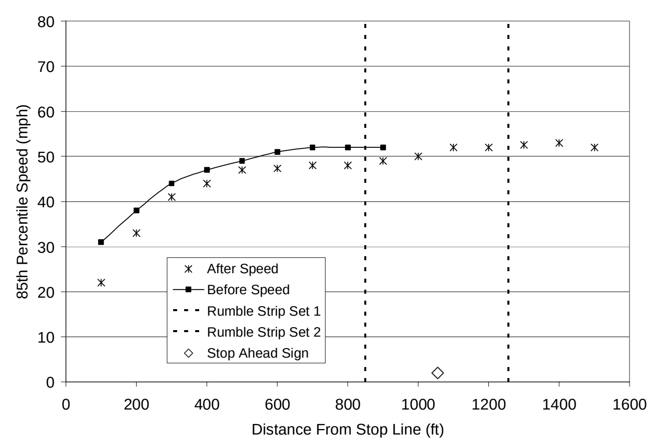
After speeds decreased by 5 mph from the upstream to the downstream rumble strips. After speeds are consistently lower than before speeds downstream of the rumble strips. The entire profile has a statistically significant difference in speed.</td><td>
</td></tr>
</table>

Table 7-15. Site-by-Site Comparison of Before and After Speed Profiles (continued).

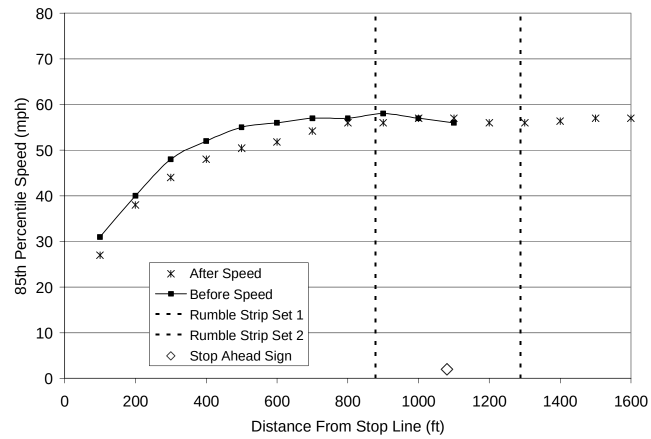
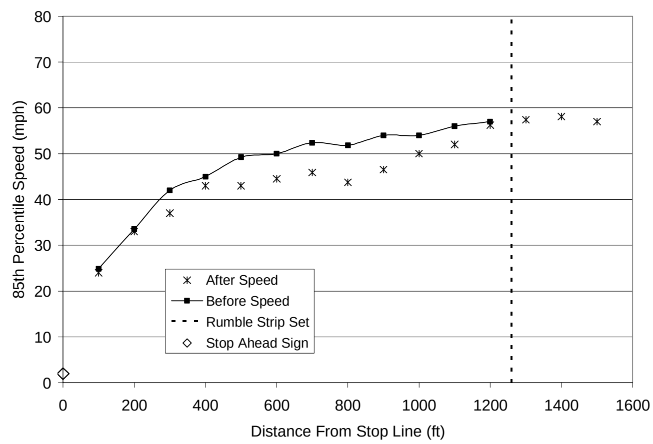
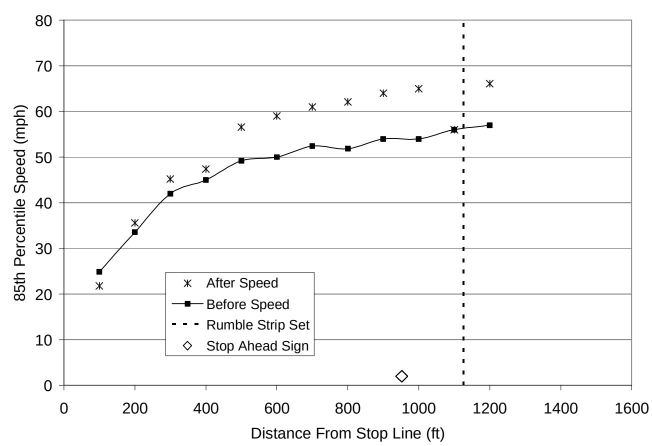
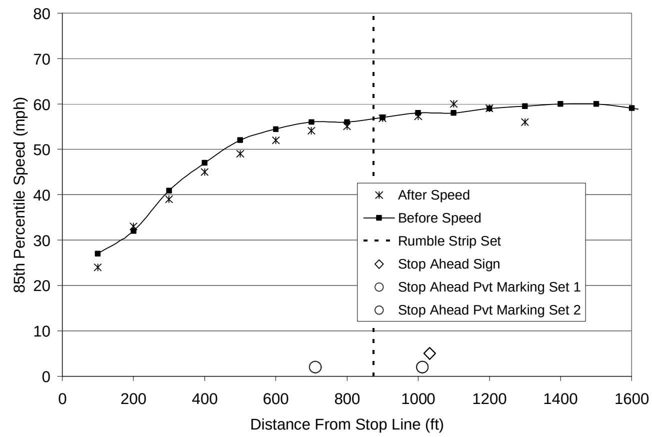
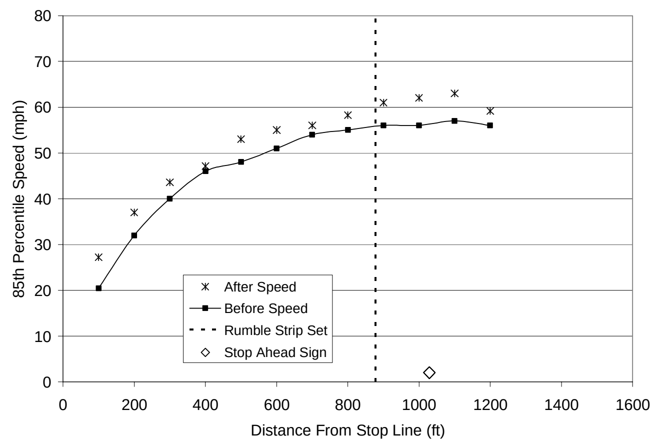
Findings	Graphs
<u>FM 2933 NB at FM 545</u> Speed profiles are similar between the rumble strips, but after speeds are about 4 mph lower than before speeds downstream of the strips. Speed difference is significant at 600 ft or less.	
<u>FM 219 EB at US-281</u> The after speed profile is noticeably lower than before, as much as 8 mph, at 500 ft or greater. However, the difference is much less inside 500 ft. Speed differences are significant from 1100 ft to 300 ft, except for 400 ft.	
<u>FM 219 WB at US-281</u> After speeds are consistently and substantially (~10 mph) higher at distances 500 ft and greater from the stop line. Hard braking inside 500 ft brings the after profile closer to the before profile. The differences in mean speeds are statistically significant at 1200-500 ft and at 200 ft.	

Table 7-15. Site-by-Site Comparison of Before and After Speed Profiles (continued).

<table>
<tr><th>Findings</th><th>Graphs</th></tr>
<tr><td><u>FM 215 NB at SH 36</u>

Before and after speeds are very similar throughout the profile, with after speeds slightly lower downstream of the rumble strips. Only at 100 ft was there a significant difference in speeds.</td><td>
</td></tr>
<tr><td><u>FM 215 SB at SH 36</u>

After speeds are consistently 2-6 mph faster than before speeds throughout the profile. The differences in mean speeds are significant at 1100 ft and from 900 to 100 ft.</td><td>
</td></tr>
<tr><td colspan="2">Note: 1 mph = 1.6 km/h; 1 ft = 0.305 m</td></tr>
</table>

The test for significance at each site revealed mixed results. Four of the 11 sites had statistically significant **increase** in speeds at a minimum of two 100 ft (30.5 m) increments:

- FM 2702 WB at 100, 200, 300, and 400 ft (30.5, 61.0, 91.4, and 121.9 m) upstream of the intersection;
- SH 92 EB from 100 to 500 ft (30.5 to 152.4 m) upstream of the intersection;
- FM 219 WB at 200 ft (61.0 m), and between 500 and 1200 ft (152.4 and 365.8 m) upstream of the intersection; and
- FM 215 SB at 100 to 900 ft (30.5 to 274.3 m) and at 1100 ft (335.3 m) upstream of the intersection.

The before and after speeds at three of the sites either had only one location or no locations with a significant change in speed:

- FM 3326,
- SH 78, and
- FM 215 NB.

The remaining four sites all had statistically significant **decreases** in speeds at a minimum of two 100 ft (30.5 m) increments:

- FM 2728 at 600, 700, and 800 ft (182.9, 213.4, and 243.8 m) upstream of the intersection;
- FM 2514 for the entire speed profile (100 to 900 ft [30.5 to 274.3 m]) upstream of the intersection;
- FM 2933 at 100 to 600 ft (30.5 to 182.9 m) upstream of the intersection; and
- FM 219 EB at 300 ft (91.4 m), and 500 to 1100 ft (152.4 to 335.3 m) upstream of the intersection.

Aggregate Comparisons

Because of the differences in location and number of rumble strip patterns, all 11 sites were not combined for an aggregate test. Those five sites with two sets of staggered rumble strips where the downstream set of strips was between 750 and 900 ft (228.6 and 274.3 m) from the stop line were combined (FM 3326, FM 2702 WB, FM 2728, FM 2514, and FM 2933).

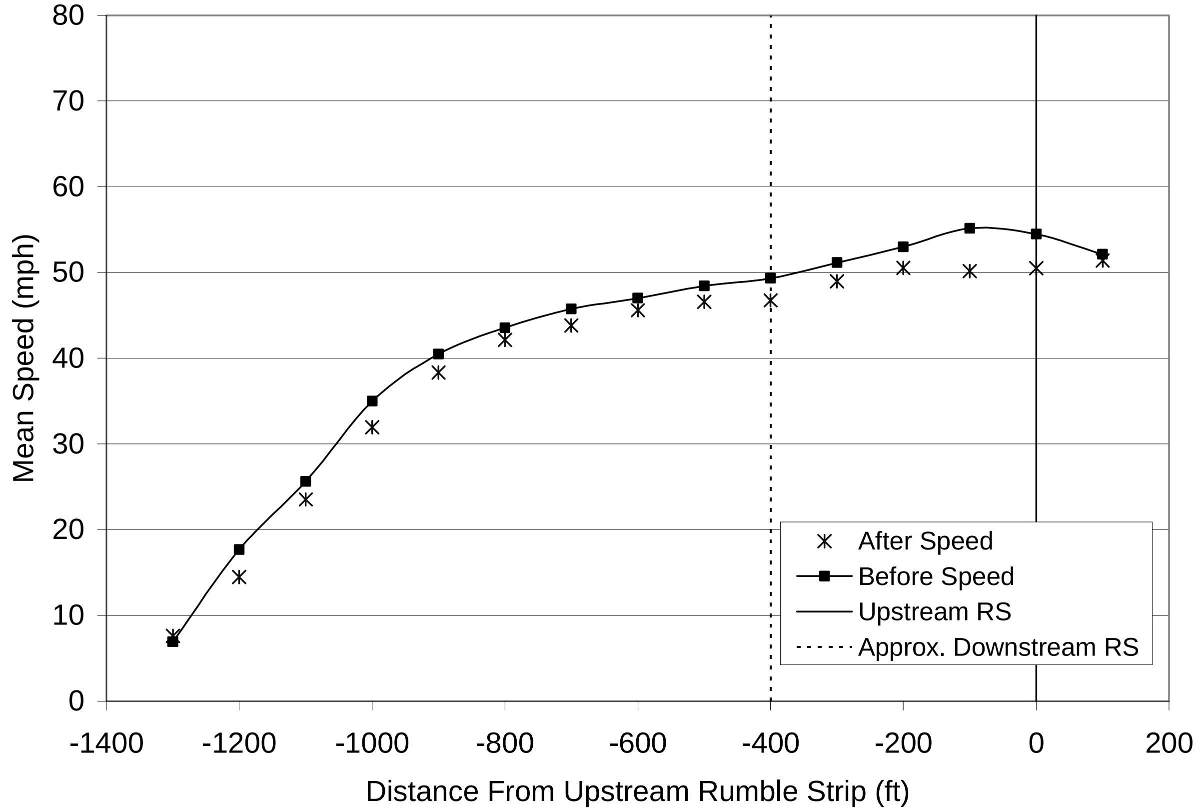
These sites from the Abilene and Dallas areas provided a more homogeneous sample of sites to use as a basis for testing. The results from testing mean speeds from the subset revealed that all of the differences in speed greater than 1.0 mph (1.6 km/h) were statistically significant at both the 0.05 and 0.01 levels of significance. Table 7-16 shows the variables and results for this aggregate test, and Figure 7-2 illustrates the mean speeds graphically, with distances adjusted to the upstream rumble strip.

Table 7-16. Results of Test for Statistical Significance.

Distance from Rumble Strip (ft)*	Number of Observations Before	Number of Observations After	Standard Deviation Before (mph)	Standard Deviation After (mph)	Mean Before (mph)	Mean After (mph)	Mean Difference (mph)	Significant at 0.05?
-1300	44	40	2.0	2.9	6.9	7.6	-0.7	**N/A**
-1200	369	378	6.9	6.4	17.7	14.5	3.2	**Yes**
-1100	628	529	9.1	8.3	25.6	23.5	2.1	**Yes**
-1000	537	496	7.2	7.9	35.0	31.9	3.0	**Yes**
-900	540	474	6.5	6.6	40.5	38.3	2.2	**Yes**
-800	573	501	6.7	6.4	43.5	42.1	1.4	**Yes**
-700	620	483	6.5	6.9	45.7	43.8	2.0	**Yes**
-600	617	558	7.3	7.4	47.0	45.6	1.4	**Yes**
-500	629	563	7.2	7.8	48.4	46.6	1.8	**Yes**
-400	514	600	7.6	8.0	49.3	46.7	2.6	**Yes**
-300	306	557	8.9	7.9	51.1	49.0	2.2	**Yes**
-200	218	515	9.9	8.3	53.0	50.5	2.5	**Yes**
-100	148	542	10.3	8.0	55.2	50.2	5.0	**Yes**
0	101	532	10.1	8.0	54.5	50.5	4.0	**Yes**
100	48	492	12.0	8.1	52.1	51.4	0.7	**N/A**

*A range of ± 25 ft. Note: 1 mph = 1.6 km/h; 1 ft = 0.305 m

Figure 7-2 illustrates that the greatest difference between before and after mean speeds is found between the two sets of rumble strips, as much as 5 mph (8 km/h). Downstream of the rumble strips, the difference is not as large (between 1 and 2 mph [1.6-3.2 km/h]), until about 1000 ft (305 m), when the difference increases again to about 3 mph (4.8 km/h).



(1 mph = 1.61 km/h, 1 ft = 0.305 m)

Figure 7-2. Comparison of Mean Speeds Relative to Upstream Rumble Strip (RS).

CONCLUSIONS

An analysis of the speed data collected at the study sites reveals small changes in mean and 85th percentile speed on approaches with rumble strips. In some cases the change was an increase in speeds and at other sites the change was a decrease in speeds on the approach to the intersection. Generally, the speed changes were less than 4 mph (6.4 km/h), with most being on the order of 1 to 2 mph (1.6 to 3.2 km/h). Statistical tests revealed that many, but not all, differences in mean speeds at each site were statistically significant at the 95th percent level of confidence.

An analysis was performed on a subset of five sites that had similar rumble strip installations. The speed data were consolidated using the first rumble strip that a driver would encounter as the common point between the five sites. For each location along the speed profile with a difference in speed greater than 1.0 mph (1.6 km/h), which is the limit of the speed measuring equipment, speeds were lower in the after period. The decrease in speeds ranged from 1.4 to 5.0 mph (2.3 to 8.0 km/h). The statistical test found that all differences in mean speeds were statistically significant for the speed profile.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

The research for this project investigated safety measures for intersections on rural highways. Recent findings in the literature were used to develop material on rural intersection safety that was included as Chapter 6 of the TxDOT report *Treatments for Crashes on Rural Two-Lane Highways in Texas* (FHWA/TX-02/4048-2, May 2002). Field studies were conducted to collect data on the performance of in-lane rumble strips on approaches to intersections and left-turn driving behavior.

CONCLUSIONS

Specific conclusions from the research include the following:

Crashes

- Department of Public Safety data for the year 2000 shows that 37 percent of rural crashes are intersection, intersection-related, or driveway-related.
- Of the 2481 rural near or at intersection/driveway crashes, 43 percent occurred at T-intersections. Treatments that inform the driver of the presence of a T-intersection should address some of these crashes.

Left-Turn Lanes

- Several methods are available for determining when to include a left-turn lane in the design at an intersection. The most widely accepted approach is a procedure first proposed by Harmelink in 1967.
- Several of the assumptions used by Harmelink appear to need updating with more current data and expanded for use with 70 mph (112 km/h) highways. Using data available from other research efforts and from this study, the guidelines in the *Green Book* Exhibit 9-75 and TxDOT *Roadway Design Manual* Table 3-11 were updated and expanded to include 30 (48 km/h) and 70 mph (112 km/h) roadways. The suggested guidelines are included in Table 5-5 of this report. An example of the change for a 50 mph (80 km/h) roadway with 200 vehicles coming from the opposing direction and 10 percent left turn is that the guidelines would suggest a left-turn lane if there were 268 advancing vehicles rather than 400 advancing vehicles.

Left-Turn Behavior

- Data were collected at six rural T-intersections to obtain a better understanding of left-turn driver behavior in Texas. At each intersection the minor road was controlled by a Stop sign. The width of the shoulder on the major road ranged between no shoulder and a 10 ft (3.0 m) shoulder.

- When a wide level shoulder was provided, a large percentage of the drivers, up to 95 percent, drove on the shoulder. At the sites where the shoulder was retrofitted and widened from 3 ft (0.9 m) to 10 ft (3.0 m) just prior to the intersection, only 20 to 30 percent of the drivers used the shoulder. At the site with minimum paved shoulder, none of the recorded drivers used the shoulder (although the number of drivers in this situation was low, on the order of 1 to 3 drivers per hour).
- Shoulder width and type also appears to influence the speeds at which the movements are performed. At the sites with the wider shoulders, higher speeds were recorded. At the site with the retrofitted shoulder, a lower speed was measured.
- At the site converted from a four-lane cross section to a three-lane cross section, the following were observed:
    - Left-turning vehicles entered the two-way left-turn lane using a more gradual deceleration rate as compared to how they slowed in anticipation of their turn when in the through lane.
    - The overall speed on the rural highway decreased slightly from 73 mph (117 km/h) to 69 mph (111 km/h). The posted speed limit is 65 mph (105 km/h).

In-Lane Rumble Strips

- Previous research on in-lane rumble strips has shown the following regarding installations:
    - They are not always proven to be effective in reducing crashes and many of the reports were inconclusive.
    - They do result in a small reduction in vehicle speeds. Some vehicles are slowed more than others, however, and it appears that speed variance on the intersection approach may be increased.
    - They generally increased the proportion of drivers who made a full stop.
- Installations at 14 sites in Texas found the following:
    - Characteristics of installation varied with respect to distance from the stop line and Stop Ahead sign, staggered or parallel configuration, and number of sets of strips.
    - Differences in 85^{th} percentile speeds before and after installation were on the order of 1 to 2 mph (1.6 to 3.2 km/h).
- A test for statistical significance on five similar sites revealed the following:
    - Differences in mean speeds were statistically significant at the 95 percent level of confidence for all distances 100 ft (30 m) upstream to 1200 ft (366 m) downstream of the upstream set of rumble strips.
    - The largest difference in mean speeds occurred between the two sets of rumble strips.
    - The next largest difference in mean speeds occurred approximately 300 ft (91 m) before the stop line.

RECOMMENDATIONS

Recommendations based upon the research include the following:

- Adopt new left-turn lane installation guidelines. The guidelines could either be those generated in this project or based on a more comprehensive study. A new project could also demonstrate the cost savings of the installation and could investigate low-cost left-turn lane guidelines that would use some of the shoulder for the left-turn lane.
- Encourage the use of rumble strips at locations where drivers need additional warning of the downstream intersection.
- Conduct a safety study on the in-lane rumble strip installations.
- Refine the current TxDOT draft standards for in-lane rumble strips to encourage consistent use.

REFERENCES

1. Intersection Safety Brief. Federal Highway Administration, National Agenda for Intersection Safety, May 2002.
2. Kalakota, K., and P. Seneveratne. “Accident Prediction Models for Two-Lane Rural Highways.” North Dakota State University, Bismarck, ND, May 1994.
3. Stokes, R.W., M.J. Rys, E.R. Russell, R.K. Robinson, and B. Budke. *Analysis of Rural Intersection Accidents Caused by Stop Sign Violation and Failure to Yield the Right-Of-Way*. Report No. K-TRAN: KSU-98-6, Final Report, Kansas State University, Manhattan, KS, September 2000.
4. Insurance Institute for Highway Safety (IIHS), Safety Report, September 2001.
5. Eck, R.W., and G. Winn. “Older Driver’s Perceptions of Problems at Unsignalized Intersections on Divided Highways.” *TRB 2002 Annual Meeting CD-ROM*, Original Paper Submittal, Submitted July 2001.
6. Staplin, L., K. Lococo, and S. Byington. *Older Driver Highway Design Handbook*, FHWA-RD-97-135, Federal Highway Administration, McLean, VA, January 1998.
7. McKnight, J.A. “Driver and Pedestrian Training.” *Transportation in an Aging Society*, Transportation Research Board Special Report 218, National Research Council, Washington, D.C., 1988. pp. 101-133.
8. Fitzpatrick, K., A. Parham, and M. Brewer. *Treatments for Crashes on Rural Two-Lane Highways in Texas.* FHWA/TX-02/4048-2, Texas Transportation Institute, College Station, TX, April 2002.
9. Walton, S., and E. Meyer. “The Effect of Rumble Strip Configuration on Sound and Vibration Levels.” *ITE Journal*, December 2002. pp. 28-32.
10. *Manual on Uniform Traffic Control Devices*. (Millennium Edition—Revised December 2001). U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., 2001.
11. Corkle, J., M. Marti, and D. Montebello. *Synthesis on the Effectiveness of Rumble Strips.* Minnesota Local Research Board Report No. MN/RC-2002-07, Minnesota Department of Transportation, St. Paul, MN, October 2001.
12. Zaidel, D., A.S. Hakkert, and R. Barkan. “Rumble Strips and Paint Stripes at a Rural Intersection.” *Transportation Research Record 1069*, Transportation Research Board, National Research Council, Washington D.C., September 1986. pp. 7-13.
13. Meyer, E. “Guidelines for the Application of Removable Rumble Strips.” Report No. KDOT Project RE-0286-01/K-TRAN Project KU-02-03, Final Report, University of Kansas, Lawrence, KS, March 2003.
14. Harwood, D.W. “Use of Rumble Strips to Enhance Safety.” *Synthesis of Highway Practice 191*, National Cooperative Highway Research Program, National Academy Press, Washington, D.C., 1993.
15. Higgins, J., and W. Barbel. “Rumble Strip Noise.” *Transportation Research Record 983*, Transportation Research Board, National Research Council, Washington, D.C., February 1985. pp. 27-36.
16. Rumble Strips. Federal Highway Administration. Available Online. <http://safety.fhwa.dot.gov/fourthlevel/pro_res_rumble.library.htm>. Accessed: July 2003.
17. Bellis, W.R. “Development of an Effective Rumble Strip Pattern.” *Traffic Engineering*, April 1969, pp. 22-25.

18. Florida Policy Specification Drawings. January 1998. Available Online. <http://safety.fhwa.dot.gov/fourthlevel/pdf/Fl.pdf>. Accessed: July 2003.
19. CalTrans Traffic Manual. *Markings, Special Pavement Treatments 6-03*. 7-1996. Available Online. <http:www.dot.ca.gov/hq/traffops/signtech/signdel/chp6.htm>. Accessed: July 31, 2003.
20. "Rumble Strips at the Beginning of Work Zone to Get Driver Attention." Ohio website (no date). Available Online. <http://ops.fhwa.dot.gov/wz/wzguidbk/Documents/hp-oh11.htm>. Accessed: March 4, 2003.
21. Korfhage, D. *Special Rumble Seal Coat Strips for Traffic Warning*. Special Study No. 279, Final Report, Minnesota Department of Highways, St. Paul, MN, 1968.
22. Mason, D.F. "An Examination of the Use and Effectiveness of Intersection Rumble Strips in New Brunswick." Paper presented at the Community Traffic Solutions for the Future Session of the 1999 Annual Conference of the Transportation Association of Canada, Saint John, New Brunswick, 1999.
23. Kermit, M.L., and T.C. Hein. "Effect of Rumble Strips on Traffic Control and Drive Behavior." *Traffic and Operations*, Highway Research Board, Washington, D.C., 1962. pp. 469-481.
24. Harder, K.A., J. Bloomfield, and B. Chihak. *The Effects of In-Lane Rumble Strips on the Stopping Behavior of Alert Drivers.* Human Factors Research Report No. MN/RC-2002-11, Minnesota Department of Transportation, St. Paul, MN, October 2001.
25. Storm, R. *Pavement Markings and Incident Reduction*. Presented at the MTC Transportation Scholars Conference, Center for Transportation Research and Education, Iowa State University, Ames, IA, 2000.
26. Harwood, D.W., F.M. Council, E. Hauer, W.E. Hughes, and A. Vogt. *Prediction of the Expected Safety Performance of Rural Two-Lane Highways*. FHWA-RD-99-207, Federal Highway Administration, September 2000.
27. Harwood, D.W., K.M. Bauer, I.B. Potts, D.J. Torbic, K.R. Richard, E.R. Kohlman Rabbani, E. Hauer, and L. Elefteriadou. *Safety Effectiveness of Intersection Left- and Right-Turn Lanes*. FHWA-RD-02-089, Federal Highway Administration, July 2002.
28. *Traffic Engineering Handbook*, 5th Edition. Institute of Transportation Engineers. 1999.
29. Harmelink, M. "Volume Warrants for Left-Turn Storage Lanes at Unsignalized Grade Intersections," *Highway Research Record 211*, Highway Research Board, National Research Council, Washington, D.C., 1967.
30. American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets*, 2001.
31. Texas Department of Transportation. *Roadway Design Manual*. http://manuals.dot.state.tx.us/dynaweb/coldesig/rdw/@Generic__BookView. October 2002. Accessed: August 29, 2003.
32. American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets* (1984).
33. American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets* (1990).
34. American Association of State Highway and Transportation Officials, *A Policy on Geometric Design of Highways and Streets* (1994).

35. Neuman, T. *Intersection Channelization Design Guide*, NCHRP Report 279, AASHTO, FHWA, Transportation Research Board, Washington, D.C., November 1985.
36. Oppenlander, J.C., and J.C. Bianchi. "Guidelines for Left-Turn Lanes." *ITE Compendium of Technical Papers, 60th Annual Meeting, Orlando, Florida*. August 1990.
37. Koepke, F. *Access Management Guidelines for Activity Centers*, NCHRP Report 348, Transportation Research Board, National Research Council, Washington, D.C., 1992.
38. Modur, S., R. Machemehl, and C. Lee. "Criteria for the Selection of a Left-Turn Median Design," Research Report 1138-1F, University of Texas, Austin, TX, January 1990.
39. Hawley, P., and V. Stover. *Guidelines for Left-Turn Bays at Unsignalized Access Locations, Second National Access Management Conference*, Colorado Department of Transportation, Federal Highway Administration, Transportation Research Board, Washington, D.C., 1996.
40. Bonneson, J., and M. Fontaine. *Engineering Study Guide for Evaluating Intersection Improvements*, NCHRP Report No. 457. Available Online. <http://trb.org/trb/publications/nchrp/esg.pdf> Accessed: August 2003. Transportation Research Board, Washington, D.C., 2001.
41. *Roadway Design Manual*, Mississippi Department of Transportation, January 2001. Available Online. <http://www.gomdot.com/business/roadwaydrawings/manual.htm>. Accessed: February 18, 2003.
42. *Highway Design Manual*, New York State Department of Transportation, August 2001. Available Online. <http://www.dot.state.ny.us/cmb/consult/hdmfiles/hdm.html>. Accessed: February 18, 2003.
43. *Project Development Manual*, Missouri Department of Transportation, February 2003. Available Online. <http://www.modot.state.mo.us/design/ppdm/files/start.pdf>. Accessed: February 18, 2003.
44. *Location and Design Manual, Volume 1*, Ohio Department of Transportation, November 2002, Available Online. <http://www.dot.state.oh.us/roadwayengineering/L&D%20Vol1_link.htm>. Accessed: February 21, 2003.
45. *Roadway Design Manual of Instruction*, State of Utah Department of Transportation, July 1998, Available Online. <http://www.udot.utah.gov/esd/Manuals/Roadway/rw-design1.htm>. Accessed: February 21, 2003.
46. Idaho Transportation Department, *Traffic Manual*, Idaho Transportation Department, Boise, Highway Approaches, August 1994.
47. Harwood, D.W., J.M. Mason, R.E. Brydia, M.T. Pietrucha, and G.L. Gittings. *Intersection Sight Distance*. NCHRP Report 383. Washington, D.C., 1996.
48. *TRB Special Report 209: Highway Capacity Manual*, 3rd ed., Transportation Research Board, Washington, D.C., 1994.
49. Miscky, R.J., and J.M. Mason, Jr. "Sight Distance for Vehicles Turning Left off Major Roadway" *1996 National Conference on Access Management, Vail, Colorado.* August 1996.
50. "PTV Planung Transport Verkehr AG (PTV AG)." *Vissim 3.60 User Manual.* Karlsruhe, Germany, December 2001. (Distributed in North America by Innovative Transportation Concepts, LLC, Corvallis, OR.)
51. Fitzpatrick, K., L. Elefteriadou, D.W. Harwood, J.M. Collins, J. McFadden, I. Anderson, R. A. Krammes, N. Irizarry, K.D. Parma, K.M. Bauer, and K. Passetti. *Speed Prediction for Two-Lane Rural Highways.* Report FHWA-RD-99-171. Washington, D.C., 2000.