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EFFECT OF RIGHT-TURNING VEHICLES ON TRAFFIC SIGNAL VOLUME WARRANTS

Final Report

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16. Abstract

This study reports on two methods used to determine the significance of variation in the percent of right turns on side street average vehicular delay at two-way stop sign controlled intersections.

The first method utilized the Traffic Experimental and Analytical Simulation (TEXAS) microcomputer model to evaluate the right turn factor on side street delay for various hypothetical geometric configurations and volume levels. Statistical analyses of the results revealed that the right turn factor was not significant for any of the scenarios tested. However, the validity of the findings was questionable, as the model was highly sensitive to the random seed number combinations used to generate the simulated traffic stream.

The second method involved a statistical analysis of an available set of field data collected at stop sign controlled intersections. The data were divided into two groups based on the number of lanes on the side street. A regression procedure revealed that the right turn percentage was significant in predicting delay for the case of two lanes on the side street, but not for the one lane case.

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CHAPTER 1

INTRODUCTION

In order to determine the necessity for the installation of a traffic signal at an intersection, a comprehensive investigation of both the traffic conditions and the physical characteristics of the location is required. Such traffic related factors as vehicular volumes, headways, turning movements, travel speeds and such physical factors as number and configuration of lanes, channelization, sight distance restrictions, and vicinity of other signals influence the performance of the intersection.

The Manual on Uniform Traffic Control Devices for Streets and Highways (MUTCD), published by the Federal Highway Administration, contains 11 warrants to serve as a guide in determining the need for traffic signals. Rather than serving as absolute criteria, the use of the warrants should be tempered with professional judgment based upon experience and consideration of all related factors (Institute of Transportation Engineers 1982).

The effect of right turning vehicles from the minor street on the application of the signal warrants is one traffic factor that the MUTCD defers to engineering judgment. The wording contained on page 4C-3 in the 1988 edition of the manual states:

> *The analysis should consider the effects of the right turn vehicles from the minor street approaches. Engineering judgment should be used to determine what, if any, portion of the right turn traffic is subtracted from the minor street traffic count when evaluating the count against the above warrants.*

While sound engineering judgment is essential in the evaluation of whether a traffic signal is needed at a candidate intersection, the lack of numerical standards is not conducive to national uniformity and can lead to unwarranted signals.

When a traffic signal is installed at an intersection where it is not warranted, it can lead to an increase in overall intersection delay, an increase in accident frequency, a disregard for signal indications, and circuitous travel by alternative routes (Institute of Transportation Engineers 1982). The consequence is an increase in travel time, an increase in fuel consumption and vehicle wear for motorists, and an unnecessary burden on taxpayers to pay for the installation and maintenance of the signal.

1.1 Concept of Gap Acceptance

The rationale for deducting right turning volume from the total approach volume on the minor street is based on the concept of gap acceptance. As the movement being executed becomes more complex, drivers will require longer gaps through which to make their maneuver (Transportation Research Board 1985). The required gap for a right turn from the minor street involves only one direction of flow on the major street. In order to execute a left-turn or through movement, however, acceptable gaps are required in both directions of the major street. When considering both directions of flow, proportionally fewer acceptable gaps would be expected than for one direction only.

A gap is defined as the elapsed time between arrival of successive vehicles on the main street at a specified point in the intersection area (i.e. the driver at the stop bar) (Desrosiers 1966). A gap is accepted if the vehicle on the minor street crosses and/or merges with the major street flow. Because different drivers will accept different minimum gaps under identical conditions, gap acceptance values are usually expressed in terms of critical gaps. A critical gap is defined as the median gap, or one in which 50 percent of the drivers will accept and 50 percent will reject.

1.2 Definition of Delay

Related to gap acceptance is vehicle delay. Delay occurs as a consequence of vehicles waiting for an acceptable gap to enter an intersection from a stop sign controlled approach. There are three types of delay that can be used as a measure of effectiveness at intersections: total or overall delay, average stopped delay and average queue delay. Total delay is defined as the actual travel time through the intersection minus the travel time that would have been required had the driver been able to maintain his desired speed throughout the intersection area. This type of delay has the most significance when comparing two types of intersection control (Lewis and Michael 1963).

Average stopped delay is defined as the total amount of time vehicles are stopped in a queue waiting to enter an intersection, divided by the total number of vehicles. Stopped delay is the preferred statistic at signalized intersections because it does not include move-up time to the intersection after the signal turns green (Lee, Rioux, and Copeland 1977). Average delay is used instead of total delay because it allows traffic flows to be compared at various volume levels.

Average queue delay is defined as the total amount of time vehicles are in a queue waiting to enter the intersection, divided by the total number of vehicles. Queue delay is the preferred statistic at unsignalized intersections because it includes move-up time (Lee, Rioux, and Copeland 1977). However, due to the difficulty in measuring queue delay in the field, stopped delay is often used as the parameter when measuring the performance of intersections controlled by stop signs.

1.3 Traffic Signal Warrants

The use of delay as the measurement of effectiveness in evaluating intersection performance is supported by the MUTCD traffic signal warrants. While only one of the 11 warrants included in the most recent edition of the MUTCD specifically uses delay as the input parameter, the four vehicular volume-based warrants indirectly use delay as a measure (Federal Highway Administration 1986). This is because delay was the most important factor in the determination of these volume warrants (Lewis and Mitchell 1963).

The four MUTCD volume-based warrants are:

- Warrant No. 1, Minimum Vehicular Volume
- Warrant No. 2, Interruption of Continuous Traffic
- Warrant No. 9, Four Hour Volumes
- Warrant No. 11, Peak Hour Volume

Generally, the goal is to install the type of traffic control device that results in the minimum delay to motorists. To facilitate this goal, the MUTCD warrants are designed to be easy to understand and easy to apply. A minimum amount of data collection is required at a candidate intersection, as the warrants are intended to be used on a regular basis by jurisdictions of all sizes, including those with limited human and economic resources (Box and Alroth 1968).

1.4 Research Objective

Before a comprehensive, objective set of numerical standards can be developed for treating the effects of right turning vehicles in the application of the MUTCD traffic signal warrants for stop sign controlled intersections, a better understanding of this factor is required. The primary objective of this study was to evaluate the effect of right turning traffic on side street vehicular delay at two-way stop sign controlled intersections, and to determine the most suitable method for developing a set of numerical standards.

1.5 Scope of Study

Three independent methods were employed to accomplish the objectives of this study:

- Review traffic engineering literature to identify previous studies related to the topic and report the findings.
- Simulate traffic at hypothetical intersections under controlled conditions using the TEXAS model and determine the significance of right turning vehicles on side street delay
- Analyze a set of existing field data collected at stop sign controlled intersections for the effect of right turning vehicles on average delay for the study approach

The first method involved reviewing previous research in three areas related to two-way stop sign control: gap acceptance, delay, and the relationship between critical gap and delay. Also, a review of current practices used by various jurisdictions in treating right turning vehicles in the application of traffic signal warrants was conducted.

The second method utilized TEXAS, a microcomputer traffic simulation model for isolated intersections, to conduct a thorough analysis of right turns under a variety of test conditions.

Included in this task was the determination of optimum testing conditions and the significance of various geometric and traffic factors in the operation of the model.

The third method involved conducting statistical analyses on an available set of intersection empirical data collected in six cities around the country as part of a National Cooperative Highway Research Program study (Transportation Research Board 1982). While the data were collected for a variety of intersections under stop sign and signal control, the analysis for this study was limited to intersections with four approaches and two-way stop control.

CHAPTER 2

BACKGROUND LITERATURE

Intersections controlled with stop signs have been of interest to traffic engineers for much of the twentieth century. Relevant studies related to this type of intersection traffic control date back to the late 1940's (Box and Alroth 1968). Despite the long history of interest, there are many aspects of the topic that are not well understood. The effect of right turns on the side street traffic stream is one such aspect.

A study by Linesman (1966) identified more than 30 factors which can affect performance at stop sign controlled intersections. These include traffic, vehicle, driver, and physical characteristics. The fact that an interaction exists between many of these factors further complicates the picture.

Undeniably, the percent of right turns from a stop sign approach is but one factor out of many interrelated factors in the dynamics of traffic flow through an intersection. Therefore, developing a comprehensive understanding of this factor requires one or two central measures which include as many of the other variables as possible.

This chapter includes a review traffic engineering literature related to the two most appropriate central measures: critical gaps and vehicle delay. Furthermore, two studies that established the relationship between these two parameters are presented.

2.1 Gap Acceptance Studies

Early work on the subject of gap acceptance at stop controlled intersections was performed by Erickson, Greenshields, and Schaperio (1947). The average acceptable gap was found to be 6.1 seconds for through movements and 4.1 seconds for right turn movements at an intersection with restricted sight distance.

Raff (1950) developed the concept of critical lag based on the study of four intersections in New Haven, Connecticut. A lag is defined as that portion of a gap which remains when a vehicle on the minor street arrives at the intersection (Ehle 1967). At one intersection, Raff isolated right turns made by drivers from the minor street and found their critical lag to be approximately 20 percent less than the the critical lag for drivers either crossing the major street or turning left.

A number of more recent studies on critical gaps at stop control intersections have been conducted in the United States, as well as other countries. Table 2-1 includes selected studies that measured right turn critical gaps separate from through and left critical gaps. All of the studies listed were conducted within the last 30 years. Of note are some of the common findings of the studies included in Table 2-1. Most apparent is the trend for right turn critical gap values to be less than critical gap values for through and left turns. Some of the studies concluded that gap acceptance varied with major street speed, rural versus urban locations, and peak hour versus off-peak. Additional factors that a few of the studies reported as significant include: sight distance availability, the width and number of lanes on the major street, and other geometrics of the intersection (curb radii, presence of a right turn acceleration lane, median, etc.).

Another publication that addresses the difference in critical gaps for different turning movements is the Highway Capacity Manual (Special Report 209), published by the Transportation Research Board (1985). As part of a procedure to estimate capacity and level-of-service at unsignalized intersections, the manual uses the concept of gap acceptance. The analysis technique is adapted from a German method originally published in 1972. Critical gap lengths were modified, based on a limited number of validation studies, to reflect conditions in the United States (Zegeer 1988).

Out of a list of five factors, the HCM identifies the type of turning movements from the side street as the most significant. The other factors are: the type of control (stop or yield), the average running speed on the major street, the number of lanes on the major street, and the geometrics and environmental conditions at the intersection.

TABLE 2-1. CRITICAL GAPS REPORTED IN PREVIOUS STUDIES

AUTHOR(S) OF STUDY	LOCATION AND YEAR	VARIABLE	VALUE	AVERAGE CRITICAL GAP BY TURNING MOVEMENT, SEC.		
				RIGHT	THROUGH	LEFT
Bakare &	Illinois					
Jovanis	1984	None	-	5.4	6.4	
Bissel	California		-			
	1960	None	-	5.4	5.8	6.2
Brilon	Germany	Speed				
	1988	(km/hr)	40	5.03	5.07	5.58
			50	5.75	5.80	6.38
			60	6.48	6.53	7.18
			70	7.20	7.26	7.99
			80	7.93	7.99	8.79
			90	8.65	8.72	9.59
Erickson,	Connetic.					
et. all.	1947	None	-	4.1	6.1	-
Hanson	Sweden	Lanes on				
	1978	Main Street	2	6.5	6.7	6.6
			4	6.2	6.9	7.5
			6	3.7	5.3	5.2
Jirava &	Czechslo-	Urban vs.	Urban	6.0	7.0	7.5
Karlicky	vakia 1988	Rural	Rural	6.5	8.0	8.5
Radwan &	Indiana	No. of	1	6.73	7.90	6.32
Sinha	1980	Maneuvers	2	-	7.21	6.60
		for				
Solberg &	Indiana	Size of	Large	7.38	7.06	8.02
Oppenlan-	1966	Urban Area	Small	7.33	7.43	7.71
der						
Wagner	Michigan	Peak vs.	Peak	6.5	7.1	6.2
	1966	Off-Peak	Off-Peak	7.2	7.5	7.7

The Highway Capacity Manual procedure for calculating capacity at unsignalized intersections involves a two-step process to determine the appropriate critical gap. First, the basic critical gap size is selected from a table based on the type of movement, the type of control, and major street speed. The second step requires the user to adjust the basic gap value for various geometric conditions and population of the study area. The HCM critical gap criteria are reproduced in Table 2-2.

2.2 Delay Studies

Drivers are more conscious of delay than any other element at an intersection (Box and Alroth 1967). Consequently, a great number of studies have been conducted to evaluate delay at stop sign controlled intersections. The literature related to delay can be categorized into three general types: field studies, simulation studies, and studies that synthesize field data with simulation.

One of the earliest field studies that measured delay at stop control intersections was conducted by Raff in 1950. Raff studied side street delay on two-way stop intersections for a range of volume levels in an attempt to develop a volume warrant for stop signs. As a result of the scattered and inconsistent nature of the delay data, however, it was concluded that additional field research was needed. This inconsistency was attributed in part to the effect of turning movements on driver behavior.

Volk (1956) conducted field measurements of delay at intersections which had several types of traffic control, including 18 controlled by two-way stop signs. He plotted average delay per vehicle against major street volumes and developed linear regression lines. The coefficients of correlation were high for plots of average delay against major street volume (0.68 to 0.91) for the different lane configurations. Plots of delay against minor street volume had a much lower correlation range over the different lane configurations (0.25 to 0.60).

Volk concluded that major street traffic volume had a greater impact than minor street volume on average delay for the vehicles on the stop control approaches (minor street). The effect of turning movements was not considered in the study.

TABLE 2-2. HIGHWAY CAPACITY MANUAL CRITICAL GAP CRITERIA FOR UNSIGNALIZED INTERSECTIONS.

Basic Critical Gap for Passenger Cars, Sec.				
Vehicle Maneuver and Type of Control	Average Running Speed, Major Road			
	30 mph		*55 mph*	
	Number of Lanes on Major Road			
	2	*4*	*2*	*4*
Minor Road RT Stop	5.5	5.5	6.5	6.5
Yield	5.0	5.0	5.5	5.5
Major Road LT	5.0	5.5	5.5	6.0
Cross Major Rd Stop	6.0	6.5	7.5	8.0
Yield	5.5	6.0	6.5	7.0
Minor Road LT	6.5	7.0	8.0	8.5
	6.0	6.5	7.0	7.5

Adjustments and Modifications to Critical Gap, Sec.	
RT from Minor Street: Curb Radius > 50 ft or Turn Angle < 60 Degrees	-0.5
RT from Minor Street: Acceleration Lane Provided	-1.0
All Movements: Population ≥ 250,000	-0.5
Restricted Sight Distance	up to + 1.0

Note: Source: HCM (1985)

The most comprehensive set of field data that includes measures of delay at two-way stop control intersections was collected by Henry and Calhoun, reported in NCHRP Report 249 (TRB 1982). More than 136 15-minute observations were made at intersections with 4 approaches and 2-way stop signs. Linear regression analysis was conducted for the effect of right turns from the minor street. However, due to the limited scope of the analysis, no conclusive significance of the right turn factor could be claimed. For this reason, the data from the NCHRP report were included as part of this study. A more detailed description of the data, along with an in-depth statistical analysis is included in Chapter 5 of this thesis.

The total range of intersection types, geometric factors, and traffic characteristics and volumes is all but impossible to sample with field data alone. Computer simulation offers researchers a tool that can provide comparative data to a degree that would be practically impossible to achieve through the use of field data alone (Box and Alroth 1967).

Traffic simulation models are computer programs that are designed to represent realistically the physical system, either on a microscopic or macroscopic level (Gartner 1981). Microscopic models describe the detailed, time-varying trajectories of individual vehicles in the traffic stream. Macroscopic models, on the other hand, represent the traffic stream in some aggregate form (e.g., employing a fluid flow analogy or a statistical representation). As a result of the need to separate vehicles by type of movement to measure the right-turn factor, a microscopic approach is appropriate for this study.

Among the first applications of microscopic simulation models to measure delay at intersections were studies by Bleyl (1963), Kell (1963), and Lewis and Michael (1963). These studies involved developing delay curves for a range of major and minor street volumes for the purpose of comparing different types of traffic control devices (two-way stop versus signal). The effect of variation in turning movements was not considered in any of these studies. However, Kell recognized the influence of this factor and recommended further analysis.

Thomasson and Wright (1967) were the first to actually include turning movements as a variable in a simulation model study. They developed a simulation model based on mathematical relationships and probability distributions derived from 12 hours of field data collected at three stop-sign controlled intersections.

The model used a Monte Carlo distribution to assign gap and lag acceptance times. Left and through movements were assumed to have similar characteristics in terms of processing time, thus they were handled identically. Right turns from the minor street were assigned shorter gap acceptance times and processed by a different sub-routine within the model than the other two movements. The effect of right turns was studied by setting this factor equal to 10, 20, and 30 percent of the side street volume over a range of total approach volumes.

Thomasson and Wright completed 90 simulation runs of 1 hour each as part of their study. Their findings showed an increase in the average delay per minor street vehicle with increases in the percentages of the more complex left and straight movements. The results showing the 30 second delay lines for various combinations of major and minor street volumes at the three levels of right turn percentage are included in Fig. 2-1.

While the Thomasson and Wright study establishes a precedent for using traffic simulation to measure the effect of right turns, it is not conclusive. Their model was very simplistic from the standpoint of its capability to replicate the complex dynamics of an intersection. Additionally, they did not establish the statistical significance of the right turn factor within the framework of their simulation results.

2.3 Relationship Between Gap Acceptance and Delay

Three studies have established a theoretical relationship between gap acceptance and vehicle delay. The first, conducted by Raff (1950) was for uncontrolled intersections. However, under conditions where the driver on the minor street is forced to stop in order to give vehicles on the major street the right-of-way, the intersection functions as if under stop sign control. For this

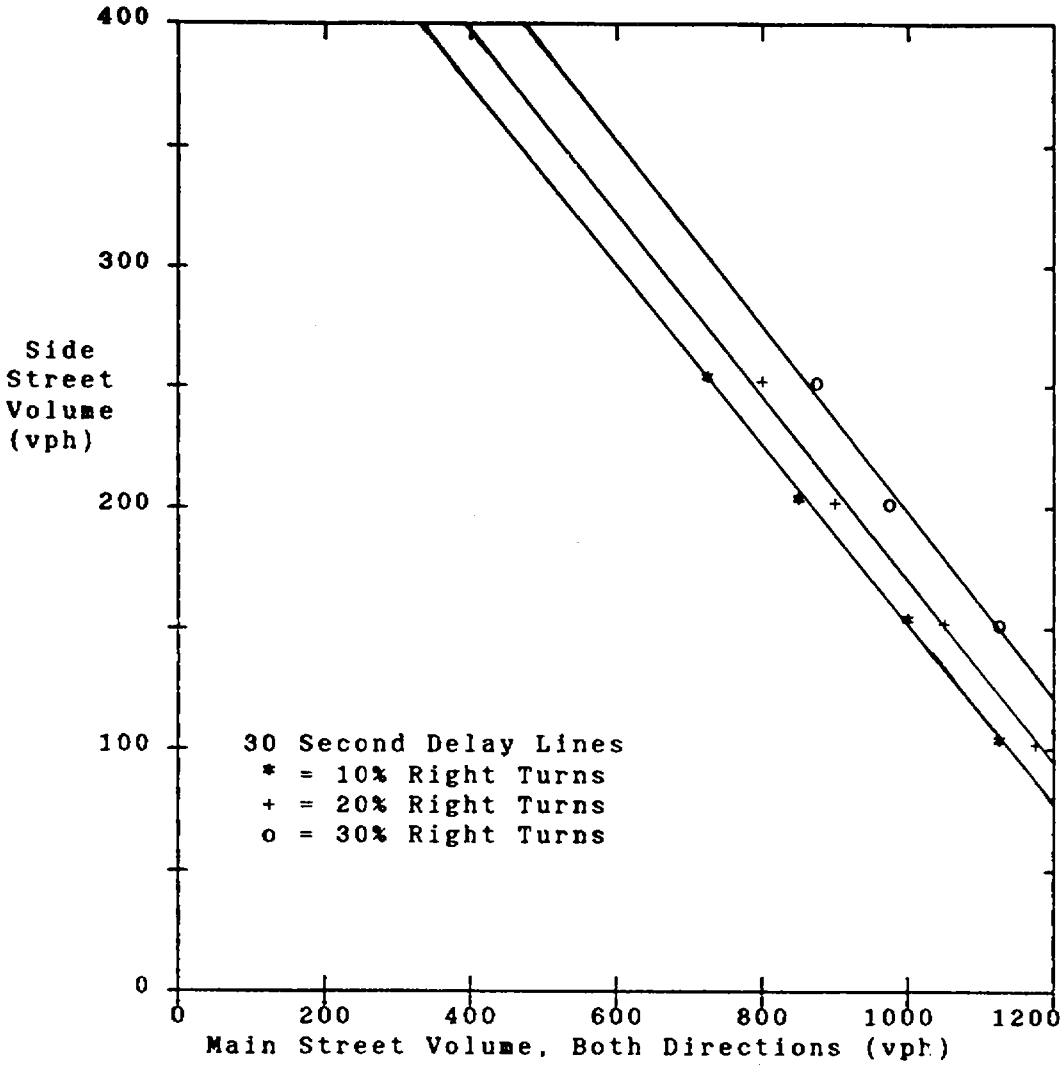


Fig. 2-1. Results from Thomasson and Wright Simulation Study--Relationship of Main and Side Street Volumes for Average Delay of 30 Seconds at 3 Right Turn Percentages.

Source: Thomasson and Wright (1967)

case, Raff developed an equation for mean delay for minor street vehicles. The relationship is expressed as:

$$d = q^{-1} (e^{qt} - qt - 1)$$

where: d = mean stopped delay (seconds)

q = major street flow rate (vehicles / second)

e = Napierian base

t = critical gap (seconds).

Ashworth (1969) developed a similar relationship based on a study done in Australia in 1962. Like Raff, Ashworth expressed delay as a logarithmic function of the critical gap. However, he assumed a distribution of major street headways in place of a constant flow rate.

Surti (1970) employed queuing theory to develop equations for average delay for side street vehicles at stop sign controlled intersections based on critical gaps. The expression for the average stopped delay for a single vehicle (not a vehicle in a queue) on the stop approach is:

$$\mathbf{d} = \frac{\int_0^T \mathbf{t \times f(t)\ dt}}{\int_T^\infty \mathbf{f(t)\ dt}}$$

where d = Average stopped delay (seconds)

t = Time gap on the major street (seconds)

T = Critical Gap (seconds).

Assuming an exponential distribution of traffic on the major street,

$$f(t) = qe^{-qt}$$

the mean delay for a given value of critical gap and major street flow rate becomes:

$$d = \frac{\int_0^T t x e^{-qt} dt}{\int_T^{\infty} e^{-qt} dt}$$

which reduces to:

where q is the flow rate on the major street (vehicles/second). This last equation was used to develop the delay curves shown in Fig. 2-2.

Surti found a high degree of correlation between theoretical predictions and results obtained from field observations at three urban intersections. He included separate critical gap values for right turn movements versus left turn and through movements in his field measurements.

While gap acceptance times have been shown to influence delay, several studies have found that delay can influence gap acceptance. Harders (1976) found that the value of critical gap can change while a driver is scanning for a sufficient gap in the first position of a queue. Often, drivers will finally accept shorter gaps than previously rejected after incurring a certain amount of delay. Retzko (1961), Findeisen (1971) and Tonke (1974) also reported this tendency of lower critical gap acceptance times with increased waiting time (delay). None of these studies, however, separated turning movements during the analysis.

2.4 Review of Current Practice

A review of current practices with respect to the treatment of right turns in traffic signal warrant applications by jurisdictions throughout the country reflects a lack of understanding of the effect of this factor. The results of a survey on the topic conducted by the National Committee on Uniform Traffic Control Devices are reported by Radwan and Upchurch (1987).

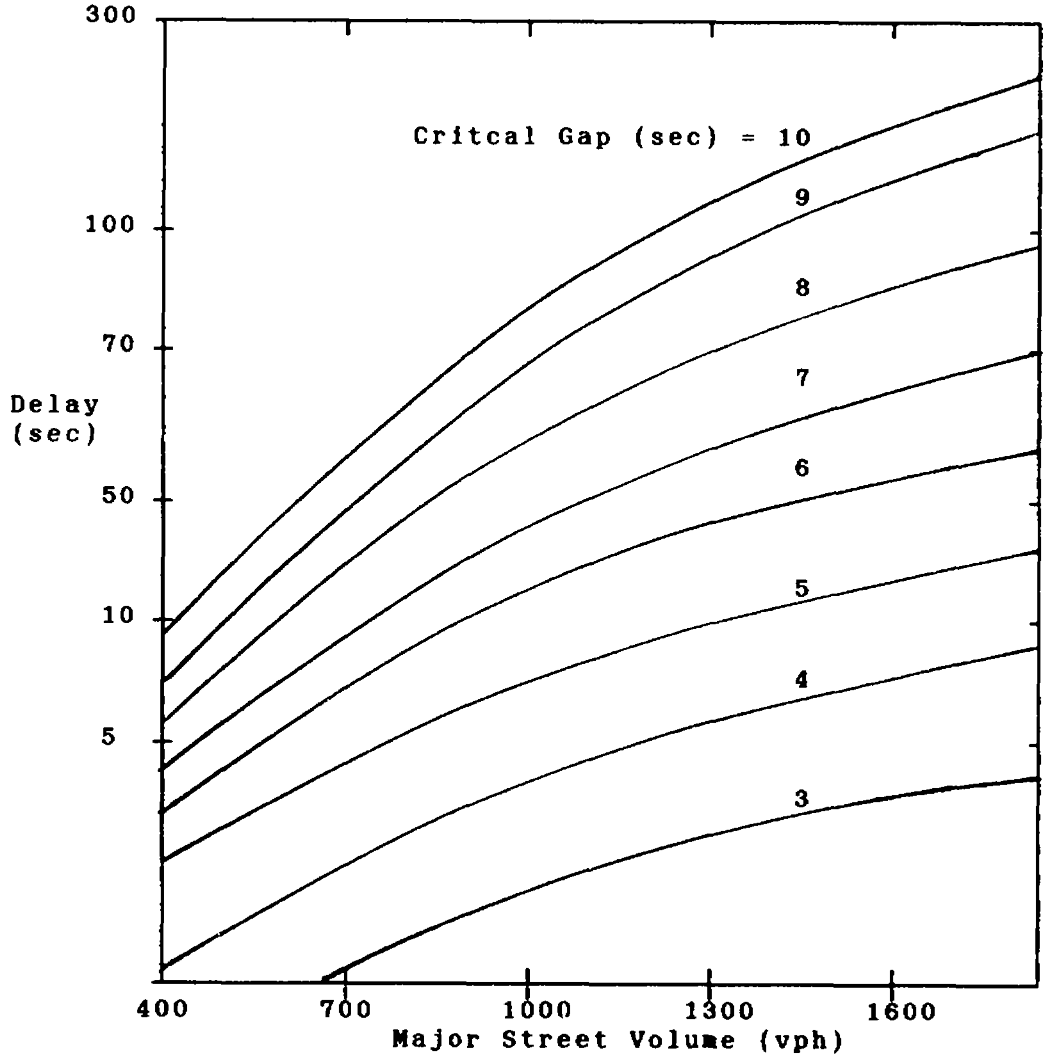


Fig. 2-2. Results from Surti Study--Delay as a Function of Major Street Volume and Critical Gaps

Source: Surti (1970)

The survey results indicate that the responding agencies consider up to 17 different factors when determining how much of the right turn volume (if any) to include in the minor street approach volume. Also, while 85 percent of the respondents indicated that they consider the effects of right turning vehicles, only 6 percent (4 jurisdictions) have written guidelines. This suggests that the majority of the jurisdictions rely on undocumented methodologies based on "engineering judgment."

Although the four written guidelines reported by Radwan and Upchurch employ a variety of methodologies, they all are fundamentally based on the parameter of side street vehicular delay. In essence, if it is determined that right-turning vehicles can enter the intersection without incurring considerable delay, some portion of the right turn volume can be deducted from the total side-street volume. The conditions that govern where this criteria can be applied vary according to jurisdiction. Each of the guidelines also rely on a certain amount of subjectivity or engineering judgment in their application.

The Arizona Department of Transportation (ADOT) developed a written guideline for the treatment of right turns in 1989. The criteria apply only to locations where a separate right turn lane exists and are intended to take into account the effect of right turn on red movements. A summation of the procedure states: The adjusted right turn volume equals the average peak hour right turn vehicle delay divided by the average peak hour total approach vehicle delay times the right turn volume.

ADOT's rationale for this procedure is that the need for a traffic signal is primarily related to vehicle delay. Because vehicles making a right turn from an exclusive turn lane do not contribute to the delay of through and left turning vehicles, a certain portion of this volume should be deducted.

CHAPTER 3

THE TEXAS SIMULATION MODEL

The Traffic EXperimental and Analytical Simulation (TEXAS) model is a microscopic computer simulator of traffic flow for isolated intersections. The purpose of the model is to provide a practical tool for transportation professionals to evaluate existing or proposed intersection designs and for assessing the effects of changes in roadway geometry, driver and vehicle characteristics, flow conditions, intersection control, and signal timing schemes upon traffic operations (Lee, Rioux, and Copeland 1977).

3.1 Development

The TEXAS model was developed at the Center for Transportation Research, The University of Texas at Austin, as part of the Cooperative Highway Research Program sponsored by the Texas State Department of Highways and Public Transportation and the Federal Highway Administration. Development of the model began in 1971.

Contrary to other simulation models of the time, which were configured primarily to handle multi-intersection, signalized networks, the TEXAS model was designed to evaluate isolated intersections operating under various types of control. Each driver-vehicle unit is treated separately throughout the simulation period. This microscopic approach allows a more detailed set of input factors and output parameters.

The first version of the TEXAS model was released in the late 1970's. It required a mainframe computer environment with data input and output on punch cards and magnetic tape. Extensive data input, including the coordinates of all lines and arcs, had to be calculated and coded individually (Lee, Machemehl, Inman, Copeland, and Sanders 1985). While this procedure was consistent with other models of the time, it was time consuming and impractical.

In 1985, Version 2.0 of the TEXAS model was released. This version offered the flexibility of running the model on either a mainframe computer or an IBM, or IBM compatible, microcomputer.

This "User-Friendly" version featured simplified data input through an interactive format. Also, it allowed a pre-defined intersection configuration to be chosen from a user-library and modified for specific geometric configurations. The need for inputting geometric coordinates was eliminated. Instead, all geometric features could be specified in terms of lengths and angles.

In addition, Version 2.0 of TEXAS incorporated an animated graphics screen display for viewing the simulation output. The animation operated on the microcomputer in either a real-time or stop-action mode, showing individual driver-vehicle units as they traveled through the intersection. Version 3.0 of the TEXAS model is in the testing stages as of this writing. The latest release features the ability to simulate diamond intersections. It also has an improved output display and enhanced animation graphics.

3.2 Structure

The microscopic nature of the TEXAS model requires that each driver-vehicle unit generated by the program be treated separately throughout the simulation period. At selected time intervals, the program provides the driver of each simulated vehicle information such as desired speed, actual speed, rate of acceleration or deceleration, destination, current position, relative position and velocity of adjacent vehicles, critical distances which must be maintained, sight distance, and the location and status of traffic control devices (Lee, et al. 1977).

The simulated driver is capable of processing this information and react by either accelerating, decelerating, maintaining current speed, or changing lanes. Drivers make decisions based on a priority logic under the premise that they want to maintain a desired speed, but will obey traffic laws and will maintain safety and comfort (Lee, et al. 1977).

To simulate this complicated scenario, the model is comprised of three primary processors: two pre-simulation processors and a simulation processor. The first processor, called GEOPRO, establishes the simulation geometry. The second processor, called DVPRO, creates the driver-vehicle pairs to be simulated. For simplicity, data input for these two pre-simulation processors are combined into one program, called GDVDATA. The third processor, SIMPRO, is for the actual traffic simulation. Fig. 3-1 shows the relationship between the various components of the TEXAS model in the form of a flow diagram.

3.2.1 Geometry Processor

The geometry processor, GEOPRO, defines the geometry of the intersection to be simulated. It calculates and stores all of the geometric details that are held constant throughout each simulation run. This processor determines the vehicle paths, both on approaches and within the intersection, as well as identifies the points of conflict between intersection paths and the minimum available sight distance between inbound vehicles (Lee, et al. 1977).

The geometry processor requires two types of input data: approach information and lane information. Included in the approach information requirements are values for the number of lanes for each approach, speed limit, and the maximum angular deviation of through and U-turn movements.

The lane information requirements include values for lane width, beginning and ending points of each lane, and turning movements allowed (if an inbound lane) and accepted (if an outbound lane). Thus, through values entered by the user, such conditions as turning bays and channelized lanes can be simulated.

Sight distance restrictions can be included in the simulation by means of user provided coordinates. The X and Y values of critical points along each obstruction must be entered in the geometry processor. For each 25 foot increment of the approach, a line is calculated from the center of the section to the coordinates of the obstruction. This line is then checked against a series of lines between perpendicular approaches. If any of the lines intersect, then a sight distance restriction has been established (Lee, et al. 1977).

Only horizontal alignment sight obstructions are considered by the model. Vertical alignment obstructions are not accommodated. Thus, when a sight distance restriction is established, the model treats it as if a vertical wall extends from the beginning of the other approach to the specified coordinates (Lee, et al. 1977).

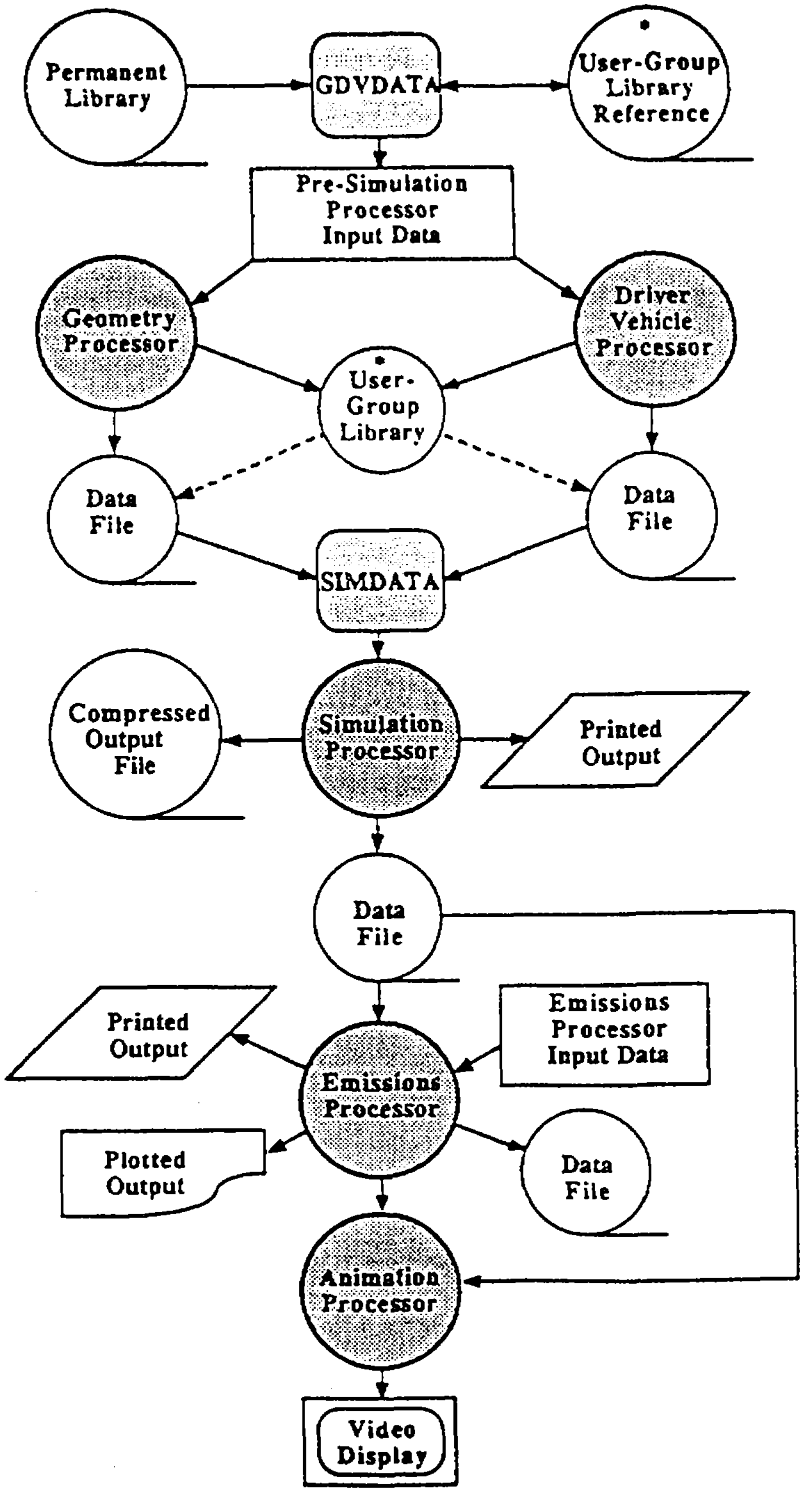


Fig. 3-1. Flow Diagram of TEXAS Model Components.

Source: Lee, Machemehl, and Sanders (1989)

The geometry processor output includes information about the approach azimuth, the X and Y coordinates of the curb arcs, and the listing of intersection paths. An example of the output listing generated by the geometry processor is included in Appendix A.

3.2.2 Driver-Vehicle Processor

The driver-vehicle processor, DVPRO, is the second pre-simulation processor. Its purpose is to generate driver-vehicle units for use by the simulation processor. Each of these units is randomly assigned a driver class, vehicle class, and the sequential order in which it will enter the simulated intersection. Thus, it establishes the simulation traffic stream and assigns attributes to the individual driver-vehicle units.

The input requirements for this program include the values for vehicular volume (expressed in hourly rates), the number of driver and vehicle classes, the mean and 85th percentile speed, turning percentages, the minimum headway, and the headway distribution. These parameters must be specified for each leg of the intersection.

The driver-vehicle processor generates the simulation traffic stream by means of random variates defined by three probability functions. The first, the empirical discrete distribution, defines the driver and vehicle class, the desired outbound approach, and the inbound lane number (Lee, et al. 1977). A normal probability distribution is then used to define the desired speed of each of these vehicles.

The third probability function is the user-defined headway distribution. One of seven types must be specified in the driver-vehicle processor program. They are: the constant, Erlang, gamma, lognormal, negative exponential, shifted negative exponetial, and uniform. Five of these distributions also require an input parameter, such as the standard deviation. This distribution assigns the queue-in time for each driver-vehicle unit.

The particular value or attribute assigned by each probability function is determined by a seed number. Each approach must have an associated seed for random numbers. The user can either supply this seed or choose automatic selection. However, for repeated runs using the same random variates, the model will retain the same seed number combination. Thus, in order to avoid duplication for replication runs, the random seeds must be input by the user.

Another parameter required by the driver-vehicle processor is the run time for the model. The run time is comprised of two parts: start-up time and simulation time. During the designated start-up time output statistics are not taken from the model. The purpose is to allow the simulated traffic to reach steady-state conditions. The run time is used by DVPRO to determine the number of driver-vehicle units needed for each approach during the simulation.

The driver-vehicle processor output includes information on the entered volumes, vehicle classes, and turning percentages. It also lists the corresponding values of these parameters generated by the model and the percent difference between the the two. An example of the output listing for the driver-vehicle processor is included in Appendix A.

3.2.3 Simulation Processor

SIMPRO simulates the the traffic behavior of each driver-vehicle unit according to the momentary surrounding conditions including any traffic control device indications which might be applicable (Lee, et al. 1985). The driver-vehicle unit is monitored moment by moment from the time it enters the inbound approach until the time it exits the system on an outbound approach. The simulation processor adjusts the forward and lateral movement, as well as the speed of the vehicle in order to respond to traffic control devices, access the desired lane, and to compensate for other vehicles in the system.

The data input requirements for the simulation processor include the start-up and simulation times, the time increment for simulation, parameters for the car following equation, and the type of intersection control. If a traffic signal is designated as the type of control, additional information is required related to phasing and timing.

While each driver-vehicle unit travels through the system, it gathers performance statistics about the traffic simulation. Once the unit exits the system, these statistics are reported within the simulation processor. The processor then sums and analyzes the relevant statistics and reports them in an output report.

The simulation processor output report lists a large number of performance statistics. Some of the more important statistics related to this study include:

- Average queue delay per vehicle
- Percent of vehicles incurring queue delay
- Average stopped delay per vehicle
- Percent of vehicles incurring stopped delay
- Average queue length
- Maximum queue length
- Number of vehicles processed
- Volume processed
- Percent of vehicles making a left turn
- Percent of vehicles going straight
- Percent of vehicles making a right turn

Each of these statistics is listed for individual turning movements, as well as for the summary of each approach and the summary of all approaches. As an example, for a four-leg intersection with no turn restrictions, the average queue delay will be listed for left turns, right turns, through movements, and all movements combined for each of the four approaches. Also, the average queue delay for all four approaches is listed at the end of the report.

Another important statistic listed for each approach and in the intersection summary is the number of vehicles eliminated from the system due to the approach lane being full. This parameter indicates when the number of vehicles being added to the system exceeds the processing rate on a given approach, resulting in a queue length longer than the approach length. The maximum approach length that the model allows is 800 feet.

Listed at the end of the summary report are several more important simulation statistics. They are: the number of vehicles processed during the start-up time, the number of vehicles processed during the simulation time, and the number of vehicles in the system at the end of the simulation period. These numbers help the user to identify when the model has reached equilibrium, hence when a sufficient start-up time has been chosen.

An example of a TEXAS model simulation processor output report is included in Appendix A. In order to avoid repetition, certain portions of the report have not been included. The listing of statistics by turning movement uses the same format for each approach. Thus, the output statistics for each turning movement are shown for only one approach.

3.3 Stop Sign Control

Two-way stop sign control is one of five types of intersection traffic control TEXAS can simulate. Several subprograms within the simulation processor contain the logic that processes a driver-vehicle unit through the stop sign controlled approach of an intersection.

The first step in the process involves logging a vehicle into the system as it reaches the stop line. This procedure maintains a list of driver-vehicle units which are arranged according to time of arrival, are stopped at the stop line, and are ready to enter the intersection (Lee, et al. 1977). Once a vehicle enters the intersection, it is removed from the list.

A driver-vehicle unit is allowed to enter the intersection when there are no other vehicles on the "waiting" list with precedence and there are no geometric conflicts along the desired travel path with the path of any other vehicles that have the right-of-way. Conflict paths are designated by the geometry processor based on intersection geometrics and allowed turning movements.

When a conflict is identified, the TEXAS model builds a safety zone around the predicted arrival time of the vehicle with the right-of-way. This safety zone consists of a time envelope in front of the vehicle as it approaches the intersection and a time envelope behind the vehicle as it departs. The driver-vehicle unit waiting at the stop bar is allowed to enter or cross the intersection when a sufficient amount of time is found between these safety zones.

The safety zone, or time envelope, in front of an approaching vehicle has two components. The first, which is set by the user, is called the time for lead zone (TLEAD). The range of possible values for this parameter is 0.50 to 3.00 seconds, with the default value being 1.30 seconds. The second component is the average perception-reaction (APIJR) time for all drivers. This parameter is subtracted from TLEAD to define the front safety zone.

The safety zone that trails a vehicle as it crosses the intended path of the driver-vehicle unit waiting at the stop bar is similarly comprised of two parameters. The first is the time for lag zone (TLAG). It is also user defined from a range of 0.50 to 3.00 seconds. The simulation model's default value for TLAG is 0.50 seconds. The second parameter for the rear safety zone is the same average perception-reaction time (APIJR) as above. Like the front safety zone, the rear zone is defined as TLAG minus APIJR.

Another variable that the TEXAS model uses in conflict checking is called the judgment error time (ERRJUD). It serves as an adjustment factor to more accurately predict a safe passage through the intersection. The concept behind ERRJUD is when a vehicle is more than 5.0 seconds from the intersection (either approaching or departing), the ability of a driver waiting at the stop line to predict this time decreases (Lee, et al. 1977).

The judgment error time is defined as:

$$ERRJUD = PIJR * (TCH - 5.0) / 7.0$$

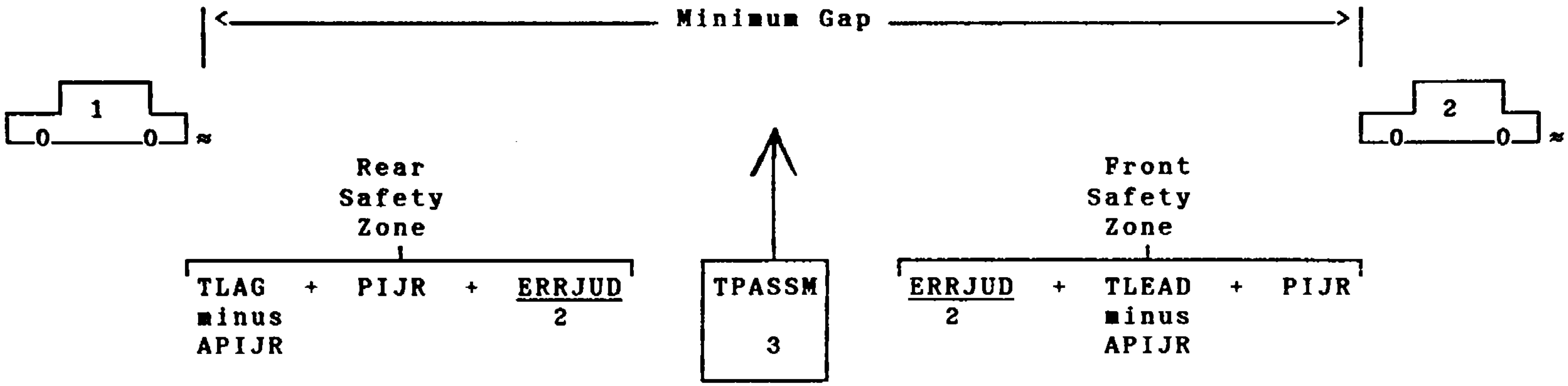
Where PIJR = the perception-reaction time of the driver waiting at the stop line

TCH = the travel time from the intersection of the vehicle with the right-of-way.

The conflict checking procedure used by the TEXAS model is comparable to the gap acceptance approach discussed in Chapter 1 of this thesis. Fig. 3-2 shows the relationship between the various components of the TEXAS conflict checking procedure.

As described in Section 3.2.1, the TEXAS model defines all the paths along the intersection approaches where a sight distance restriction exists. However, for stop controlled intersections, the simulation processor does not check for these restrictions. As stated in The Texas Model for Intersection Traffic-Development (Lee, et al. 1977):

If the inbound lane is stop sign controlled or signal controlled, the assumption is made that sight distance restrictions are not critical and, therefore, do not need to be checked. If adequate sight distance is not available to a unit stopped at the stop line, this will not be detected in SIMPRO.



Where: TPASSM = time for Vehicle 3 to pass through the point of intersection conflict

TLAG = time for lag safety zone behind Vehicle 1

TLEAD = time for lead safety zone in front of Vehicle 2

PIJR = average perception-reaction time for driver 3

APIJR = average PIJR for all drivers

ERRJUD = judgement error

Fig. 3-2. Intersection Conflict Checking Components in the TEXAS Model
Source: Lee, Rioux, and Copeland (1977)

CHAPTER 4

TEXAS MODEL ANALYSES

Traffic flow through an intersection is a dynamic and complex system involving many interrelated factors. The percent of right turns from a stop sign controlled approach is but one of these factors. Therefore, the evaluation of the effect of right turns on vehicular delay at two-way stop control intersections requires a very detailed level of analysis.

In principle, microscopic computer simulation is an ideal tool for this task. It offers the advantage of a "controlled" testing environment. Parameters can optionally be held constant or varied incrementally, in any number of combinations. The result is comparative data to a degree that would be practically impossible to achieve through field data measurements (Box and Alroth 1967).

The TEXAS model is perhaps the most microscopic simulation model for isolated intersections. In addition to the required user input values, the model has many parameters that can optionally be changed from the developer-supplied default values. The purpose is to allow fine-tuning, or calibration, of the model so it can replicate real world conditions as accurately as possible. For this reason, the TEXAS model was chosen as the primary "tool" for this portion of the study.

Rather than simulating specific intersections, the intent of this study was to evaluate the right turn factor for a variety of typical intersection configurations under common traffic conditions. The vehicular volumes of interest were those combinations of major and minor street flow at the threshold of warranting a traffic signal. Empirical data was not used as input for the model. Consequently, calibration of the TEXAS model required a self-optimization approach.

The study was organized into three sequential tasks. The first task involved establishing the universal, or overall, model testing environment. The two parameters that determine this environment are: the simulation time and the number of replications for each set of model runs.

The second task involved determining the significance of various secondary traffic and geometric factors with respect to the selected measure of effectiveness (average vehicle delay). The factors chosen for testing were those believed to most likely interact with the right turn factor within the framework of the TEXAS model.

Once the testing environment and significant secondary factors were established, the principle task of measuring the effect of right turns from the minor street on vehicular delay, with respect to the primary factors, could be performed. The vehicular approach volumes and the number of lanes on the major and minor streets were considered to be the primary factors.

4.1 Optimum Testing Conditions

The two parameters that have the greatest effect on the overall testing environment are the simulation time and the number of replications for each set of analyses using the TEXAS model. The simulation time is the duration, specified in minutes, that a single set of traffic and geometric parameters are modeled. The total simulation time is comprised of a start-up period and simulation period. Performance statistics are collected only during the actual simulation period and reported at the end of each run.

Replications are repeated runs of the same set of traffic and geometric parameters with the random seed number for each approach being the only variable. Each replication run will theoretically have a different set of results, as each set of random seeds will generate a unique set of driver-vehicle units entering the simulated intersection in different orders with different headways. However, the difference in the results between replications should be small, as the traffic and geometric parameters are held constant for each run.

The goal was to run the model long enough to produce a consistent set of results for the specified conditions. That is, the difference in the values of the output statistics between any two replications should be as small as possible. The purpose of using replications was to offset any differences between runs that existed by taking the average, or mean, of each set of runs. Ideally, the combination of simulation time and number of replications should produce a set results that are completely repeatable with any other set of random seed numbers.

4.1.1 Method

The method used to determine the number of replications for this study involved a series of trial simulation runs. A standard 4x2 intersection with stop sign control on the single lane (minor) approaches was used as the test case. The set of traffic and geometric parameters were held constant for all model runs. The TEXAS model's default values for start-up and simulation time were used.

Four replication levels were tested, ranging from 5 to 20, in intervals of 5. Within each level, 60 runs (observations) were conducted and the results grouped into 3 to 6 populations, depending on the replication level. The exception was the lowest replication level, where only 30 runs were made. Thus, for the lowest replication level (5), the 30 runs were grouped into 6 populations of 5 replications.

The mean and standard deviation of the average queue delay for each population were calculated and an analysis of variance (ANOVA) conducted to determine the significance of the variability between each population compared to the variability within the population for each of the 4 replication levels. The optimum replication level was chosen on the combined basis of the ANOVA results and simulation model user judgment.

A similar method was employed to determine the optimum simulation time to use for the study. Theoretically, the longer the simulation time the more precise the results. The TEXAS model allows a total simulation time range of 12 to 70 minutes. Due to the large number of runs required for this study, however, a shorter simulation time was desired for reasons of practicality. Thus, the optimum simulation time was defined as the minimum length of a run that will provide a satisfactory level of precision with respect to the results.

Nine different simulation times were tested, ranging from 15 to 55 minutes, in 5 minute increments. For each time interval, a high and a low volume level were run. The means and standard deviations for queue delay was calculated and an analysis of variance conducted.

The values of the mean were also plotted for each volume level over the range of simulation times. The intent was to locate the point on the plot where the mean delay began to stabilize, or level out. The simulation time corresponding to this point would be taken as the optimum value.

4.1.2 Results

The traffic volumes for the replication analysis were specified as 500 vehicles per hour (vph) on the major street and 150 vph on minor street. Turning percentages were held constant at 15% left and right on the major streets and 33% left and right on the minor approaches. The default values of 5 minutes and 15 minutes were used for the start-up and simulation time, respectively.

The mean and standard deviation of each population for the four replication levels are listed in Table 4-1. The F-statistic along with the critical value of F for a confidence level of 95% (alpha = .050) are also included. No significant differences between the population means were found for any of the four replication levels. Therefore, from a statistical standpoint, 5 replications are as acceptable as 20 replications for these particular test conditions.

Because a variety of test conditions were included in the study, a value greater than the minimum acceptable number of replications was desired. The rational being that additional replications would offset larger variations between individual run results that may occur with other combinations of input parameters (i.e., a safety factor). Thus, sets of 10 replications were chosen as the optimum number for the TEXAS model analyses.

A list of random seed number combinations for 10 replications was produced to use as a standard for the TEXAS model runs. The purpose was to eliminate potential variability within replication groups--as certain random seed combinations generate consistently higher or lower

TABLE 4-1. STATISTICAL RESULTS OF TEXAS MODEL REPLICATION NUMBER DETERMINATION.

No. of Replications	Group Number	Mean Delay (sec.)	Std. Dev. (sec.)	ANOVA	
				F	$F_{.95}$
5	1	6.54	1.05	0.27	2.01
	2	6.82	0.84		
	3	7.60	1.10		
	4	6.90	3.29		
	5	6.48	1.00		
	6	6.49	0.69		
10	1	6.82	0.90	0.38	2.39
	2	6.71	0.93		
	3	6.74	2.27		
	4	7.05	1.26		
	5	6.69	0.99		
	6	6.25	1.15		
15	1	6.37	1.22	0.90	2.80
	2	7.11	1.77		
	3	6.58	1.21		
	4	6.85	0.90		
20	1	6.96	1.76	0.48	3.16
	2	6.60	1.01		
	3	6.62	1.06		

values of average delay relative to other combinations, independent of the other input parameters. The seeds, listed in Table 4-2, were selected from a random number chart.

The same intersection configuration and turning movement percentages as used in the replication tests were input into the runs for the determination of the optimum simulation time. The only model parameters that differed were approach volumes. The high volume level was input as 700 vph on the major street approaches and 150 vph on the minor street. The low volume level was specified as 300 vph on the major street and 75 vph on the minor. Ten replications were run for both volume levels for each of the nine simulation times tested.

An analysis of variance revealed no significant difference in mean delay at the 95% confidence level for either volume level, $\underline{F}(8,166) < 1$, N.S. (Not Significant). The ANOVA table B-1 is included in Appendix B.

The means of the average queue delay for each set of runs was plotted at both the low and high volume level to identify any trends in the relationship between delay and simulation time (Fig. 4-1). The plot revealed no distinguishable stabilization of delay over time for the high volume.

On the other hand, the mean delay remained relatively constant throughout the range of simulation times for the low volume. Since no statistical advantage was found for using a longer simulation time, the optimum simulation time for the remaining analyses was chosen as 15 minutes, with a 5 minute start-up time. These times correspond with the TEXAS model default values.

4.2 Sensitivity Analyses of Factors

Six secondary factors in the TEXAS model were identified as having a possible effect on minor street delay. These included four traffic parameters and two parameters related to intersection geometry. The traffic factors were: the parameter for the selected headway distribution, traffic composition, major street speed limit, and the percentage of vehicles turning left from the minor street. The two geometric factors were: the presence or absence of an

TABLE 4-2. RANDOM SEED NUMBERS SELECTED FOR TEXAS MODEL REPLICATION RUNS.

SEED NUMBER	APPROACH NUMBER			
COMBINATION	1	2	3	4
1	84921	51427	19620	12172
2	75132	89703	44319	73185
3	20986	97852	61477	51336
4	90735	12159	21463	22114
5	85170	31800	59362	38727
6	80485	18866	60292	92165
7	22889	8227	71623	48932
8	33484	71597	95536	82438
9	47642	69253	87895	66631
10	15307	43872	31569	77436

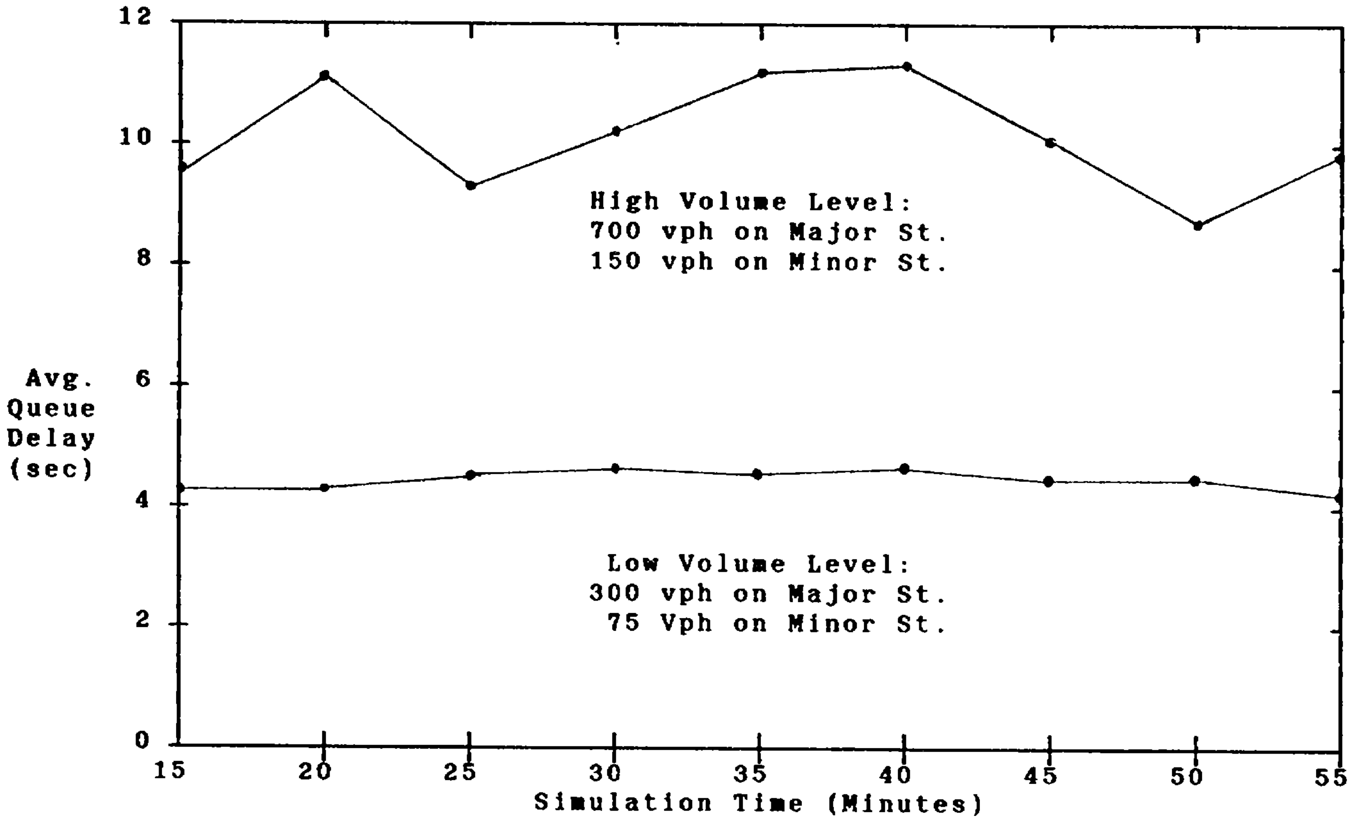


Fig. 4-1. Delay vs. Simulation Time at Two Volume Levels for a 4x2 Intersection with Stop Sign Control.

exclusive right turn lane on the minor street and the presence or absence of right turn channelization onto the major street.

Two additional factors that potentially effect right turn delay are restricted sight distance on the minor street and pedestrians crossing the major street. These parameters could not be tested, however, due to the limitations of the TEXAS model.

4.2.1 Method

The significance of the effect, if any, of the identified factors involved a series of sensitivity analyses. For each factor, groups of model runs were conducted over a specified range of values and compared to one another statistically. With the exception of the factor in question, all parameters were held constant. This allowed the inference that any changes in the measure of effectiveness were attributable to either the factor itself or to random variation resulting from the seed combinations. An analysis of variance was performed to identify the relationship between these two parameters.

The sensitivity analyses for two of the factors were performed at several volume levels. This allowed a better evaluation of the effects of these factors over the range of volumes of interest in this study.

4.2.2 Results

A shifted negative exponential headway frequency distribution was selected for use in this study. This distribution was chosen because of the seven types available in the TEXAS model, the shifted negative exponential best represents poisson (random) arrivals. This is the most common type of headway frequency distribution used to predict vehicular arrivals for isolated intersections at relatively low volumes (ITE 1982). It is also the default distribution for the TEXAS model.

The purpose of the analysis performed for the headway frequency distribution was to evaluate any effect that changing the corresponding parameter value may have on average delay. The particular parameter required for the shifted negative exponential distribution is the minimum allowable headway. The TEXAS model requires this headway value to be less than or equal to the mean headway.

While the model does not use the mean headway as an input parameter, the value of the minimum headway is user specified. The range of possible values for this parameter are 1.0 to 3.0 seconds. The default value of 1.0 minimum headway was chosen for this study, as it results in a less conservative simulated traffic flow.

The headway distribution parameter was tested at three volume levels on the major street (300, 500, and 700 vehicles per hour), which consisted of two lanes on each approach. The minor street (stop controlled) was specified as one lane per approach and the volume was held constant at 150 vehicles per hour.

Two sets of replication runs were conducted at each of the three major street volume levels; one set with a 1.0 second parameter and one set with a 2.0 second parameter value. The means and standard deviations of the average stopped delay were computed for each of the six replication sets and an analysis of variance performed.

No significant difference was found between the two headway values over the range of volumes, $F(1,54) < 1$, N.S.. A value of 1.0 second was selected as the headway distribution parameter for this study on the basis of the lower standard deviation in the average delay as compared to a 2.0 second value. The ANOVA table B-2 is included in Appendix B.

The second traffic factor tested was traffic composition. A 4x2 lane stop-controlled intersection was used to determine whether the traffic mix (i.e. percent trucks) has an impact on average stopped delay. Two levels of traffic composition (5% and 15% trucks) were tested at a high and a low volume level. These values mark the lower and upper range of trucks in the traffic mix typically found on urban streets (ITE 1982).

The low volume level was specified as 75 vph on the minor street and 150 vph per major street. The high volume level was set at 300 vph on the minor street and 600 vph on the major street.

The TEXAS model uses a default value of eight percent trucks. However, the model divides this percentage into two categories: single-unit trucks (5.6%) and tractor semi-trailer trucks (2.6%). Furthermore, within each of these categories, there are designations for gasoline or diesel powered trucks as well as distinctions between partially-loaded and fully loaded trucks.

Each of the eight possible truck classifications is distinguished by a different set of performance characteristics. The same relative proportion as used in the default case was specified for each of the eight classifications in the two levels tested. An analysis of variance was performed on the four sets of replications did not reveal a significant difference between the two levels of traffic composition at either volume level, $F(1,39) < 1$, N.S. The ANOVA table B-3 is included in Appendix B.

Based on the results of this analysis, the TEXAS default value of eight percent trucks was selected as the single value to use for the remainder of the study. Because variation in the traffic mix (over a small range) does not effect average delay in the TEXAS model, it is not necessary to consider more than one value.

The third traffic factor tested was major street speed limit. A 4x2 lane intersection with stop control on the one-lane approaches was used in the test scenario. Volumes were specified as 700 vph on the major street and 150 vph on the minor street. Three speed limits were tested: 30 mph, 40 mph, and 50 mph. The corresponding mean and 85th percentile speeds for the simulated vehicles were adjusted to one mile per hour below the corresponding speed limit and one mile per hour above, respectively.

One set of replications per speed limit were run and an analysis of variance performed using average stopped delay on the stop control approaches as the measure of effectiveness. The statistical analysis revealed no significant difference between the three speed limits at the 95% confidence interval, $F(2,29) = 1.19$, N.S. The ANOVA table B3 is included in Appendix B.

On the basis of the analysis results, a single value for the major street speed limit was determined to be adequate for the remaining TEXAS model runs. The mid-range value of 40 mph was selected as this model parameter.

The fourth traffic factor tested was the minor street left turn percentage. The same test scenario as used for the speed limit test was used for this analysis. A one lane approach on the minor street was considered the critical case, as left, right, and through traffic movements have the greatest interaction when executed from a single queue.

Three left turn percentages were tested: 10%, 20%, and 30%. The minor street right turn percentage was held constant at 10%. The percentage of vehicles traveling straight (through movements) was adjusted to account for the variation in left turns. Thus, for the case of 10% left turns, the percent of through movements was specified as 80%.

An analysis of variance performed on the three sets of replication runs revealed no significant difference between the levels of left turn percentages for this test scenario. At the 95% confidence level, $F(2,29) < 1$, N.S. The ANOVA table B-5 is included in Appendix B. For the sake of uniformity, a value of 10% left turns was selected for most of the remaining model runs. The first of the two geometric factors tested was the presence or absence of an exclusive right-turn lane. Four cases were analyzed: a base case (no exclusive right turn lane), and three cases with an exclusive turn lane. The difference between the later three being the length of the lane (100 ft, 150 ft, and 200 ft). The length of the turn lane determines its capacity for storing queued vehicles.

A 4x2 lane intersection was used for the base case. The minor street had a one lane stop-controlled approach in both directions. Traffic movements were restricted to through (70%) and right turns (30%) from the minor approaches. Volumes were specified at 700 vph on the major street approaches and 150 vph on the minor.

The intersection used for the three exclusive right turn cases had a 4x3 lane configuration. The permitted traffic movements and their corresponding percentages, as well as the volumes, were set at the same values as the base case. A generalized representation of an intersection with an exclusive right turn lane is shown in Fig. 4-2.

One set of replication runs was performed for each case and an analysis of variance conducted using the results. No significant difference was found between any of the four cases, $F(3,39) < 1$, N.S. The ANOVA table B-6 is included in Appendix B.

On the basis of this analysis, it was determined that it was not necessary to consider the case of an exclusive right turn lane in the remaining model tests. Rather, the base case, where the lanes have shared turning movements, was chosen as the model parameter.

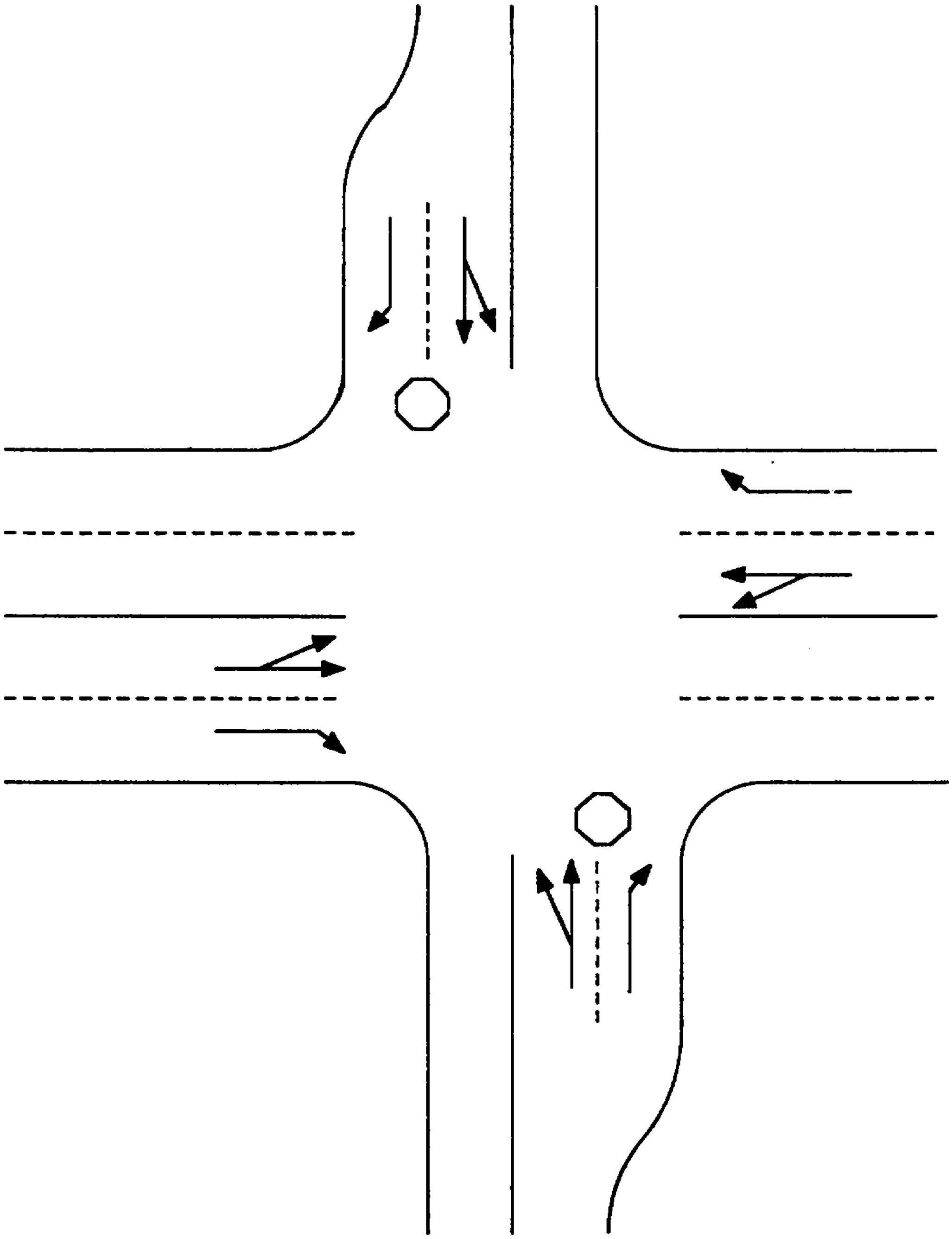


Fig. 4-2. 4x3 Intersection with Exclusive Right Turn Lane.

The other geometric factor tested was the presence or absence of intersection channelization for right turns from the minor street. Channelization is defined as the separation or regulation of conflicting traffic movements into definite paths of travel by means of traffic islands or pavement markings to facilitate the safe and orderly movements of both vehicles and pedestrians (AASHTO 1973).

The channelized case used a 4x4 lane intersection configuration similar to the one shown in Fig. 4-3. For this intersection, vehicles turning right from the two minor street approaches utilize a channelized lane with yield control. The other lane on the minor street approaches is restricted to through and left turn movements only, and is controlled by a stop sign. Though movements were specified for 70% of the approach volume (150 vph) and right turns were assigned the remaining 30%. The major street volume was set at 700 vph.

The base case used a standard 4x4 lane intersection with stop control for all of the minor street lanes. The volumes and turning percentages were set at the same levels as in the channelized case.

The initial statistical analysis performed on the results of the replication runs revealed a significant difference between the two cases, $F(1,19) = 40.67$, $p < .05$. However, upon inspection, the mean value for average queue delay was higher for the channelized case than for the base case (15.8 seconds vs. 7.5 seconds). Upon examination, this pattern was discovered to be a result of the method used in TEXAS to calculate queue delay.

A vehicle does not report queue delay statistics as it "travels" through the simulated intersection unless it slows to under a certain speed. The TEXAS default value for this parameter is 10 mph. This was the value used in the analysis. For the channelized intersection, the vehicles turning right entered the major street in a protected lane after yielding. Hence, few, if any of these vehicles slowed to below 10 mph.

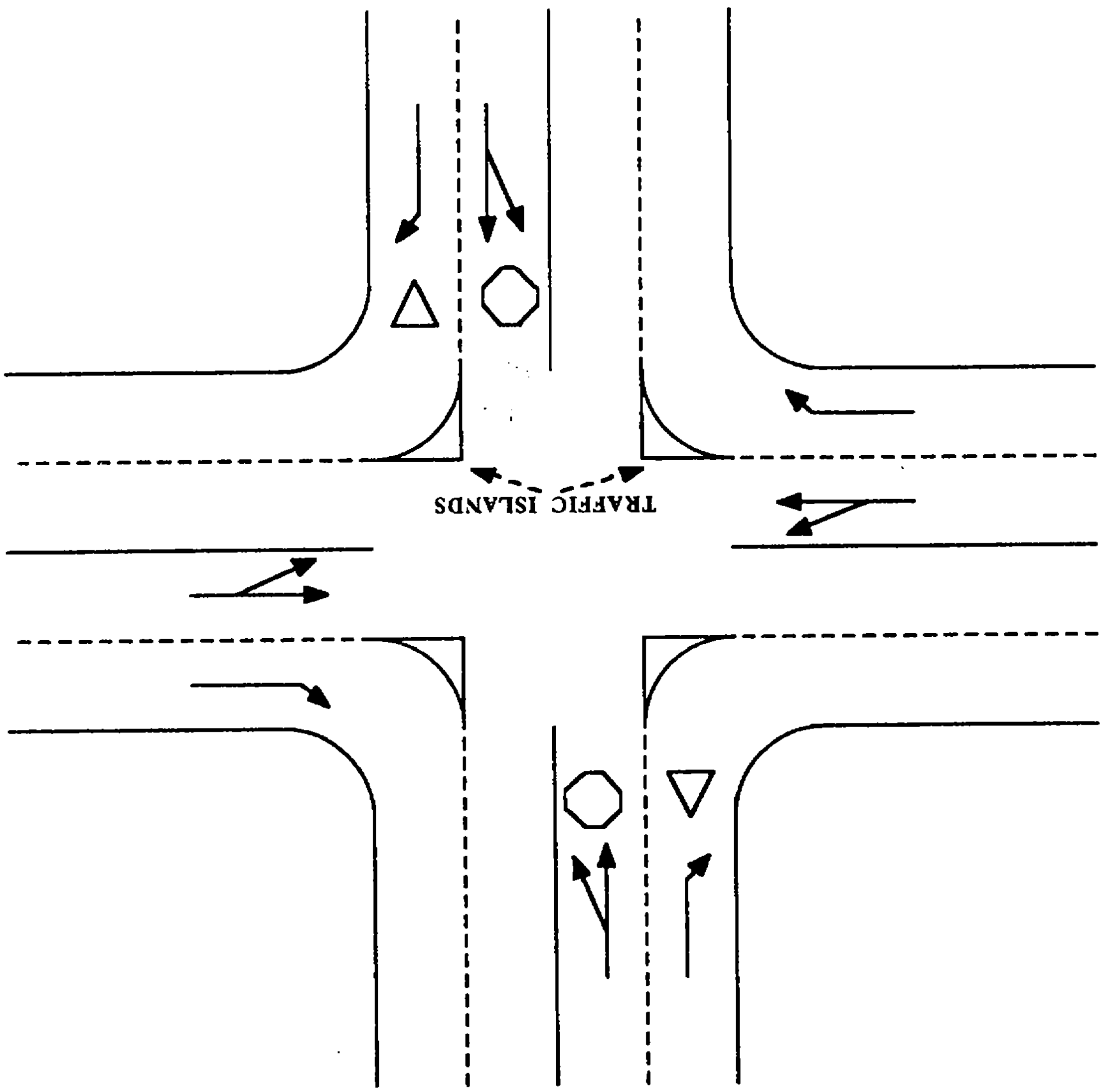


Fig. 4-3. 4x4 Intersection with Right Turn Channelization

The delay statistics, therefore, were reported only for the through vehicles. Since the delay is reported as an average for only those vehicles encountering queue delay, the value for average queue delay was not representative of the total approach volume.

This phenomenon was accounted for by recalculating the average queue delay for the two minor street approaches using a different formula. The total queue delay was divided by the sum of the through and right turn volume giving a weighted value for average queue delay per vehicle. The effect was a lower value for this parameter for a given run.

A second analysis of variance was performed using the recalculated values. As in the first analysis, the results showed a significant difference between the base case and the channelized case, $F(1, 19) = 10.90$, $p < .05$. Unlike the first analysis, however, the mean average queue delay was greater for the base case than for the channelized case (11.9 seconds vs. 9.6 seconds). The ANOVA table B-7 for this analysis is included in Appendix B.

The results of the second analysis indicate that the use of right turn channelization significantly reduced the average queue delay for vehicles on the minor street approaches for this particular set of conditions by separating right-turning vehicles from vehicles executing other movements. Additionally, the channelized lane virtually eliminated queue delay for right turning vehicles.

Because the goal of this study was to evaluate the effect of right turns on the total minor street traffic stream, it was of interest to consider the case where all the traffic movements interact. While the channelization analysis was important from the perspective of demonstrating the potential benefits of this geometric treatment, the base case (non-channelization) was selected as the intersection configuration for the remaining analyses.

Table 4-3 summarizes the findings from the analyses performed for all six secondary factors, plus the analyses to determine the values for the optimum number of replications and the simulation time. Also included are the values for these parameters chosen for use in most of the remaining TEXAS model runs conducted for this study. These values were used as model input unless specified otherwise.

4.3 Right Turn Factor Analyses

Following the establishment of the optimum simulation model testing environment and the determination of the role of secondary traffic and geometric factors, the primary goal of evaluating the effect of right turns from the minor street on average vehicular delay at two-way stop controlled intersections could be undertaken.

The primary factors considered in this analysis were the major street volume, the minor street volume, the number of lanes on the major street and minor street, and the percent of vehicles turning right from the minor street approaches.

4.3.1 Method

Prior to conducting a detailed analysis for the right-turn factor, a determination of the worst case geometric scenario (i.e., number of lanes on each approach) with respect to the right turn factor was necessary. Due to the extensive amount of time required to set-up and perform the number of simulation replications necessary for a statistically valid analysis, the focus of this study was limited to the geometric configuration considered most sensitive to the right-turn factor.

Following the determination of the appropriate lane combination, the task of measuring the effect of variation in right turns on average queue delay could be performed. The right turn factor evaluation consisted of two separate analyses.

The first analysis involved a series of TEXAS model runs performed over a range of minor street volumes and right turn percentages. The runs were repeated for two volume levels on the major street. An analysis of variance was conducted on the simulation results to measure what effect, if any, varying the percentage of right turns had on the measure of effectiveness. The

TABLE 4-3. SUMMARY OF TEXAS MODEL PARAMETER ANALYSES AND VALUES SELECTED FOR USE IN RIGHT TURN ANALYSES.

PARAMETER	RESULT OF ANALYSIS	VALUE SELECTED
Number of Replications	Not Significant	10 replications
Simulation Time	Not Significant	20 min. (w/ 5 min. start-up time)
Headway Distr. Parameter	Not Significant	1.0 second
Traffic Composition	Not Significant	8.0% Trucks
Major Street Speed Limit	Not Significant	40 mph
Minor Street Left Turns	Not Significant	10 %
Right Turn Lane	Not Significant	No exclusive right turn lane
Right Turn Channelization	Significant	No right turn channelization

average queue delay per vehicle on the minor street approaches was chosen as the measure of effectiveness.

The second analysis involved another series of TEXAS model runs over a wide range of values for seven traffic parameters. In addition to the primary factors of volume and minor street right turn percentage, minor street left turn percentage, major street turning percentages, and major street speed were included in the analysis.

A multivariate analysis of variance was performed on the simulation results. The purpose of this statistical test was to determine which parameters significantly accounted for the variance in minor street delay with the effects of the random seed variation removed. It also measured any interactions that occurred between the seven factors.

4.3.2 Results

Three sets of common intersection lane combinations were evaluated to determine the one most sensitive to the right turn factor. Configuration 1 consisted of a 2x2 lane combination (one lane on each approach). Configuration 2 was a 4x2 lane combination (two lanes on the major street approaches and one lane on the stop-sign controlled approaches). Configuration 3 utilized a 4x4 lane combination (two lanes on all approaches).

The volume level for this traffic simulation analysis was set at 700 vph on the major street approaches and 150 vph on the minor street approaches. One set of replications runs was conducted for each case with 10% of the minor street traffic volume assigned to turn right. A second set of TEXAS model runs was performed for each of the three cases with the minor street right turns specified at 30%.

The average queue delay per vehicle on the minor street approaches was used as the measure of effectiveness. The mean and standard deviation for the values of delay were calculated for each set of replications (Table 4-4). An analysis of variance did not reveal a significant difference between the two levels of right turn percentages for any of the three lane configurations, $F(1,59) = 1.56$, N.S. However, there was a significant effect for the type of lane configuration, $F(2,59) = 29.29$, $p < .001$. The ANOVA table B-8 is included in Appendix B.

On the basis of this analysis, none of the three lane configurations represents a worst case scenario for intersection traffic simulation in the TEXAS model. Therefore, a 4x2 lane arrangement (configuration 2) was selected as the geometric scenario for the remaining right turn analysis. This choice was made on the basis that configuration 2 (4x2) is the middle, or median, case.

The first of the two right turn factor analyses was initially conducted for a low traffic volume level. The major street volume was set at 200 vehicles per hour per approach. The minor street volume was varied over a range of 150 vph to 190 vph, in increments of 10 vph. Thus, there were five values of minor street volume.

Three values of minor street right turn percentage (10%, 20%, and 30%) were tested at each level of minor street volume. The left turn movements from the minor street was held constant at 10%. The percent of through movements was varied from 60% to 80% in order to account for 100% of the the minor street traffic stream. All other model parameters were held constant.

One set of replications (10 runs) was conducted for each of the 15 volume/percent-right-turn combinations. The two minor street approaches were treated as separate cases with respect to collecting the simulation output statistics. Thus, for each of the 10 simulation model replication runs performed for each volume/percent-right-turn combination there were 20 values of average queue delay collected.

TABLE 4-4. MEAN AND STANDARD DEVIATION OF DELAY FOR LANE CONFIGURATION ANALYSIS.

Lane Configu-ration(Case No.)	10% Right Turns		30% Right Turns	
	Mean (sec.)	Std. Dev. (sec.)	Mean (sec.)	Std. Dev. (sec.)
2x2 (Case 1)	25.75	10.49	22.09	9.56
4x2 (Case 2)	16.02	3.79	15.12	4.32
4x4 (Case 3)	9.47	1.28	7.97	1.35

The mean and standard deviation was calculated for each set of replications (Table 4-5). There is an observable trend for the mean value of average queue delay to decrease with increasing values of right turn percentages at the different volume levels. An analysis of variance, however, revealed that this trend is not significant, $F(2,296) = 1.13$, N.S. The ANOVA table B-9 for this analysis is included in Appendix B.

Another trend observable from the table is a tendency for the standard deviation of the delay values to increase with an increase in the volume level. No statistical analysis was performed to verify the significance of this pattern, however.

A trend that is not as readily apparent from observing Table 4-5, but revealed in the ANOVA, is the existence of a significant difference in the mean delay with respect to volume level. While the analysis of variance establishes this condition, $F(4,296) = 9.68$, $p < .001$, additional statistical analysis would be required to determine which volume levels varied significantly from each other. As understanding the effects of variation in the volume level is not the the focus of this study, no additional analysis was conducted for this set of TEXAS model runs.

Since the right turn factor was not significant at a low volume level, a higher volume level was tested. The same intersection configuration and model parameters were used with exception of the approach volumes. The traffic volume on the major street was set at 700 vph and the minor street approach volumes was specified over a range from 100 vph to 140 vph, in increments of 10 vph. Thus, the same number of cases were tested as for the low volume scenario.

The mean and standard deviation for each replication set is shown in Table 4-6. Once again, there is an observable trend for a decrease in the mean delay with an increase in the right turn percentage over the range of volumes. However, an analysis of variance revealed that the amount of change in the delay values is not significant relative to other factors that cause variation, $F(2,297) = 2.10$, N.S.

TABLE 4-5. MEAN AND STANDARD DEVIATION OF DELAY FOR RIGHT TURN FACTOR ANALYSIS WITH 200 VPH ON MAJOR STREET

SIDE STREET VOLUME (VPH)	PERCENT RIGHT TURNS (%)	MEAN (SEC.)	STD. DEV. (SEC.)
150	10	5.14	0.84
	20	5.17	1.14
	30	4.92	0.72
160	10	5.08	0.69
	20	4.82	0.62
	30	4.82	0.94
170	10	5.62	0.97
	20	5.36	0.94
	30	5.40	1.31
180	10	6.10	1.85
	20	6.33	1.68
	30	5.99	1.46
190	10	5.78	1.63
	20	5.60	1.17
	30	5.26	1.17

Table 4-6. Mean and Standard Deviation of Delay for Right Turn Factor Analysis with 700 VPH on Major Street.

Side Street Volume (VPH)	Percent Right Turns (%)	Mean (Sec.)	Std. Dev. (Sec.)
100	10	16.60	6.38
	20	15.96	6.40
	30	15.20	6.42
110	10	18.20	6.17
	20	17.31	6.09
	30	15.12	4.58
120	10	22.10	12.04
	20	19.97	11.44
	30	18.39	11.23
130	10	21.39	9.38
	20	18.70	6.69
	30	17.71	5.76
140	10	24.54	10.67
	20	27.04	18.68
	30	22.74	13.13

As in the low volume case, the standard deviation tends to increase with an increase in the minor street volume. Also, the mean delay tends to increase with an increase in the volume level. Like the low volume case, this trend is statistically significant, $\underline{F}(4,297) = 7.59$, $\underline{p} < .001$. Unlike, the former case, however, this trend is more obvious from examining the table for the higher volume levels. The ANOVA table B-10 is included in Appendix B.

The second analysis for the right turn factor was conducted over a wider range of parameter values (percent right turn and volume levels) than the first analysis. Also, four additional parameters that had been held constant in the first analysis were treated as variables in the second analysis. The purpose of adding more variables was to determine the level of contribution of the right turn factor relative to all the remaining factors (i.e., the other variables).

The four additional factors used in this analysis were: the major street speed limit, the minor street left turn percentage, the major street left turn percentage, and the major street right turn percentage. These factors were selected on the basis that they are traffic related factors that could potentially affect queue delay on the side street.

No geometric parameters were included as variables in this analysis, as it was desired to test the same intersection configuration as in the first analysis. Furthermore, the TEXAS model does not allow the number of lanes on an approach to be changed once a configuration is selected. Thus, an entirely new file must be established for each intersection configuration.

Three values were chosen for each of the seven traffic variables: a low value, a high value, and a mid-point value. The ranges for these values were selected on the basis of engineering judgment. That is, values thought to be the upper and lower practical limits that exist in the field under similar geometric and traffic control conditions. The parameters and the corresponding selected range of values are listed in Table 4-7.

TABLE 4-7. SELECTED RANGE OF VALUES FOR TRAFFIC PARAMETERS USED IN SECOND RIGHT TURN ANALYSIS.

PARAMETER	MINIMUM VALUE	MID-POINT VALUE	MAXIMUM VALUE
Major Street Volume (vph)	200	550	900
Minor Street Volume (vph)	75	237	400
Major Street Speed Limit (mph)	25	40	55
Minor Street Right Turns (%)	0	20	40
Minor Street Left Turns (%)	0	20	40
Major Street Right Turns (%)	0	20	40
Major Street Left Turns (%)	0	20	40

The experimental design selected for this analysis required 17 sets of replication runs. One of the 17 sets consisted of the mid-point values for all 7 parameters. The other 16 sets of runs used various combinations of the high and low values. This experimental approach reduced the statistical "noise" caused by the random seed numbers.

For each replication set three TEXAS model runs were conducted. The mean and standard deviation of the average queue delay, as well as the average stopped delay, recorded from both minor street approaches for the three runs was then calculated for each of the 17 replication sets. A separate multivariate analysis of variance was performed for each type of delay.

The ANOVA for the case with average queue delay as the dependent variable revealed there were main effects for two factors (major and minor street volume), $\underline{F}(3,15) = 20.66$, $\underline{p} < .001$. There was also a two-way interaction between these two volume parameters. The ANOVA for the case with average stopped delay as the dependent variable produced similar results, $\underline{F}(3,15) = 16.98$, $\underline{p} < .001$.

None of the other five factors, including the percent of right turns from the minor street, accounted for a significant amount of the variation in average queue delay or average stopped delay, either as a main effect, or as a part of a two-way interaction. Higher order interactions were not tested. The ANOVA table B-11 for both of these analyses is included in Appendix B.

4.4 Discussion

The right turn factor did not prove to be statistically significant with respect to impacting minor street delay in any of the analyses conducted for this study. The only traffic related factors that did have a significant effect on delay were the major and minor street volumes. This was expected, as volume is the primary predictor of delay at stop sign controlled intersections (Volk 1956).

A geometric factor that proved to be significant was lane configuration (number of lanes on the major and minor street). This was also expected, as the number of lanes determines the approach capacity. Approach capacity, in turn, is related to volume and delay via the concept of volume to capacity ratio (V/C ratio) (ITE 1982).

The other geometric factor that proved significant in the simulation analyses was right turn channelization. This is understandable, as channelization, in effect, isolates right-turning vehicles from all other vehicles in the traffic streams on all four approaches.

The fact that neither the right turn factor, nor any of the other secondary traffic factors, did not significantly affect delay from a statistical standpoint in this simulation study does not mean they do not contribute to delay in the TEXAS model. It simply indicates that relative to other parameters, these factors do not account for a statistically significant amount of the variation in the model's measure of effectiveness for the scenarios tested.

Perhaps the most important parameters in determining the statistical outcome of this study were the random seed numbers. As discussed in Chapter 3 of this thesis, the random seeds control the driver/vehicle combinations and the time of their entry into the simulated intersection. These numbers account for the variation in the model's performance from one replication run to another.

The results of any one model run were indeed highly variable. Given identical input values, with a change of only the random seed numbers, the simulation model measure of effectiveness (average queue delay per vehicle) varied by as much as 400 percent.

The purpose of using replications was to minimize the effects of model variability and to produce repeatable output statistics. The mean and standard deviation of the delay from one set of replications should be approximately the same as the mean and standard deviation from any other set of replications.

This was not necessarily the case, particularly for the analyses conducted at higher volume levels. As the values input for approach volumes increased, the variability in the simulation results increased. This trend is noticeable in Figure 4-1, where the plot of delay versus simulation time fluctuates much more for the high volume level than for the low volume level.

Table 4-5 and Table 4-6 also illustrate this pattern. As the the minor street volume increased (down the tables) and as the major street volume increased (from one table to the other), the standard deviation of average queue delay increased for the replication sets. Standard deviation is an indicator of the variation of individual model run output values of delay versus the average for the entire replication set of ten runs.

The standard deviation of average queue delay (18.68 seconds) for the highest minor street volume level in Table 4-6 is 69% of the mean for that particular replication set (27.04 seconds). Also, the average value of the standard deviation (10.10 seconds) for the entire table is approximately 52% of the average mean (19.40 seconds). This compares to 26% for the lower volume level used in this analysis.

The amount of variation in the value of average queue delay within a set of replications affects the level at which a factor being analyzed becomes significant. The greater the amount of variation of the dependent variable (delay) within the replication sets, the greater the effect required for the independent variable (right turn factor) being analyzed before it becomes significant. In the scenarios tested for the first set of right turn analyses, the effect of the right turn factor on side street queue delay was not great enough to overcome the variation due to the randomness of the TEXAS model.

In the second right turn factor analysis, the portion of the model's variability caused by the random seed numbers was reduced using repeated measures, or a block design. The results agreed with the findings from the first set of analyses. That is, the volume on the major and minor streets are the only traffic parameters that significantly effect delay in the TEXAS model.

CHAPTER 5

FIELD DATA ANALYSES

It was of interest to conduct an analysis of data collected at actual intersections to measure the effect of the right turn factor. The intent was not to compare the results from the field data analysis with the results from the TEXAS model analysis. Rather the goal was to perform a complimentary, or parallel, empirical study which would supplement the theoretically-based computer simulation study.

The approach to the field data analysis was altogether different than for the TEXAS analyses by virtue of the level of controllability of the study parameters. With computer simulation, the user can set the values of any number of parameters--either varying them over a predetermined range, or holding them constant in order to isolate the effect of any one variable. This method is virtually impossible to conduct when using field data, as a prohibitively large number of observations would be necessary to obtain the required data points.

The best approach for evaluating the effect of the right turn factor when using field data is to measure the significance of its contribution to delay within the context of a dynamic intersection. Hence, analyze the right turn factor over the range of all the other relevant traffic and geometric variables, as opposed to the attempting to control their values.

5.1 Database Description

The first task was either to locate a suitable existing database or to collect a sufficient quantity of field data to create a new one. Fortunately, a comprehensive database was located that contained most of the relevant traffic and geometric parameters of interest for this study. The field data were collected as part of a National Cooperative Highway Research Program study entitled "Peak-Hour Traffic Signal Warrant" (TRB 1982).

The NCHRP database consisted of 817 25-minute observations collected at 241 intersections in six U.S. cities. The cities included in the study were Atlanta, Denver, Hartford, Phoenix, Tucson, and Washington, D.C. An average of four observations were made at each intersection just before, during, and just after the peak demand traffic flow.

Two-way stop sign control accounted for 115 of the study intersections. The remaining intersections were controlled either by a traffic signal or a police officer. The 72 stop controlled intersections that had a three approach, or "T", configuration were not included in the database for this study, as they have a different traffic flow pattern.

This left 42 intersections, with a total of 124 observations, suitable for this analysis. The breakdown of these observations by the number of lanes on the major and minor street is shown in Table 5-1.

In addition to the geometric configuration, the parameters included in the NCHRP database of interest to this study were:

- Intersection identification number (including the identity of city)
- Observation number
- Posted speed limit on the major street (grouped into three levels)
- Distance to the nearest traffic signal to the left and right of the study approach (grouped into three distance ranges and listed as a separate parameter for each direction)
- Main street volume expressed as an hourly flow rate (vph) for both travel directions combined
- Side street volume expressed as an hourly flow rate (vph) for the study approach
- Average stopped delay per vehicle for the side street study approach
- Average queue on the side street study approach

While the right turn percentage for the study approach was not included in the database obtained for this study, it could be calculated from a set of sheets that contained intermediary computations for the final database. The percent of left turns from the study approach or the percent turns (left or right) from the main street were not available.

TABLE 5-1. NUMBER OF OBSERVATIONS FOR EACH LANE COMBINATION IN DATABASE.

NUMBER OF LANES ON SIDE STREET	NUMBER OF LANES ON MAJOR STREET: 1	NUMBER OF LANES ON MAJOR STREET: 2 OR MORE	TOTAL OBSERVATIONS
1	28	44	72
2	13	39	52
TOTAL OBSERVATIONS	41	83	124

A new database was created using the 124 observations collected at 4-approach, 2-way stop sign controlled intersections. The 10 variables listed above were included for each observation. A copy of this database is included in Appendix C.

The average values of the various parameters included in the database, as well as their range, were of interest in this study. The mean and standard deviation for the relevant variables are listed in Table 5-2. The statistics are listed separately for the cases of one and two lanes on the side street approach.

5.2 Method

The first step was an evaluation of which database variables to include in the analysis. While it was important to include the most influential variables with respect to predicting delay, too many variables would reduce the accuracy of the results. The determination was made by combining a review of the factors in the MUTCD traffic signal warrants with a correlation analysis. Correlation analysis attempts to measure the strength of relationships between two variables by means of a correlation coefficient. A correlation coefficient with an absolute value of 1.0 indicates a perfect linear relationship, while a value of 0.0 indicates no linear relationship.

Following the evaluation of variables, a regression analysis was performed to measure the effect of the right turn factor on average stopped delay. The regression procedure chosen involved a two step entry of the independent variables. In the first step, all of the relevant variables, with the exception of the right turn percentage, were entered into the regression model simultaneously. The right turn factor was then entered in separate step.

This approach allowed the determination of the proportion of variance in delay accounted for by the right turn factor after the effects of the other independent variables have been accounted for. Expressed in terms of sample coefficients of determination,

$$R^2_{change} = R^2_2 - R^2_1$$

where $R^2_2 =$ the sample coefficient of determination for the second step

$R^2_1 =$ the sample coefficient of determination for the first step

TABLE 5-2 MEAN AND STANDARD DEVIATION OF VARIABLES IN THE DATABASE.

Variable (Unit)	Number of Lanes on the Side Street			
	1		2	
	Mean	Std. Dev.	Mean	Std. Dev.
Minor Street Right Turns (%)	60.0	28.7	49.5	31.9
Major Street Volume (Two-Way) (vph)	1370.5	736.4	1675.7	1128.9
Minor Street Volume (One-Way) (vhp)	208.2	144.0	291.2	209.8
Major Street Speed Limit (mph)	45.8	17.9	46.2	19.7
Lanes on Major Street (number)	1.8	0.7	2.0	0.8
Distance to Signal (feet) -Left	3417	1790	3500	1810
-Right	3806	1684	3269	1682
Average Stopped Delay (sec)	28.9	25.9	55.3	87.6

R^2_{change} = the increase in the sample coefficient of determination when the variable from the second step is added.

A large change in R^2 indicates that a variable provides unique information about the dependent variable that is not available from the other independent variables in the equation (Norusis 1986). For this study, the percentage of vehicles turning right from the side street was the variable entered in the second step and average stopped delay was the dependent variable. The statistical significance of the R^2 change was measured using a partial F-test.

5.3 Results

A review of the traffic signal volume warrants in the MUTCD identified the geometric configuration, the volume on the major and minor streets, and the major street speed limit as important data. The distance to the nearest signal is listed as one of more than a dozen factors to consider when constructing an intersection conditions diagram. None of these other secondary factors were available in the NCHRP database.

A correlation analysis was performed for the eight independent variables, as well as the dependent variable (average stopped delay). The results of this analysis (Table 5-3) showed an insignificant level of correlation between the two signal distance variables and the average delay (.059 and -.098). This indicated that neither parameter would be important as predictors of side street delay in a regression analysis.

Based on the results of the correlation analysis, along with the fact that the distance to the nearest traffic signal is considered a secondary factor in the MUTCD criteria, these two variables were not included in the final regression analysis.

Two additional variables (number of lanes on the major street and side street volume), while not highly correlated with delay, were included in the final analysis, as both are primary factors in the MUTCD criteria. The major street volume, speed limit, and the number of lanes of the side street were all correlated with delay, as was the percent right turns. These four variables were included in the regression model.

TABLE 5-3. CORRELATION MATRIX FOR VARIABLES IN DATABASE

Variable	Average Delay	Percent Right Turns	Lanes on Main Street	Lanes on Side Street	Speed Limit	Main Street Volume	Side Street Volume	Signal Dist. (Left)	Signal Dist. (Right)
Average Delay	1.000	.278*	.023	-.214*	.492*	.278*	.062	.059	-.098
Percent Right Turn	-	1.000	.052	.171	.234*	.289*	.097	.157	-.023
Lanes on Main Street	-	-	1.000	-.146	.049	.694*	-.376*	-.451*	-.431
Lanes on Side Street	-	-	-	1.000	-.048	-.163	-.230*	-.023	.156
Speed Limit	-	-	-	-	1.000	.210*	-.119	.369*	-.083
Main Street Volume	-	-	-	-	-	1.000	-.256*	-.213*	-.323*
Side Street Volume	-	-	-	-	-	-	1.000	.300*	.041
Signal Dist. (Left)	-	-	-	-	-	-	-	1.000	.352*
Signal Dist. (Right)	-	-	-	-	-	-	-	-	1.000

Note: (*) Indicates a Significant Correlation at a = .01.

While it was desired to conduct a separate analysis for each of the four lane combinations, this was not possible due to an insufficient number of data points for two of the cases (Table 5-1). Approximately 30 data points were needed for a statistically valid analysis.

Therefore, analyses were performed only for the two side street geometric configurations. There were 72 data points for the first case of 1 lane on the side street and 52 data points for the second case of 2 lanes on the side street.

Preliminary regression analyses for the combined case (1 and 2 lanes on the side street) revealed a non-linear relationship between the dependent, or response, variable (average stopped delay) and the independent, or regressor, variables. This non-linearity was most apparent from the cumulative normal probability plot of the residuals (Fig. 5-1).

Residuals that follow a cumulative normal distribution should plot as a straight line. The double-bowed pattern of the residuals for the first case indicated a transformation of one or more of the variables was needed to eliminate the nonnormality (Montgomery and Peck 1982).

The first transformation performed was to take the natural logarithm (ln) of delay. The justification for this transform stems from the exponential relationship between volume and delay, as two of the five regressors in the model were volumes. The regression analysis was rerun with this transformation.

As illustrated in Fig. 5-2, the residuals for this case plotted virtually as a straight line. This indicated that no additional variable transformations were necessary and that the natural logarithm of delay should be used for this study.

The first step of the regression analysis for the case of one lane on the side street resulted in an R^2 of .2876. Thus, the first four variables entered into the equation accounted for approximately 28.8 percent of the variation in delay. When the percent of right turns was entered into the regression model in the second step, the R^2 value increased to .2907. The resulting value of R^2_{change} was .0031. A partial $\underline{F}$-test revealed that this change was not significant, $F_{change} = .2896$, N.S. The summary statistics for this analysis are included in Appendix D.

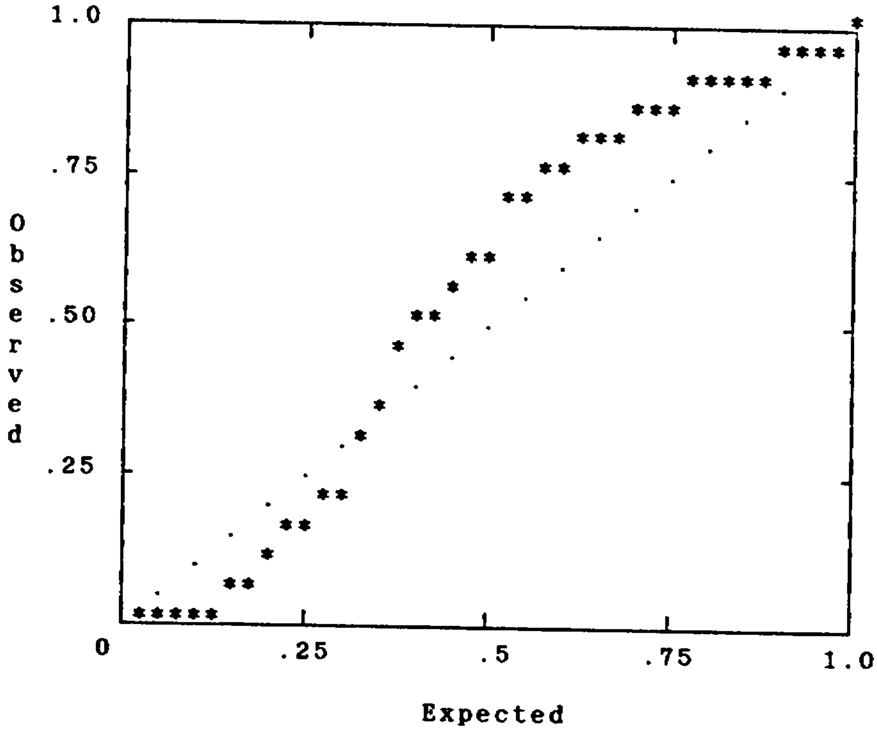


Fig. 5-1. Cumulative Normal Probability Plot of Standardized Residuals for the Non-Transformed Case.

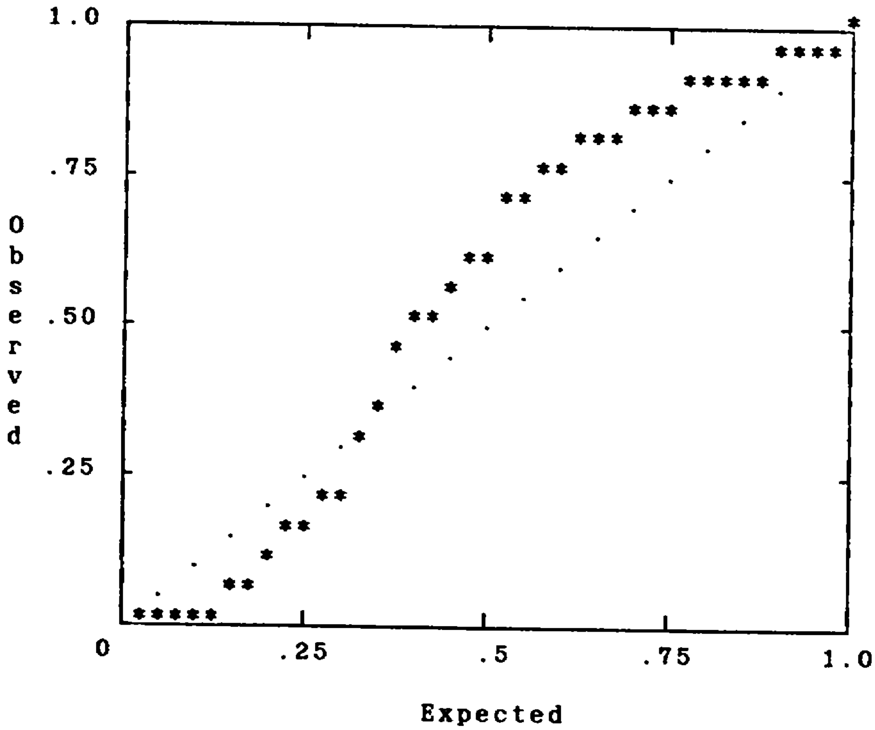


Fig. 5-1. Cumulative Normal Probability Plot of Standardized Residuals for the Non-Transformed Case.

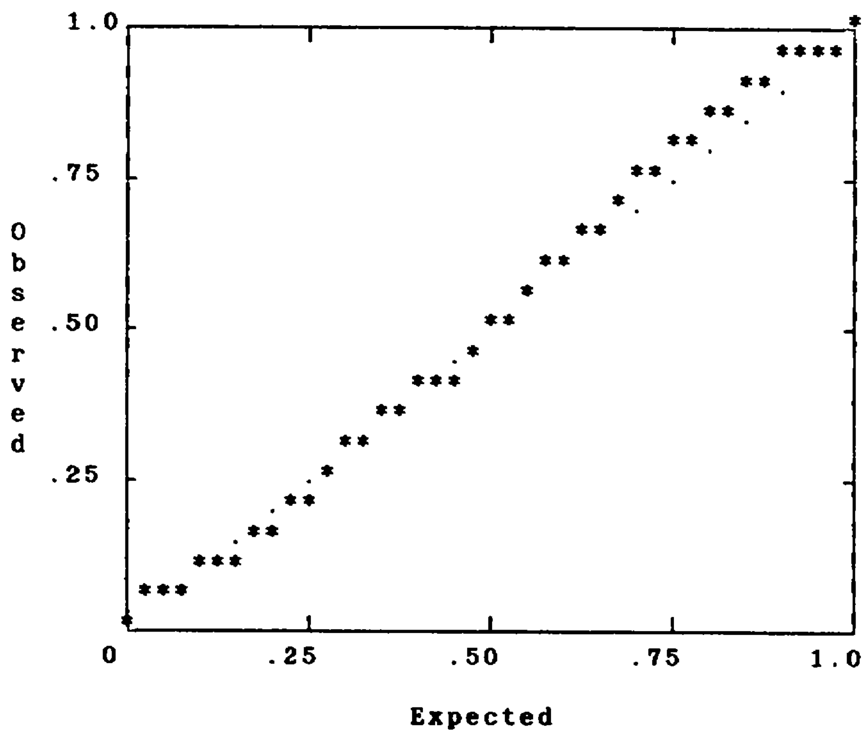


Fig. 5-2. Cumulative Normal Probability Plot of Standardized Residuals for the Transformed Case.

The same regression procedure was performed for the data set with two lanes on the side street. The R^2 for the first step, with four independent variables in the model, was .6198. The R^2 with the addition of the percent right turn variable in the second step was .7591. The resulting R^2_{change} of .1393 was significant, $F_{change} = 26.6107$, $p < .001$. This indicated that the right turn factor accounted for approximately 14% of the variation in the average delay. The summary statistics for this analysis are also included

5.4 Discussion

The finding that the right turn factor is significant for the case of two lanes on the side street, but not for the one lane case, is not intuitively obvious. Generally, delay for the one lane case would be suspected to be more sensitive to variation in the percent of right turns, as well as, to variation in the other traffic and geometric parameters.

With a one lane approach, all the vehicles must generally remain in a single queue while waiting to enter the intersection. Therefore, each vehicle in the queue incurs the cumulative delay of every other vehicle already in the queue when it arrives. The desired traffic maneuver of each vehicle as it reaches the stop bar theoretically affects the average delay of all vehicles in the queue.

As discussed in Chapter 2 of this thesis, the time required to find an acceptable gap and enter the intersection has been found to vary according to the type of movement (i.e., left turn, right turn, through). Likewise, the other traffic and geometric factors that effect delay would also be expected to be more critical for the one lane case.

However, for this set of data, the major and minor street volumes, the speed limit, the number of lanes on the major street, and the percent of right turns from the minor street account for only 29% of the variation in the delay for the 1 lane case. This compares to almost 76% for the 2 lane case.

The fact that the same regression procedure resulted in such different levels of success in predicting the same response variable from the same regressors indicated that the two data subsets originated from intersections operating under different dynamics. Perhaps an additional independent variable, transformation, or combination of variables would improve the R^2 in the model for the one lane case. However, additional statistical analyses using this database would not change the findings concerning the effect of the right turn factor.

A possible explanation for the unexpected results of this analysis can be found from an investigation of the origin of the source database. The field data used in this study was collected as part of a NCHRP peak-hour traffic signal warrant evaluation. The intersections studied in the NCHRP project tended to have certain common geometric and traffic characteristics. In particular, two general types of intersections were found among the 241 that were included in the database.

The first type was an industrial parking lot feeder road that intersected with a higher volume arterial or collector street. The second type was an urban intersection of two streets heavily impacted by commuter traffic. Both types of intersections exhibited highly peaked traffic demand profiles.

The duration and extent of the resulting delays were highly variable. In some cases the delay was of extremely short duration, but with a high average delay per vehicle. Other cases had delay that extended over a longer period of time, but with a considerably smaller average delay per vehicle (TRB 1982).

The traffic parameters were also highly variable within, as well as, among the study intersections. As Table 5-2 shows, the volumes on both the major and minor streets for the portion of NCHRP data used in this analysis had standard deviations greater than 50 percent of the mean. The minor street right turn percentage exhibited a comparable degree of variation.

Another important characteristic of the database was the trend toward high means for the volumes and the percent right turns (Table 5-2). The mean volumes for both cases of one and two lanes on the side street, exceed the requirements for installing a traffic signal as stipulated in the Peak Hour Volume Warrant (Warrant Number 11), as contained in the current edition of the MUTCD (FHWA 1988).

The mean right turn percentages were 60.0% and 49.5% for the case of 1 lane and 2 lanes on the side street, respectively. Both of these values are higher than the maximum values used in the TEXAS analyses (30% and 40%). Additionally, the maximum percent of vehicles turning right at typical urban intersections is reported as 30% (ITE 1982).

One explanation for the significance of the right turn factor in predicting delay for the two lane approach case is related to the high percentage of right turns. When approximately one-half of the vehicles on the stop approach were turning right (as was the case during the average observation), it is highly probable that a large portion of the other half of the approach traffic were turning left. While the percent of left turns from the minor approach was not recorded in the NCHRP field data, certain inferences can be made.

By virtue of the types of intersections where data was collected, a majority of the side street traffic would desire to enter the major street (i.e., leaving a factory parking lot via a feeder street that intersects with an arterial street). In most cases, there would be little demand to remain on the feeder street by traveling through the intersection.

For this scenario, both lanes on the stop sign controlled approach operate as an exclusive turn lane--one for left turns and one for right turns. Essentially, the intersection functions as a three approach, or T-configuration. Thus, from observation to observation and from intersection to intersection, the only variation in vehicle movements is a change in the proportion of right turns to left turns.

Consequently, the percent of each type of turn would have a major impact on the average delay per vehicle. When the proportion of left turns to right turns was high, for example, the average delay would be expected to be much greater than when the proportion was low. This effect would be less pronounced when a higher percentage of the side street traffic was traveling through the intersection.

The fact that the field data was not necessarily representative of a "typical" four-leg intersection operating under two-way stop sign control, does not invalidate the results. Rather, it limits the applicability of the findings to similar intersections that exhibit a high degree of traffic volume peaking.

The question that remains unanswered at this point is: have the findings of the field data analysis addressed the objective of this research? The answer is both yes and no. Yes the research concluded that right turning traffic could have significant impact on vehicular delay for a certain intersection configuration. No the research did not produce numerical standards for signal warrants that take into consideration right turn traffic.

The significant finding of the two lane approach indicated that a positive correlation existed between right turn percentages and average delay per vehicle. This finding does not agree with the MUTCD statement about subtraction of right turn volume when conducting signal warrant analysis. It seems appropriate that the minor street volume be adjusted upward to take into consideration the impact of right turning traffic. The other alternative is to revise the signal warrants boundaries to reflect changes in the right turn traffic percentages.

5.5 MUTCD Volume Warrants

Findings of the field data analysis were used to revise the MUTCD volume warrants. The minimum vehicular volume warrant (Warrant 1) recommends a minimum major street volume (total both approaches) of 600 vehicles per hour and a minor street volume (one direction only) of 200 vehicles per hour for an intersection of 2 or more lanes on the major street and 2 or more lanes on the minor street. The manual does not document the percentage of right turn traffic applicable to these figures. Previous studies have indicated that the maximum percent of vehicles turning right at typical urban intersections is 30% (ITE 1982). This value was assumed as the MUTCD percentage for right turn traffic.

The second regression equation of Appendix D was used to derive the revised volume warrants. In this equation, the main volume was 600, the minor volume was 200 and the right turn percentage was 30. The average delay per vehicle was calculated as the left side of the equation. The delay was held constant, and the main volume was kept at 600, two right turn percentages 10 and 20 were substituted into the equation. The minor street volumes were found to be 935 and 565 vehicles per hour for 10 and 20 percents of right turn, traffic, respectively. This means that Warrant 1 is satisfied when, for each of any 8 hours of an average day, the traffic volumes of 600 on major street, 935 on minor street are exceeded with 10 percent right turn traffic on the minor street.

The same procedure was applied for Warrant 2 of the MUTCD. The proposed vehicular volumes are 900 and 100 for major street and minor street approaches, respectively. The minor street volumes were determined to be 835 and 465 for 10 and 20 percent right turns, respectively.

As for Warrant 9, four hour volume Warrant, the curve relating the major street total volume to the minor street high volume approach, 2 or more lanes for both major and minor streets (Figure 4-5, of the MUTCD), page 4C-9 was used to produce two other curves. The same regression equation was applied to selected points on the curve and the other two curves represented 10 and 20 percents right turns on the minor street. All three curves are shown in Figure 5.3. The four hour volume warrant is satisfied when each of any four hours of an average day the plotted points representing the vehicles per hour on the major street (total of both approaches) and the corresponding vehicles per hour on the higher volume minor street approach (one direction only) all fall above the curve in Figure 5.3 for the appropriate turning percentage.

The peak hour volume Warrant, Warrant 11, is also satisfied when the plotted point representing the vehicles per hour on the major street (total of both approaches) and the corresponding vehicles per hour on the higher volume minor street approach (one direction only) all fall above the curve in Figure 5.4 for the appropriate turning percentage.

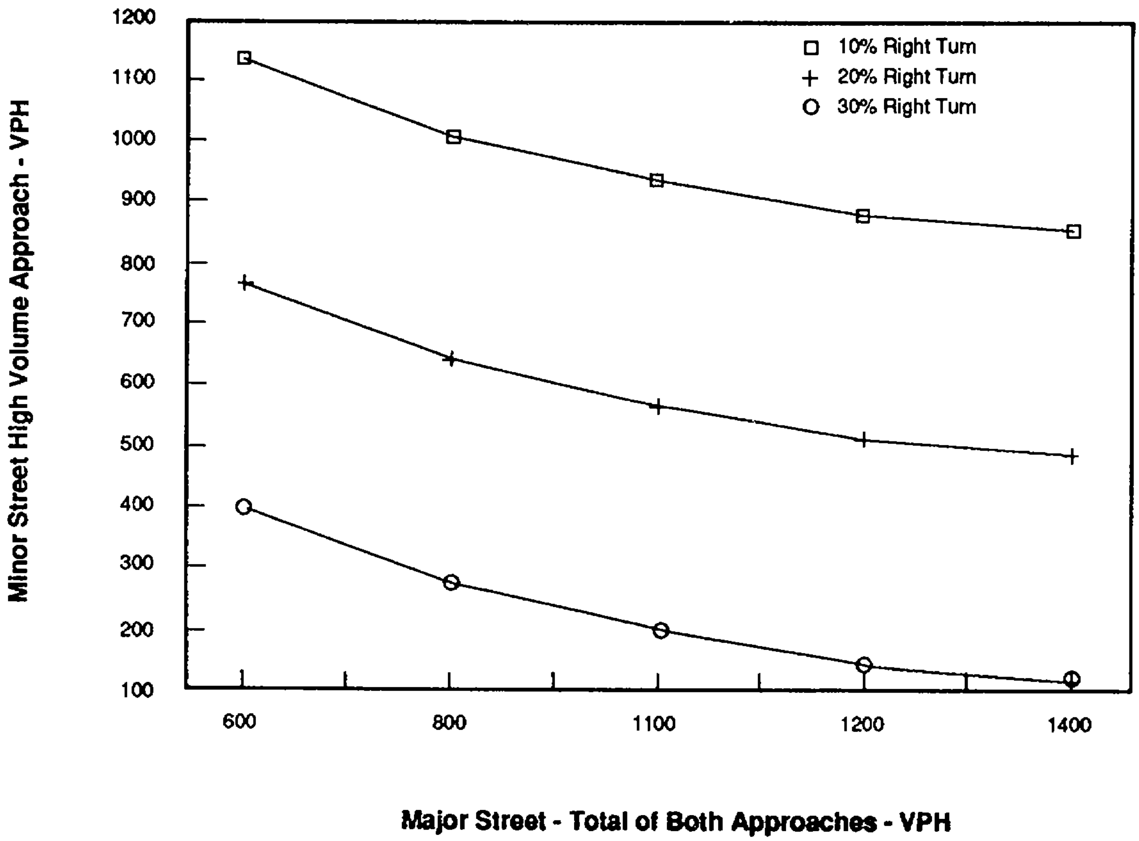


Figure 5.3 Four Hour Volume Warrant

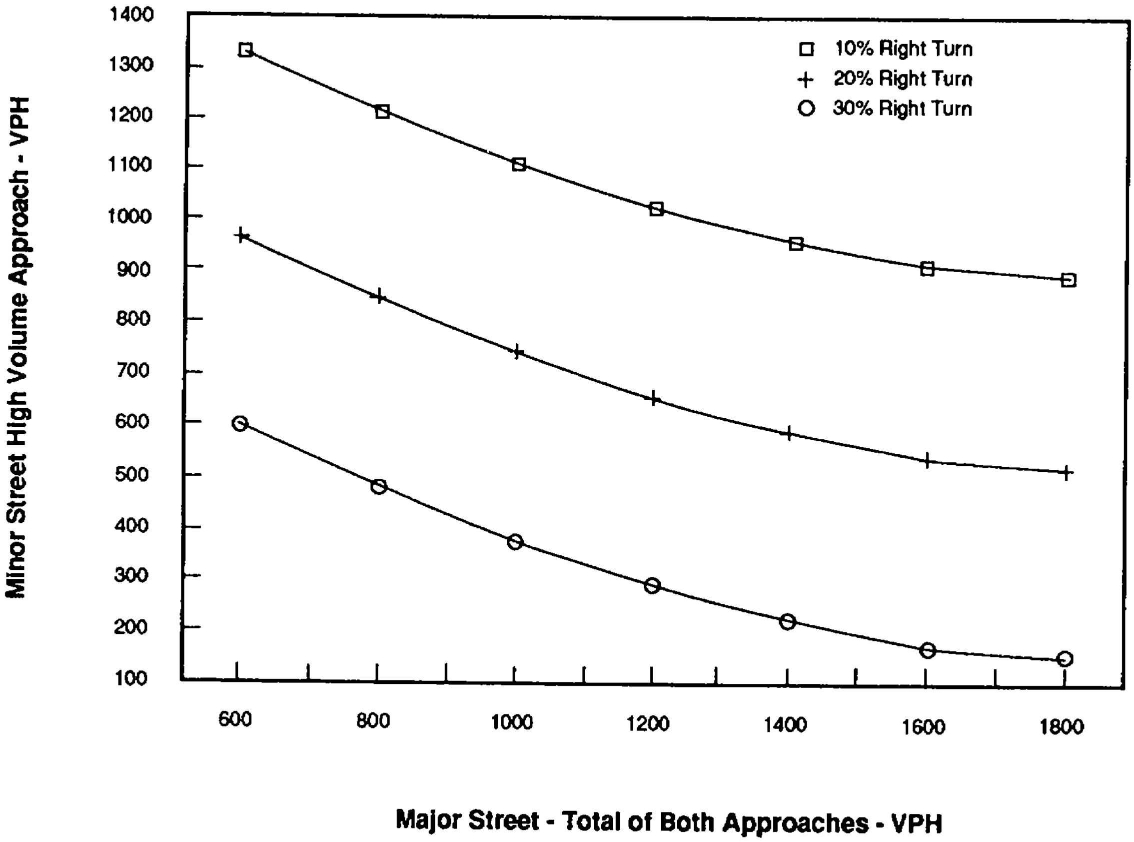


Figure 5.4. Peak Hour Volume Warrant

CHAPTER 6

CONCLUSION

The justification for studying the right turn factor has been well established in previous studies in the areas of gap acceptance, delay, intersection capacity, and traffic signal warrants. While these studies have established the theoretical basis for treating right turning vehicles separately from other movements in applying signal warrants, a review of current practice revealed a lack of uniformity with respect to procedure.

The purpose of this study was to evaluate the significance of the right turn factor with respect to a common measure of intersection performance--average vehicular delay. In the process, a better understanding of the advantages and disadvantages of the two study methods was desired, as the eventual goal will be to use one of the approaches to develop a comprehensive set of numerical guidelines for the treatment of right turns.

The simulation method utilizing the TEXAS model was not fruitful. The principle findings for this set of analyses were that the only significant variables within the context of the test scenario were the major and minor street volumes and the random seed numbers. The variation within replication sets was extremely high, particularly for simulation runs at higher volumes.

To what degree this variation exists at actual intersections in the field is not known. The TEXAS model analyses conducted in this study were based on hypothetical intersections. A validation of the simulation model was not performed, as it was assumed the results were reasonably close to those that would be obtained from field data for the same geometric scenarios and traffic conditions, had it been collected and analyzed.

Conducting a thorough validation of TEXAS was beyond the scope of this study. The task of identifying candidate intersections, as well as collecting, reducing, and analyzing the required amount of data would constitute a major undertaking. Even so, there would be no guarantee of locating intersections where a sufficient amount of "quality" data could be gathered.

The possibility of using a different computer simulation model to conduct the analyses for this study was also investigated. The only other available model capable of microscopic simulation of a single intersection was the UTCS model (known as NETSIM). A set of trial runs were conducted with NETSIM using one of the same intersection scenarios and the same values of traffic parameters used in the TEXAS analysis.

While the variation within the replication sets was smaller for the NETSIM runs, the level of sensitivity to the various input parameters (percent right turns, volume, etc.) was lower. Thus, it was concluded that using the NETSIM model for this study, or to develop a set of numerical standards, would not be productive.

The data set used for this study made it possible to evaluate the effect of right turns for two particular intersection configurations operating primarily under peak-hour traffic conditions. However, these were specialized cases which do not necessarily have the same operational dynamics as intersections that are subject to less of a peaking characteristic in traffic volume. Therefore, it may be inappropriate to generalize the findings from this study to all intersections fitting the same geometric configuration descriptions.

Perhaps if supplemented by additional field data collected at stop sign controlled intersections with more uniform operating characteristics (i.e., less peaking in traffic volume), this data set would be representative of a "typical" four-leg intersection. Also, a larger database would allow analyses of more geometric configurations.

Indeed, at this point in time, the use of field data to further evaluate the right turn factor appears much more promising than computer traffic simulation. Given a large enough set of data collected for a variety of geometric scenarios, a thorough understanding of the relationship between the right turn factor and delay would be possible. Furthermore, through the use of predictive mathematical models, this data could be used to develop a comprehensive set of numerical standards for treating right turns in the application of the MUTCD traffic signal warrants.

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APPENDIX A

TEXAS MODEL INPUT AND OUTPUT EXAMPLES

GEOMETRY PROCESSOR FOR THE TEXAS TRAFFIC SIMULATION PACKAGE (V3.00)
GEOPRO COPYRIGHT (c) 1989 BY THE UNIVERSITY OF TEXAS AT AUSTIN

2X2 INTERSECTION

TABLE 1 - LISTING OF INBOUND APPROACH NUMBERS

1
2
3
4

TOTAL NUMBER OF INBOUND APPROACHES = 4

TABLE 2 - LISTING OF OUTBOUND APPROACH NUMBERS

5
6
7
8

TOTAL NUMBER OF OUTBOUND APPROACHES = 4

TOTAL NUMBER OF INBOUND AND OUTBOUND APPROACHES = 8

TABLE 3 - LISTING OF APPROACHES

APPROACH NUMBER ------------------	1
APPROACH AZIMUTH -----------------	180
BEGINNING CENTERLINE X COORDINATE -	1237
BEGINNING CENTERLINE Y COORDINATE -	2057
SPEED LIMIT (MPH) ----------------	30
NUMBER OF DEGREES FOR STRAIGHT ----	20
NUMBER OF DEGREES FOR U-TURN ------	10
NUMBER OF LANES ------------------	2

```
LANE IL IBLN WIDTH ---LANE GEOMETRY--- LEGAL TURNS
  1   1   1   12       0    1  799  800    ( L   )  (MEDIAN LANE)
  2   2   2   12       0  800    0  800    ( LSR)  (CURB LANE)

APPROACH NUMBER -------------------       8
APPROACH AZIMUTH ------------------     270
BEGINNING CENTERLINE X COORDINATE -    1193
BEGINNING CENTERLINE Y COORDINATE -    1225
SPEED LIMIT (MPH) -----------------      30
NUMBER OF DEGREES FOR STRAIGHT ----      20
NUMBER OF DEGREES FOR U-TURN ------      10
NUMBER OF LANES -------------------       1

LANE IL IBLN WIDTH ---LANE GEOMETRY--- LEGAL TURNS
  1   3   0   12       0  250    0  250    ( LSR)  (MEDIAN LANE)

APPROACH NUMBER -------------------       2
APPROACH AZIMUTH ------------------     270
BEGINNING CENTERLINE X COORDINATE -    2069
BEGINNING CENTERLINE Y COORDINATE -    1225
SPEED LIMIT (MPH) -----------------      30
NUMBER OF DEGREES FOR STRAIGHT ----      20
NUMBER OF DEGREES FOR U-TURN ------      10
NUMBER OF LANES -------------------       1

LANE IL IBLN WIDTH ---LANE GEOMETRY--- LEGAL TURNS
  1   4   3   12       0  800    0  800    ( LSR)  (MEDIAN LANE)
```

```
APPROACH NUMBER ------------------     5
APPROACH AZIMUTH -----------------     0
BEGINNING CENTERLINE X COORDINATE - 1237
BEGINNING CENTERLINE Y COORDINATE - 1257
SPEED LIMIT (MPH) ----------------    30
NUMBER OF DEGREES FOR STRAIGHT ----   20
NUMBER OF DEGREES FOR U-TURN ------   10
NUMBER OF LANES ------------------     1

LANE IL IBLN WIDTH ---LANE GEOMETRY--- LEGAL TURNS
  1   5   0   12      0  250    0  250   ( LSR)  (MEDIAN LANE)

APPROACH NUMBER ------------------     3
APPROACH AZIMUTH -----------------     0
BEGINNING CENTERLINE X COORDINATE - 1225
BEGINNING CENTERLINE Y COORDINATE -  393
SPEED LIMIT (MPH) ----------------    30
NUMBER OF DEGREES FOR STRAIGHT ----   20
NUMBER OF DEGREES FOR U-TURN ------   10
NUMBER OF LANES ------------------     2

LANE IL IBLN WIDTH ---LANE GEOMETRY--- LEGAL TURNS
  1   6   4   12      0    1  799  800   ( L  )  (MEDIAN LANE)
  2   7   5   12      0  800    0  800   ( LSR)  (CURB LANE)

APPROACH NUMBER ------------------     6
APPROACH AZIMUTH -----------------    90
BEGINNING CENTERLINE X COORDINATE - 1269
BEGINNING CENTERLINE Y COORDINATE - 1225
SPEED LIMIT (MPH) ----------------    30
NUMBER OF DEGREES FOR STRAIGHT ----   20
NUMBER OF DEGREES FOR U-TURN ------   10
NUMBER OF LANES ------------------     1
```

```
LANE IL IBLN WIDTH ---LANE GEOMETRY--- LEGAL TURNS
  1   8   0   12      0  250    0  250   ( LSR)  (MEDIAN LANE)

APPROACH NUMBER -------------------    4
APPROACH AZIMUTH ------------------   90
BEGINNING CENTERLINE X COORDINATE -  393
BEGINNING CENTERLINE Y COORDINATE - 1225
SPEED LIMIT (MPH) -----------------   30
NUMBER OF DEGREES FOR STRAIGHT ----   20
NUMBER OF DEGREES FOR U-TURN ------   10
NUMBER OF LANES -------------------    1

LANE IL IBLN WIDTH ---LANE GEOMETRY--- LEGAL TURNS
  1   9   6   12      0  800    0  800   ( LSR)  (MEDIAN LANE)

APPROACH NUMBER -------------------    7
APPROACH AZIMUTH ------------------  180
BEGINNING CENTERLINE X COORDINATE - 1225
BEGINNING CENTERLINE Y COORDINATE - 1193
SPEED LIMIT (MPH) -----------------   30
NUMBER OF DEGREES FOR STRAIGHT ----   20
NUMBER OF DEGREES FOR U-TURN ------   10
NUMBER OF LANES -------------------    1

LANE IL IBLN WIDTH ---LANE GEOMETRY--- LEGAL TURNS
  1  10   0   12      0  250    0  250   ( LSR)  (MEDIAN LANE)

TABLE  4  -  LISTING OF ARCS (FOR PLOTTING ONLY)

ARC NUMBER ------------------------    1
CENTER X COORDINATE --------------- 1193
CENTER Y COORDINATE --------------- 1257
BEGINNING AZIMUTH -----------------   90
SWEEP ANGLE -----------------------   90
RADIUS OF ARC ---------------------   20
ROTATION FROM BEGINNING AZIMUTH ---CLOCKWISE
```

ARC NUMBER ----------------------- 2
CENTER X COORDINATE --------------- 1269
CENTER Y COORDINATE --------------- 1257
BEGINNING AZIMUTH ----------------- 180
SWEEP ANGLE ----------------------- 90
RADIUS OF ARC --------------------- 20
ROTATION FROM BEGINNING AZIMUTH ---CLOCKWISE

ARC NUMBER ----------------------- 3
CENTER X COORDINATE --------------- 1269
CENTER Y COORDINATE --------------- 1193
BEGINNING AZIMUTH ----------------- 270
SWEEP ANGLE ----------------------- 90
RADIUS OF ARC --------------------- 20
ROTATION FROM BEGINNING AZIMUTH ---CLOCKWISE

ARC NUMBER ----------------------- 4
CENTER X COORDINATE --------------- 1193
CENTER Y COORDINATE --------------- 1193
BEGINNING AZIMUTH ----------------- 0
SWEEP ANGLE ----------------------- 90
RADIUS OF ARC --------------------- 20
ROTATION FROM BEGINNING AZIMUTH ---CLOCKWISE

TOTAL NUMBER OF ARCS = 4

TABLE 5 - LISTING OF OPTIONS AND ADDITIONAL DATA

PRIMARY PATHS SELECTED

A STRAIGHT LINE WILL BE USED FOR A PATH WITH A RADIUS GT 500.00 FT

PROGRAM CHECKS TO SEE IF THE CENTER TO CENTER DISTANCE
BETWEEN VEHICLES BECOMES LESS THAN OR EQUAL TO 10 FEET

TABLE 6 - LISTING OF PATHS

```
PATH   1 GOES FROM LANE 1 OF APPROACH  1 TO LANE 1 OF APPROACH  6
   LENGTH OF PATH =  60 FEET AND SPEED OF PATH = 20 FEET PER SECOND
   NUMBER OF CONFLICTS =  7 AND TURN CODE FOR PATH IS LEFT
   CONFLICT ENTRY NUMBERS ORDERED BY DISTANCE DOWN THIS PATH ARE
          6    3    4    2    1    5    7

PATH   2 GOES FROM LANE 2 OF APPROACH  1 TO LANE 1 OF APPROACH  6
   LENGTH OF PATH =  72 FEET AND SPEED OF PATH = 20 FEET PER SECOND
   NUMBER OF CONFLICTS =  9 AND TURN CODE FOR PATH IS LEFT
   CONFLICT ENTRY NUMBERS ORDERED BY DISTANCE DOWN THIS PATH ARE
          9   13   12    8   10   14    1   11   15

PATH   3 GOES FROM LANE 2 OF APPROACH  1 TO LANE 1 OF APPROACH  7
   LENGTH OF PATH =  64 FEET AND SPEED OF PATH = 44 FEET PER SECOND
   NUMBER OF CONFLICTS =  7 AND TURN CODE FOR PATH IS STRAIGHT
   CONFLICT ENTRY NUMBERS ORDERED BY DISTANCE DOWN THIS PATH ARE
         17   19   20   18   21   16   22

PATH   4 GOES FROM LANE 2 OF APPROACH  1 TO LANE 1 OF APPROACH  8
   LENGTH OF PATH =  41 FEET AND SPEED OF PATH = 17 FEET PER SECOND
   NUMBER OF CONFLICTS =  3 AND TURN CODE FOR PATH IS RIGHT
   CONFLICT ENTRY NUMBERS ORDERED BY DISTANCE DOWN THIS PATH ARE
         23   24   25

PATH   5 GOES FROM LANE 1 OF APPROACH  2 TO LANE 1 OF APPROACH  5
   LENGTH OF PATH =  41 FEET AND SPEED OF PATH = 17 FEET PER SECOND
   NUMBER OF CONFLICTS =  2 AND TURN CODE FOR PATH IS RIGHT
   CONFLICT ENTRY NUMBERS ORDERED BY DISTANCE DOWN THIS PATH ARE
         26   27
```

```
PATH   6 GOES FROM LANE 1 OF APPROACH  2 TO LANE 1 OF APPROACH  7
   LENGTH OF PATH =  72 FEET AND SPEED OF PATH = 20 FEET PER SECOND
   NUMBER OF CONFLICTS =  9 AND TURN CODE FOR PATH IS LEFT
   CONFLICT ENTRY NUMBERS ORDERED BY DISTANCE DOWN THIS PATH ARE
          2   29    8   31   30   32   28   16   33

PATH   7 GOES FROM LANE 1 OF APPROACH  2 TO LANE 1 OF APPROACH  8
   LENGTH OF PATH =  76 FEET AND SPEED OF PATH = 44 FEET PER SECOND
   NUMBER OF CONFLICTS =  9 AND TURN CODE FOR PATH IS STRAIGHT
   CONFLICT ENTRY NUMBERS ORDERED BY DISTANCE DOWN THIS PATH ARE
         35    3   38    9   17   36   23   34   37

PATH   8 GOES FROM LANE 1 OF APPROACH  3 TO LANE 1 OF APPROACH  8
   LENGTH OF PATH =  60 FEET AND SPEED OF PATH = 20 FEET PER SECOND
   NUMBER OF CONFLICTS =  7 AND TURN CODE FOR PATH IS LEFT
   CONFLICT ENTRY NUMBERS ORDERED BY DISTANCE DOWN THIS PATH ARE
         28   41   18   40   24   34   39

PATH   9 GOES FROM LANE 2 OF APPROACH  3 TO LANE 1 OF APPROACH  5
   LENGTH OF PATH =  64 FEET AND SPEED OF PATH = 44 FEET PER SECOND
   NUMBER OF CONFLICTS =  7 AND TURN CODE FOR PATH IS STRAIGHT
   CONFLICT ENTRY NUMBERS ORDERED BY DISTANCE DOWN THIS PATH ARE
         43   10   29    4   35   26   42

PATH  10 GOES FROM LANE 2 OF APPROACH  3 TO LANE 1 OF APPROACH  6
   LENGTH OF PATH =  41 FEET AND SPEED OF PATH = 17 FEET PER SECOND
   NUMBER OF CONFLICTS =  3 AND TURN CODE FOR PATH IS RIGHT
   CONFLICT ENTRY NUMBERS ORDERED BY DISTANCE DOWN THIS PATH ARE
          5   11   44

PATH  11 GOES FROM LANE 2 OF APPROACH  3 TO LANE 1 OF APPROACH  8
   LENGTH OF PATH =  72 FEET AND SPEED OF PATH = 20 FEET PER SECOND
   NUMBER OF CONFLICTS =  9 AND TURN CODE FOR PATH IS LEFT
   CONFLICT ENTRY NUMBERS ORDERED BY DISTANCE DOWN THIS PATH ARE
         46   30   12   45   19   36   25   37   39
```

```
PATH  12 GOES FROM LANE 1 OF APPROACH  4 TO LANE 1 OF APPROACH  5
   LENGTH OF PATH =  72 FEET AND SPEED OF PATH = 20 FEET PER SECOND
   NUMBER OF CONFLICTS =  9 AND TURN CODE FOR PATH IS LEFT
   CONFLICT ENTRY NUMBERS ORDERED BY DISTANCE DOWN THIS PATH ARE
         20   40   45   31   13   38    6   27   42

PATH  13 GOES FROM LANE 1 OF APPROACH  4 TO LANE 1 OF APPROACH  6
   LENGTH OF PATH =  76 FEET AND SPEED OF PATH = 44 FEET PER SECOND
   NUMBER OF CONFLICTS =  9 AND TURN CODE FOR PATH IS STRAIGHT
   CONFLICT ENTRY NUMBERS ORDERED BY DISTANCE DOWN THIS PATH ARE
         21   41   32   46   43   14    7   15   44

PATH  14 GOES FROM LANE 1 OF APPROACH  4 TO LANE 1 OF APPROACH  7
   LENGTH OF PATH =  41 FEET AND SPEED OF PATH = 17 FEET PER SECOND
   NUMBER OF CONFLICTS =  2 AND TURN CODE FOR PATH IS RIGHT
   CONFLICT ENTRY NUMBERS ORDERED BY DISTANCE DOWN THIS PATH ARE
         22   33
```

DRIVER-VEHICLE PROCESSOR FOR THE TEXAS TRAFFIC SIMULATION PACKAGE (V3.00)
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2X2 INTERSECTION

TABLE 1 - LISTING OF INBOUND APPROACH NUMBERS

1
2
3
4

TOTAL NUMBER OF INBOUND APPROACHES = 4

TABLE 2 - LISTING OF OUTBOUND APPROACH NUMBERS

5
6
7
8

TOTAL NUMBER OF OUTBOUND APPROACHES = 4

TOTAL NUMBER OF INBOUND AND OUTBOUND APPROACHES = 8

TABLE 3 - DRIVER-VEHICLE PROCESSOR OPTIONS

TIME FOR GENERATING VEHICLES (MIN) ----	20
MINIMUM HEADWAY FOR VEHICLES (SEC) ----	1.0
NUMBER OF VEHICLE CLASSES -------------	12
NUMBER OF DRIVER CLASSES --------------	3
PERCENT OF LEFT TURNS IN MEDIAN LANE --	80.
PERCENT OF RIGHT TURNS IN CURB LANE ---	80.

TABLE 4 - LISTING OF APPROACHES

```
    APPROACH NUMBER -----------------------    1
    APPROACH AZIMUTH ----------------------  180
    NUMBER OF LANES -----------------------    2
    NUMBER OF DEGREES FOR STRAIGHT --------   20

    HEADWAY DISTRIBUTION NAME ------------- SNEGEXP  PARAMETER =    1.00
    EQUIVALENT HOURLY VOLUME (VPH) --------  150
    APPROACH MEAN SPEED (MPH) -------------   29.0
    APPROACH 85 PERCENTILE SPEED (MPH) ----   31.0
        OUTBOUND APPROACH NUMBER ----------    5    6    7    8
    PERCENT GOING TO OUTBOUND APPROACHES --    0.  10.  60.  30.
    USER SUPPLIED PERCENT OF VEHICLES -----   NO
        VEHICLE CLASS NUMBER --------------    1    2    3    4     5     6
    PROGRAM SUPPLIED PERCENT OF VEHICLES --  1.5 22.5 23.3 44.7   2.6   2.6
                                               7    8    9   10    11    12
                                             0.2  0.2  0.2  0.2   1.0   1.0
    SEED FOR RANDOM NUMBERS---------------- 22889
    PERCENT OF TRAFFIC ENTERING ON LANE 2 -  100.  (CURB LANE)

    APPROACH NUMBER -----------------------    8
    APPROACH AZIMUTH ----------------------  270
    NUMBER OF LANES -----------------------    1
    APPROACH NUMBER -----------------------    2
    APPROACH AZIMUTH ----------------------  270
    NUMBER OF LANES -----------------------    1
    NUMBER OF DEGREES FOR STRAIGHT --------   20
    HEADWAY DISTRIBUTION NAME ------------- SNEGEXP  PARAMETER =    1.00
    EQUIVALENT HOURLY VOLUME (VPH) --------  700
    APPROACH MEAN SPEED (MPH) -------------   29.0
    APPROACH 85 PERCENTILE SPEED (MPH) ----   31.0
        OUTBOUND APPROACH NUMBER ----------    5    6    7    8
    PERCENT GOING TO OUTBOUND APPROACHES --   10.   0.  10.  80.
    USER SUPPLIED PERCENT OF VEHICLES -----   NO
```

```
    VEHICLE CLASS NUMBER --------------    1    2    3    4    5    6
PROGRAM SUPPLIED PERCENT OF VEHICLES --   1.5 22.5 23.3 44.7  2.6  2.6
                                           7    8    9   10   11   12
                                          0.2  0.2  0.2  0.2  1.0  1.0
SEED FOR RANDOM NUMBERS---------------- 8227
PERCENT OF TRAFFIC ENTERING ON LANE 1 - 100.  (MEDIAN LANE)

APPROACH NUMBER ----------------------    5
APPROACH AZIMUTH ---------------------    0
NUMBER OF LANES ----------------------    1

APPROACH NUMBER ----------------------    3
APPROACH AZIMUTH ---------------------    0
NUMBER OF LANES ----------------------    2
NUMBER OF DEGREES FOR STRAIGHT --------  20
HEADWAY DISTRIBUTION NAME ------------- SNEGEXP  PARAMETER =    1.00
EQUIVALENT HOURLY VOLUME (VPH) -------- 150
APPROACH MEAN SPEED (MPH) ------------- 29.0
APPROACH 85 PERCENTILE SPEED (MPH) ---- 31.0
    OUTBOUND APPROACH NUMBER ----------   5    6    7    8
PERCENT GOING TO OUTBOUND APPROACHES -- 60.  30.   0.  10.
USER SUPPLIED PERCENT OF VEHICLES -----  NO
    VEHICLE CLASS NUMBER --------------    1    2    3    4    5    6
PROGRAM SUPPLIED PERCENT OF VEHICLES --   1.5 22.5 23.3 44.7  2.6  2.6
                                           7    8    9   10   11   12
                                          0.2  0.2  0.2  0.2  1.0  1.0
SEED FOR RANDOM NUMBERS---------------- 71623
PERCENT OF TRAFFIC ENTERING ON LANE 2 - 100.  (CURB LANE)
```

```
    APPROACH NUMBER -----------------------    6
    APPROACH AZIMUTH ----------------------   90
    NUMBER OF LANES -----------------------    1

    APPROACH NUMBER -----------------------    4
    APPROACH AZIMUTH ----------------------   90
    NUMBER OF LANES -----------------------    1
    NUMBER OF DEGREES FOR STRAIGHT --------   20
    HEADWAY DISTRIBUTION NAME ------------- SNEGEXP  PARAMETER =    1.00
    EQUIVALENT HOURLY VOLUME (VPH) --------  700
    APPROACH MEAN SPEED (MPH) -------------   29.0
    APPROACH 85 PERCENTILE SPEED (MPH) ----   31.0
        OUTBOUND APPROACH NUMBER ----------    5     6     7     8
    PERCENT GOING TO OUTBOUND APPROACHES --   10.   80.   10.    0.
    USER SUPPLIED PERCENT OF VEHICLES -----   NO
        VEHICLE CLASS NUMBER --------------    1     2     3     4     5     6
    PROGRAM SUPPLIED PERCENT OF VEHICLES --   1.5  22.5  23.3  44.7   2.6   2.6
                                               7     8     9    10    11    12
                                              0.2   0.2   0.2   0.2   1.0   1.0
    SEED FOR RANDOM NUMBERS--------------- 48932
    PERCENT OF TRAFFIC ENTERING ON LANE 1 -  100.  (MEDIAN LANE)

    APPROACH NUMBER -----------------------    7
    APPROACH AZIMUTH ----------------------  180
    NUMBER OF LANES -----------------------    1

    TOTAL NUMBER OF APPROACHES =  8

TABLE  5  -  DRIVER AND VEHICLE CLASS CHARACTERISTICS

    USER SUPPLIED DRIVER CLASS SPLIT ------   NO
    USER SUPPLIED VEHICLE CHARACTERISTICS -   NO
    USER SUPPLIED DRIVER CHARACTERISTICS --   NO
    DRIVER LOGOUT SUMMARY REQUESTED -------   NO
```

DRIVER CLASS SPLIT (PROGRAM SUPPLIED VALUES)

DRIVER CLASS NUMBER	1	2	3
VEHICLE CLASS NUMBER 1	50.0	40.0	10.0
VEHICLE CLASS NUMBER 2	30.0	40.0	30.0
VEHICLE CLASS NUMBER 3	35.0	35.0	30.0
VEHICLE CLASS NUMBER 4	25.0	45.0	30.0
VEHICLE CLASS NUMBER 5	40.0	30.0	30.0
VEHICLE CLASS NUMBER 6	40.0	30.0	30.0
VEHICLE CLASS NUMBER 7	40.0	30.0	30.0
VEHICLE CLASS NUMBER 8	40.0	30.0	30.0
VEHICLE CLASS NUMBER 9	50.0	40.0	10.0
VEHICLE CLASS NUMBER 10	50.0	40.0	10.0
VEHICLE CLASS NUMBER 11	50.0	40.0	10.0
VEHICLE CLASS NUMBER 12	50.0	40.0	10.0

VEHICLE CHARACTERISTICS (PROGRAM SUPPLIED VALUES)

VEHICLE CLASS NUMBER	1	2	3	4	5	6	7	8	9	10	11	12
LENGTH OF VEHICLES (FT)	14	15	16	18	32	32	32	32	60	60	60	60
VEHICLE OPERATIONAL FACTOR	115	90	100	110	85	80	80	75	70	65	75	70
MAXIMUM DECELERATION (FT/SEC/SEC)	10	9	9	8	7	5	7	5	6	4	6	4
MAXIMUM ACCELERATION (FT/SEC/SEC)	14	8	9	11	7	6	6	5	4	3	5	4
MAXIMUM VELOCITY (FT/SEC)	205	120	135	150	100	85	100	85	95	75	100	80
MINIMUM TURNING RADIUS (FT)	20	20	22	24	42	42	42	42	45	45	45	45

DRIVER CHARACTERISTICS (PROGRAM SUPPLIED VALUES)

DRIVER CLASS NUMBER ---------------	1	2	3
DRIVER OPERATIONAL FACTOR ---------	110	100	85
DRIVER REACTION TIME (SEC) --------	0.5	1.0	1.5

TABLE 6 - GENERATION OF APPROACH HEADWAYS

APPROACH NUMBER	DISTRIBUTION NAME	NUMBER GENERATED	VOLUME GENERATED	INPUT VOLUME	PERCENT DIFFERENCE
1	SNEGEXP	46	138	150	-8.00
2	SNEGEXP	248	744	700	6.29
3	SNEGEXP	66	198	150	32.00
4	SNEGEXP	227	681	700	-2.71
TOTAL		587	1761	1700	3.59

TABLE 7 - FINAL APPROACH VOLUMES

	SPECIAL VEHICLES		GENERATED VEHICLES		TOTAL VEHICLES		
APPR NO	NUMBER FOR SIMULATION	VOLUME FOR SIMULATION	NUMBER FOR SIMULATION	VOLUME FOR SIMULATION	NUMBER FOR SIMULATION	VOLUME FOR SIMULATION	INPUT VOL
1	0	0	46	138	46	138	150
2	0	0	248	744	248	744	700
3	0	0	66	198	66	198	150
4	0	0	227	681	227	681	700
TOTAL	0	0	587	1761	587	1761	1700

THE INTERSECTION HAS A JAM DENSITY OF 227 VEHICLES PER MILE

TABLE 8 - STATISTICS OF GENERATION

APPROACH STATISTICS

APPROACH NUMBER	1					
OUTBOUND APPROACH NUMBER	5	6	7	8		
PERCENT GOING TO OUTBOUND APPROACHES	0.0	10.9	58.7	30.4		
VEHICLE CLASS NUMBER	1	2	3	4	5	6
GENERATION PERCENT OF VEHICLES	2.2	21.7	21.7	45.7	2.2	2.2
	7	8	9	10	11	12
	0.0	0.0	0.0	0.0	2.2	2.2
PERCENT OF TRAFFIC ENTERING ON LANE 1	0.0	(MEDIAN LANE)				
PERCENT OF TRAFFIC ENTERING ON LANE 2	100.0	(CURB LANE)				

APPROACH NUMBER	2					
OUTBOUND APPROACH NUMBER	5	6	7	8		
PERCENT GOING TO OUTBOUND APPROACHES	10.1	0.0	10.1	79.8		
VEHICLE CLASS NUMBER	1	2	3	4	5	6
GENERATION PERCENT OF VEHICLES	1.6	22.2	23.0	44.0	2.4	2.8
	7	8	9	11	11	12
	0.4	0.4	0.4	0.4	1.2	1.2
PERCENT OF TRAFFIC ENTERING ON LANE 1	100.0	(MEDIAN LANE)				

APPROACH NUMBER	3					
OUTBOUND APPROACH NUMBER	5	6	7	8		
PERCENT GOING TO OUTBOUND APPROACHES	60.6	30.3	0.0	9.1		
VEHICLE CLASS NUMBER	1	2	3	4	5	6
GENERATION PERCENT OF VEHICLES	1.5	21.2	22.7	42.4	3.0	3.0
	7	8	9	10	11	12
	0.0	0.0	1.5	1.5	1.5	1.5
PERCENT OF TRAFFIC ENTERING ON LANE 1	0.0	(MEDIAN LANE)				
PERCENT OF TRAFFIC ENTERING ON LANE 2	100.0	(CURB LANE)				

APPROACH NUMBER ----------------------	4					
OUTBOUND APPROACH NUMBER ----------	5	6	7	8		
PERCENT GOING TO OUTBOUND APPROACHES --	9.7	80.2	10.1	0.0		
VEHICLE CLASS NUMBER --------------	1	2	3	4	5	6
GENERATION PERCENT OF VEHICLES --------	1.8	22.5	22.5	43.6	2.6	2.6
	7	8	9	10	11	12
	0.4	0.4	0.4	0.4	1.3	1.3

PERCENT OF TRAFFIC ENTERING ON LANE 1 - 100.0 (MEDIAN LANE)

DRIVER CLASS SPLIT STATISTICS

DRIVER CLASS NUMBER ------------------	1	2	3
VEHICLE CLASS NUMBER 1 (10 VEH) -----	60.0	30.0	10.0
VEHICLE CLASS NUMBER 2 (130 VEH) -----	30.0	39.2	30.8
VEHICLE CLASS NUMBER 3 (133 VEH) -----	34.6	33.8	31.6
VEHICLE CLASS NUMBER 4 (257 VEH) -----	24.9	45.1	30.0
VEHICLE CLASS NUMBER 5 (15 VEH) -----	40.0	26.7	33.3
VEHICLE CLASS NUMBER 6 (16 VEH) -----	37.5	31.2	31.2
VEHICLE CLASS NUMBER 7 (2 VEH) -----	50.0	0.0	50.0
VEHICLE CLASS NUMBER 8 (2 VEH) -----	0.0	50.0	50.0
VEHICLE CLASS NUMBER 9 (3 VEH) -----	66.7	33.3	0.0
VEHICLE CLASS NUMBER 10 (3 VEH) -----	33.3	33.3	33.3
VEHICLE CLASS NUMBER 11 (8 VEH) -----	62.5	37.5	0.0
VEHICLE CLASS NUMBER 12 (8 VEH) -----	62.5	37.5	0.0

```
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GEOMETRY PROCESSOR FILE NAME AND TITLE:
C:\TEXAS\USER_DAT\GP0007
 2X2 INTERSECTION

DRIVER-VEHICLE PROCESSOR FILE NAME AND TITLE:
C:\TEXAS\USER_DAT\DV0007
 2X2 INTERSECTION

SIMULATION PROCESSOR FILE NAME AND TITLE:
SIM
 2X2 INTERSECTION

START-UP TIME (MINUTES) -------------------------- =       5.00
SIMULATION TIME (MINUTES) ------------------------ =      15.00
STEP INCREMENT FOR SIMULATION TIME (SECONDS) ------ =       0.50

SPEED FOR DELAY BELOW XX MPH (MPH) --------------- =      10.00

MAXIMUM CLEAR DISTANCE FOR BEING IN A QUEUE (FT) -- =      30.00

CAR FOLLOWING EQUATION LAMBDA -------------------- =       2.800
CAR FOLLOWING EQUATION MU ------------------------ =       0.800
CAR FOLLOWING EQUATION ALPHA --------------------- =    4000.000

SUMMARY STATISTICS PRINTED BY TURNING MOVEMENTS --- =     YES
SUMMARY STATISTICS PRINTED BY INBOUND APPROACH ---- =     YES

PUNCHED OUTPUT OF STATISTICS --------------------- =     NO
WRITE TAPE FOR POLLUTION DISPERSION MODEL --------- =     YES

LEAD TIME GAP FOR CONFLICT CHECKING (SECONDS) ----- =       1.30
LAG TIME GAP FOR CONFLICT CHECKING (SECONDS) ------ =       0.50
```

INTERSECTION TRAFFIC CONTROL ---------------------- = 3 (LESS-THAN-ALL-WAY STOP SIGN)

MAXIMUM NUMBER OF VEHICLES IN SYSTEM IS 500
HESITATION FACTOR ADDED TO PIJR IS 1.0 SECONDS

LANE CONTROL FOR THE 10 LANES = 4 4 1 2 1 4 4 1 2 1

WHERE 1 = OUTBOUND (OR BLOCKED INBOUND) LANE
2 = UNCONTROLLED
3 = YIELD SIGN
4 = STOP SIGN
5 = SIGNAL
6 = SIGNAL WITH LEFT TURN ON RED
7 = SIGNAL WITH RIGHT TURN ON RED

SUMMARY STATISTICS FOR INBOUND APPROACH 1	LEFT	STRAIGHT	RIGHT	APPRCH 1
TOTAL DELAY (VEHICLE-SECONDS) -------------------- =	258.7	923.2	386.0	1567.8
NUMBER OF VEHICLES INCURRING TOTAL DELAY --------- =	4	18	10	32
PERCENT OF VEHICLES INCURRING TOTAL DELAY -------- =	100.0	100.0	100.0	100.0
AVERAGE TOTAL DELAY (SECONDS) -------------------- =	64.7	51.3	38.6	49.0
AVERAGE TOTAL DELAY/AVERAGE TRAVEL TIME ---------- =	70.3 %	66.0 %	58.6 %	64.7 %
QUEUE DELAY (VEHICLE-SECONDS) -------------------- =	229.5	811.5	320.0	1361.0
NUMBER OF VEHICLES INCURRING QUEUE DELAY --------- =	4	18	10	32
PERCENT OF VEHICLES INCURRING QUEUE DELAY -------- =	100.0	100.0	100.0	100.0
AVERAGE QUEUE DELAY (SECONDS) -------------------- =	57.4	45.1	32.0	42.5
AVERAGE QUEUE DELAY/AVERAGE TRAVEL TIME ---------- =	62.3 %	58.0 %	48.6 %	56.1 %
STOPPED DELAY (VEHICLE-SECONDS) ------------------ =	191.0	731.5	289.0	1211.5
NUMBER OF VEHICLES INCURRING STOPPED DELAY ------- =	4	18	10	32
PERCENT OF VEHICLES INCURRING STOPPED DELAY ------ =	100.0	100.0	100.0	100.0
AVERAGE STOPPED DELAY (SECONDS) ------------------ =	47.7	40.6	28.9	37.9
AVERAGE STOPPED DELAY/AVERAGE TRAVEL TIME -------- =	51.9 %	52.3 %	43.9 %	50.0 %

DELAY BELOW 10.0 MPH (VEHICLE-SECONDS) ------------ =	245.0	903.0	373.5	1521.5
NUMBER OF VEHICLES INCURRING DELAY BELOW 10.0 MPH - =	4	18	10	32
PERCENT OF VEHICLES INCURRING DELAY BELOW 10.0 MPH =	100.0	100.0	100.0	100.0
AVERAGE DELAY BELOW 10.0 MPH (SECONDS) ------------ =	61.2	50.2	37.3	47.5
AVERAGE DELAY BELOW 10.0 MPH/AVERAGE TRAVEL TIME -- =	66.6%	64.6%	56.7%	62.7%
VEHICLE-MILES OF TRAVEL --------------------------- =	0.850	3.799	2.067	6.715
AVERAGE VEHICLE-MILES OF TRAVEL ------------------- =	0.213	0.211	0.207	0.210
TRAVEL TIME (VEHICLE-SECONDS) --------------------- =	368.1	1398.0	658.8	2424.9
AVERAGE TRAVEL TIME (SECONDS) --------------------- =	92.0	77.7	65.9	75.8
NUMBER OF VEHICLES PROCESSED ---------------------- =	4	18	10	32
VOLUME PROCESSED (VEHICLES/HOUR) ------------------ =	16.0	72.0	40.0	128.0
TIME MEAN SPEED (MPH) = MEAN OF ALL VEHICLE SPEEDS =	9.8	12.7	14.2	12.8
SPACE MEAN SPEED (MPH) = TOT DIST / TOT TRAVEL TIME =	8.3	9.8	11.3	10.0
AVERAGE DESIRED SPEED (MPH) ----------------------- =	28.0	28.9	27.3	28.3
AVERAGE MAXIMUM ACCELERATION (FT/SEC/SEC) --------- =	3.8	4.9	3.7	4.4
AVERAGE MAXIMUM DECELERATION (FT/SEC/SEC) --------- =	5.1	6.0	6.2	6.0
OVERALL AVERAGE TOTAL DELAY (SECONDS) ------------- =	64.7	51.3	38.6	49.0
OVERALL AVERAGE QUEUE DELAY (SECONDS) ------------- =	57.4	45.1	32.0	42.5
OVERALL AVERAGE STOPPED DELAY (SECONDS) ----------- =	47.7	40.6	28.9	37.9
OVERALL AVERAGE DELAY BELOW 10.0 MPH (SECONDS) ---- =	61.2	50.2	37.3	47.5
PERCENT OF VEHICLES MAKING A LEFT TURN ------------ =	12.5			12.5
PERCENT OF VEHICLES GOING STRAIGHT ---------------- =		56.2		56.2
PERCENT OF VEHICLES MAKING A RIGHT TURN ----------- =			31.2	31.2
MAXIMUM AND AVERAGE QUEUE LENGTH FOR LANE 1 ------- =				0 0.0
MAXIMUM AND AVERAGE QUEUE LENGTH FOR LANE 2 ------- =				7 1.5
NUMBER OF CLEAR ZONE INTRUSIONS ------------------- =				1
AVERAGE OF LOGIN SPEED/DESIRED SPEED (PERCENT) ---- =				100.0

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2X2 INTERSECTION

SUMMARY STATISTICS FOR ALL APPROACHES	APPR 1	APPR 2	APPR 3	APPR 4	ALL
TOTAL DELAY (VEHICLE-SECONDS) ------------------ =	1567.8	974.8	4176.1	492.8	7211.6
NUMBER OF VEHICLES INCURRING TOTAL DELAY -------- =	32	178	50	164	424
PERCENT OF VEHICLES INCURRING TOTAL DELAY ------- =	100.0	94.2	100.0	97.0	96.4
AVERAGE TOTAL DELAY (SECONDS) ------------------ =	49.0	5.5	83.5	3.0	17.0
AVERAGE TOTAL DELAY/AVERAGE TRAVEL TIME --------- =	64.7%	17.2%	75.8%	10.1%	39.5%
QUEUE DELAY (VEHICLE-SECONDS) ------------------ =	1361.0	125.0	3808.5	57.5	5352.0
NUMBER OF VEHICLES INCURRING QUEUE DELAY -------- =	32	18	50	9	109
PERCENT OF VEHICLES INCURRING QUEUE DELAY ------- =	100.0	9.5	100.0	5.3	24.8
AVERAGE QUEUE DELAY (SECONDS) ------------------ =	42.5	6.9	76.2	6.4	49.1
AVERAGE QUEUE DELAY/AVERAGE TRAVEL TIME --------- =	56.1%	21.8%	69.1%	21.5%	113.9%
STOPPED DELAY (VEHICLE-SECONDS) ---------------- =	1211.5	69.0	3080.0	43.5	4404.0
NUMBER OF VEHICLES INCURRING STOPPED DELAY ------ =	32	18	50	9	109
PERCENT OF VEHICLES INCURRING STOPPED DELAY ----- =	100.0	9.5	100.0	5.3	24.8
AVERAGE STOPPED DELAY (SECONDS) ---------------- =	37.9	3.8	61.6	4.8	40.4
AVERAGE STOPPED DELAY/AVERAGE TRAVEL TIME ------- =	50.0%	12.1%	55.9%	16.3%	93.7%
DELAY BELOW 10.0 MPH (VEHICLE-SECONDS) ---------- =	1521.5	372.0	4194.0	92.0	6179.5
NUMBER OF VEHICLES INCURRING DELAY BELOW 10.0 MPH =	32	53	50	15	150
PERCENT OF VEHICLE INCURRING DELAY BELOW 10.0 MPH =	100.0	28.0	100.0	8.9	34.1
AVERAGE DELAY BELOW 10.0 MPH (SECONDS) ---------- =	47.5	7.0	83.9	6.1	41.2

VEHICLE-MILES OF TRAVEL ------------------------- =	6.71	40.12	10.49	35.92	93.30
AVERAGE VEHICLE-MILES OF TRAVEL ---------------- =	0.21	0.21	0.21	0.21	0.21
TRAVEL TIME (VEHICLE-SECONDS) ------------------ =	2424	6010	5511	5017	18964
AVERAGE TRAVEL TIME (SECONDS) ------------------ =	75.8	31.8	110.2	29.7	43.1
NUMBER OF VEHICLES PROCESSED ------------------- =	32	189	50	169	440
VOLUME PROCESSED (VEHICLES/HOUR) --------------- =	128	756	200	676	1760
TIME MEAN SPEED (MPH) = MEAN OF ALL VEH SPEEDS =	12.8	24.9	10.0	26.1	22.8
SPACE MEAN SPEED (MPH) = TOT DIS/TOT TRAVEL TIME =	10.0	24.1	6.9	25.8	17.7
AVERAGE DESIRED SPEED (MPH) -------------------- =	28.3	28.8	28.3	28.6	28.6
AVERAGE MAXIMUM ACCELERATION (FT/SEC/SEC) ------ =	4.4	1.9	4.3	1.7	2.3
AVERAGE MAXIMUM DECELERATION (FT/SEC/SEC) ------ =	6.0	1.6	5.0	1.4	2.2
OVERALL AVERAGE TOTAL DELAY (SECONDS) ---------- =	49.0	5.2	83.5	2.9	16.4
OVERALL AVERAGE QUEUE DELAY (SECONDS) ---------- =	42.5	0.7	76.2	0.3	12.2
OVERALL AVERAGE STOPPED DELAY (SECONDS) -------- =	37.9	0.4	61.6	0.3	10.0
OVERALL AVERAGE DELAY BELOW 10.0 MPH (SECONDS) - =	47.5	2.0	83.9	0.5	14.0
PERCENT OF VEHICLES MAKING A LEFT TURN --------- =	12.5	10.1	12.0	10.1	
PERCENT OF VEHICLES GOING STRAIGHT ------------- =	56.2	79.9	54.0	79.3	
PERCENT OF VEHICLES MAKING A RIGHT TURN -------- =	31.2	10.1	34.0	10.7	
MAXIMUM AND AVERAGE QUEUE LENGTH FOR LANE 1 ---- =	0 .0	5 .1	0 .0	3 .1	
MAXIMUM AND AVERAGE QUEUE LENGTH FOR LANE 2 ---- =	7 1.5		11 4.2		
NUMBER OF CLEAR ZONE INTRUSIONS ---------------- =	1	2	3	2	8
NUMBER OF VEHICLES ELIMINATED (LANE FULL) ------ =		2			2
AVERAGE OF LOGIN SPEED/DESIRED SPEED (PERCENT) - =	100.0	98.4	100.0	99.1	99.0

START-UP TIME = 300.000 SECONDS   NUMBER OF VEHICLES PROCESSED = 125

SIMULATION TIME = 900.000 SECONDS   NUMBER OF VEHICLES PROCESSED = 440

NUMBER OF VEHICLES IN THE SYSTEM AT SUMMARY = 20
AVERAGE NUMBER OF VEHICLES IN THE SYSTEM -- = 21.2 MAX = 34

APPENDIX B

STATISTICAL TABLES FOR TEXAS ANALYSES

TABLE B-1. ANOVA FOR SIMULATION TIME

Significance Test for Delay Using UNIQUE Sums of Squares

Source of Variation	SS	DF	MS	F	Sig of F
Within Cells	2078.06	166	12.52		
Constant	55.96	8	7.00	.5588	.8104
Total	2134.02	174			

TABLE B-2. ANOVA FOR HEADWAY DISTRIBUTION PARAMETER

Significance Test for Delay Using UNIQUE Sums of Squares

Source of Variation	SS	DF	MS	F	Sig of F
Within Cells	77.15	54	1.43		
Constant	2809.87	1	2809.87	1966.78	.000
Volume	239.38	2	119.69	83.78	.000
Headway Parameter	.11	1	.11	.08	.780
Volume by Headway Parameter	.11	2	.05	.04	.964

TABLE B-3. ANOVA FOR TRAFFIC COMPOSITION

Significance Test for Delay Using UNIQUE Sums of Squares

Source of Variation	SS	DF	MS	F	Sig of F
Main Effects	167.720	2	83.860	59.197	.000
Volume	167.690	1	167.690	112.374	.000
% Trucks	.030	1	.030	.020	.888
Two-Way Interactions	.272	1	.272	.182	.672
Explained	167.993	3	55.998	37.526	.000
Residual	53.721	36	1.492		
Total	221.714	39	5.685		

TABLE B-4. ANOVA FOR MAJOR STREET SPEED LIMIT

Significance Test for Delay Using UNIQUE Sums of Squares

Source of Variation	SS	DF	MS	F	Sig of F
Main Effects Speed Limit	113.993	2	56.996	1.186	.321
Explained	113.993	2	56.996	1.186	.321
Residual	1297.922	27	48.071		
Total	1411.915	29	48.687		

TABLE B-5. ANOVA FOR MINOR STREET LEFT TURN PERCENTAGE

Significance Test for Delay Using UNIQUE Sums of Squares

Source of Variation	SS	DF	MS	F	Sig of F
Main Effects LT Percent	75.853	2	37.926	.862	.434
Explained	75.853	2	37.926	.862	.434
Residual	1187.602	27	43.985		
Total	1263.455	29	43.567		

TABLE B-6. ANOVA FOR EXCLUSIVE RIGHT TURN LANE

Significance Test for Delay Using UNIQUE Sums of Squares

Source of Variation	SS	DF	MS	F	Sig of F
Main Effects RT Lane	94.735	3	31.578	1.740	.176
Explained	94.735	3	31.578	1.740	.176
Residual	653.463	36	18.152		
Total	748.198	29	43.567		

TABLE B-7. ANOVA FOR RIGHT TURN CHANNELIZATION

Significance Test for Delay Using UNIQUE Sums of Squares

Source of Variation	SS	DF	MS	F	Sig of F
Main Effects of Channelization	27.612	1	27.612	10.900	.004
Explained	27.612	1	27.612	10.900	.004
Residual	46.597	18	2.533		
Total	73.209	19	3.853		

TABLE B-8. ANOVA FOR LANE CONFIGURATION

Significance Test for Delay Using UNIQUE Sums of Squares

Source of Variation	SS	DF	MS	F	Sig of F
Main Effects	2384.427	3	794.809	20.045	.000
Configuration	2322.613	2	1161.307	29.288	.000
RT Percentage	61.814	1	61.814	1.559	.217
Two-Way Interactions	21.567	2	10.784	.272	.763
Explained	2405.994	5	481.199	12.136	.000
Residual	2141.179	54	39.651		
Total	4547.173	59	77.071		

TABLE B-9. ANOVA RIGHT TURN FACTOR: LOW VOLUME LEVEL

Significance Test for Delay Using UNIQUE Sums of Squares

Source of Variation	SS	DF	MS	F	Sig of F
Within Cells	410.42	285	1.44		
Constant	8849.81	1	8849.81	6145.43	
Volume	55.75	4	13.94	9.68	.000
Percent RT	3.25	2	1.63	1.13	.325
Volume by Percent RT	2.35	8	.29	.20	.990

TABLE B-10. ANOVA RIGHT TURN FACTOR: HIGH VOLUME LEVEL

Significance Test for Delay Using UNIQUE Sums of Squares

Source of Variation	SS	DF	MS	F	Sig of F
Within Cells	27018.58	28	94.80		
Constant	112876.96	1	112876.96	1190.66	
Volume	2876.97	4	719.24	7.59	.000
Percent RT	397.82	2	198.91	2.10	.125
Volume by Percent RT	192.93	8	24.12	.25	.979

TABLE B-11. MULTIVARIATE ANOVA FOR RIGHT TURN FACTOR

Significance Test for Average Queue Delay

Source of Variation	SS	DF	MS	F	Sig of F
Model	556524.002	3	285508.001	20.665	.0001
Error	107724.315	12	8977.026		
Cor Total	664248.318	15			

Variable	Estimate	DF	SS	T	Sif of T
Major St Volume	117.350	1	220336.360	4.954	.0003
Minor St Volume	126.488	1	255985.402	5.340	.0002
Two-Way Interaction	70.800	1	80202.240	2.989	.0113

Significance Test for Average Stopped Delay

Source of Variation	SS	DF	MS	F	Sig of F
Model	481220.342	3	160406.781	16.975	.0001
Error	113396.907	12	9449.742		
Cor Total	594617.249	15			

Variable	Estimate	DF	SS	T	Sif of T
Major St Volume	119.231	1	227457.456	4.906	.0004
Minor St Volume	101.044	1	163357.431	4.158	.0013
Two-Way Interaction	75.169	1	90405.456	3.093	.0093

APPENDIX C

DATABASE USED IN FIELD DATA ANALYSIS

Int No	Obs No	No Lanes Main St	No Lanes Side St	Main St Speed	Main St Volume	Side St Volume	Avg Delay	% Rt Turns
411	4	3	1	3	3017	77	15	94
521	1	3	1	2	1890	132	13	49
521	2	3	1	2	2044	149	17	58
530	1	2	1	2	693	221	15	19
530	2	2	1	2	720	144	21	17
542	1	3	1	2	1630	89	14	75
542	2	3	1	2	1823	132	18	64
542	3	3	1	2	2020	132	18	64
558	1	1	1	2	2427	414	22	100
558	2	1	1	2	1416	360	17	100
653	1	1	1	2	734	308	11	93
653	2	1	1	2	791	430	27	90
653	3	1	1	2	751	414	25	90
653	4	1	1	2	549	301	7	93
660	1	1	1	2	1065	204	22	76
660	2	1	1	2	1057	346	54	69
660	3	1	1	2	1070	338	35	70
660	4	1	1	2	1018	427	48	55
609	0	3	1	1	2244	93	27	0
650	0	1	1	2	900	285	19	51
651	0	1	1	2	1083	117	28	8
653	0	1	1	2	849	417	32	91
110	1	4	2	2	3488	19	48	37
110	2	4	2	2	4102	26	55	54
110	3	4	2	2	4499	290	44	13
110	4	4	2	2	5182	57	65	9
114	1	1	2	2	536	297	11	24
114	2	1	2	2	552	516	16	32
114	3	1	2	2	537	299	13	37
114	4	1	2	2	392	194	25	34
133	1	2	2	2	536	434	1	3
133	2	2	2	2	527	466	1	3
133	3	2	2	2	981	665	6	9
133	4	2	2	2	1115	768	6	5
159	1	2	2	2	1106	55	10	65
159	2	2	2	2	1123	94	11	69
159	3	2	2	2	1377	413	11	87
159	4	2	2	2	1461	60	15	68
163	1	2	2	3	1339	79	22	57
163	2	2	2	3	1520	1087	20	23
163	3	2	2	3	1649	108	16	40
163	4	2	2	3	1937	4	14	0
204	1	2	2	2	840	407	8	92
204	2	2	2	2	945	598	42	88
204	3	2	2	2	894	657	37	86
204	4	2	2	2	915	355	13	89
220	1	2	2	2	898	279	4	21
220	2	2	2	2	919	324	4	30
220	3	2	2	2	1013	273	12	29
510	1	3	2	2	2574	225	25	45

Int No	Obs No	No Lanes Main St	No Lanes Side St	Main St Speed	Main St Volume	Side St Volume	Avg Delay	% Rt Turns
130	1	1	1	2	719	350	3	86
130	2	1	1	2	986	607	4	83
130	3	1	1	2	1220	760	40	90
130	4	1	1	2	1346	446	6	84
158	1	1	1	1	86	226	5	3
158	2	1	1	1	129	508	26	1
158	3	1	1	1	99	264	11	0
158	4	1	1	1	99	98	9	0
201	1	1	1	3	1253	165	21	45
201	2	1	1	3	1507	124	47	44
201	3	1	1	3	1682	173	42	58
201	4	1	1	3	2132	204	117	60
201	5	1	1	3	2208	199	172	75
201	6	1	1	3	1600	192	67	53
201	7	1	1	3	1284	104	24	42
230	1	2	1	2	924	168	97	57
230	2	2	1	2	922	175	25	66
230	3	2	1	2	652	108	24	67
310	1	2	1	2	296	34	16	35
310	2	2	1	2	434	237	10	41
310	3	2	1	2	754	194	15	37
310	4	2	1	2	602	78	11	49
363	1	2	1	3	427	69	9	42
363	2	2	1	3	609	113	7	23
363	3	2	1	3	665	117	10	10
363	4	2	1	3	489	36	6	0
405	1	2	1	2	2328	327	37	95
405	2	2	1	2	2201	247	23	97
405	3	2	1	2	2268	317	16	96
405	4	2	1	2	2227	429	59	98
405	5	2	1	2	1562	96	25	85
418	1	3	1	3	1469	79	16	76
418	2	3	1	3	1578	51	24	67
418	3	3	1	3	1785	190	17	85
418	4	3	1	3	1682	160	25	82
432	1	2	1	3	1978	70	30	66
432	2	2	1	3	2378	60	44	80
432	3	2	1	3	1387	65	17	71
439	1	2	1	3	1842	140	42	41
439	2	2	1	3	1793	123	34	43
439	3	2	1	3	1999	147	29	56
439	4	2	1	3	1918	245	37	63
439	5	2	1	3	2083	192	22	54
445	1	2	1	2	972	69	52	52
445	2	2	1	2	1245	100	33	48
445	3	2	1	2	1310	83	28	66
445	4	2	1	2	1101	62	38	61
411	1	3	1	3	2936	134	31	86
411	2	3	1	3	2696	187	44	87
411	3	3	1	3	3022	137	28	88

Int No	Obs No	No Lanes Main St	No Lanes Side St	Main St Speed	Main St Volume	Side St Volume	Avg Delay	% Rt Turns
510	2	3	2	2	2469	153	16	56
510	3	3	2	2	2364	146	33	54
529	1	1	2	3	500	226	44	4
529	2	1	2	3	870	219	42	5
529	3	1	2	3	656	243	25	10
534	1	2	2	2	1609	143	14	22
534	2	2	2	2	1472	160	32	34
534	3	2	2	2	1782	66	10	27
557	1	1	2	3	1325	412	107	96
557	2	1	2	3	1252	389	77	98
601	1	2	2	4	2074	264	131	92
601	2	2	2	4	2354	324	351	87
601	3	2	2	4	1872	348	305	85
601	4	2	2	4	1949	350	461	92
645	1	3	2	2	3195	99	32	90
645	2	3	2	2	3710	101	87	88
645	3	3	2	2	3847	105	80	82
677	1	1	2	2	704	279	20	34
677	2	1	2	2	1127	312	35	38
677	3	1	2	2	1373	473	102	53
677	4	1	2	2	1140	338	47	36
601	0	2	2	4	2010	315	89	95
620	0	2	2	2	1269	453	27	63
645	0	3	2	2	3255	174	154	83

APPENDIX D

STATISTICAL TABLES FOR FIELD DATA ANALYSIS

Regression Analysis For:
1 Lane on Side Street

Equation Number 1   Dependent Variable..   LNDELAY

Beginning Block Number 1. Method: Enter
MAIN   SPEED   MAINVOL   SIDEVOL

Variable(s) Entered on Step Number
1.. SIDEVOL
2.. MAINVOL
3.. SPEED
4.. MAIN

Multiple R	.53627		
R Square	.28759	R Square Change	.28759
Adjusted R Square	.24505	F Change	6.76161
Standard Error	.64798	Signif F Change	.0001

Analysis of Variance

	DF	Sum of Squares	Mean Square
Regression	4	11.35611	2.83903
Residual	67	28.13159	.41987

F = 6.76161   Signif F = .0001

------------------ Variables in the Equation ------------------

Variable	B	SE B	Beta	Cor	Part Cor	Partial
SIDEVOL	-9.2194E-04	7.02087E-04	-.1780	-.0327	-.1354	-.1584
MAINVOL	6.2409E-04	1.38957E-04	.6162	.4396	.4631	.4810
SPEED	.0138	.15840	.0110	.2543	.0090	.0106
MAIN	-.4200	.14500	-.4189	-.0190	-.2987	-.3336
(Const)	3.1533	.49905				

----------- in ------------

Variable	T	Sig T
SIDEVOL	-1.313	.1936
MAINVOL	4.491	.0000
SPEED	.087	.9306
MAIN	-2.897	.0051
(Constant)	6.319	.0000

------------- Variables not in the Equation -------------

Variable	Beta In	Partial	Min Toler	T	Sig T
PCTRT	-.07608	-.06610	.45021	-.538	.5923

End Block Number 1   All requested variables entered.

Beginning Block Number 2. Method: Enter PCTRT

Variable(s) Entered on Step Number
5.. PCTRT

Multiple R	.53916		
R Square	.29070	R Square Change	.00311
Adjusted R Square	.23696	F Change	.28964
Standard Error	.65144	Signif F Change	.5923

Analysis of Variance

	DF	Sum of Squares	Mean Square
Regression	5	11.47902	2.29580
Residual	66	28.00867	.42437

F = 5.40986 Signif F = .0003

------------------ Variables in the Equation ----------------

Variable	B	SE B	Beta	Cor	Part Cor	Partial
SIDEVOL	-7.1918E-04	8.0013E-04	-.1388	-.0327	-.0931	-.10997
MAINVOL	6.59723E-04	1.5459E-04	.6514	.4396	.4424	.46503
SPEED	.0299	.1620	.0239	.2543	.0191	.02277
MAIN	-.4141	.1462	-.4130	-.0190	-.2936	-.32922
PCTRT	-1.9792E-03	3.67758E-0	-.0760	.1872	-.0557	-.06610
(Constant)	3.13300	.5031				

----------- in ------------

Variable	T	Sig T
SIDEVOL	-.899	.3720
MAINVOL	4.267	.0001
SPEED	.185	.8538
MAIN	-2.832	.0061
PCTRT	-.538	.5923
(Constant)	6.227	.0000

End Block Number 2 All requested variables entered.

Summary table

Step	MultR	Rsq	F(Eqn)	SigF		Variable	BetaIn
1					In:	SIDEVOL	-.0327
2					In:	MAINVOL	.4426
3					In:	SPEED	.0807
4	.5363	.2876	6.762	.000	In:	MAIN	-.4190
5	.5392	.2907	5.410	.000	In:	PCTRT	-.0761

Residuals Statistics:

	Min	Max	Mean	Std Dev	N
*PRED	2.3995	3.9739	3.0901	.4021	72
*RESID	-1.7327	1.8340	-.0000	.6281	72
*ZPRED	-1.7176	2.1981	.0000	1.0000	72
*ZRESID	-2.6597	2.8153	-.0000	.9641	72

Total Cases = 72

Durbin-Watson Test = 1.49210

Outliers - Standardized Residual

Case #	*ZRESID
16	2.81532
1	-2.65974
2	-2.21391
4	-2.13078
13	1.80147
44	1.68516
5	-1.57759
51	-1.56812
26	-1.38107
66	1.37880

Regression Analysis For:
2 Lanes on the Side Street

```
Equation Number 1    Dependent Variable..   LNDELAY

Beginning Block Number  1.  Method:  Enter
   MAIN      SPEED     MAINVOL  SIDEVOL

Variable(s) Entered on Step Number
   1..     SIDEVOL
   2..     SPEED
   3..     MAINVOL
   4..     MAIN

Multiple R              .78726
R Square                .61979          R Square Change      .61979
Adjusted R Square       .58743          F Change           19.15358
Standard Error          .81020          Signif F Change      .0000

Analysis of Variance
                   DF       Sum of Squares        Mean Square
Regression          4             50.29197           12.57299
Residual           47             30.85223             .65643

F =       19.15358        Signif F =  .0000

---------------- Variables in the Equation ----------------

Variable       B          SE B       Beta   Cor Part Cor Partial

SIDEVOL 5.24818E-04 5.9717E-04  .0872-.13988  .07905  .12715
SPEED         .8139       .2819  .2890 .44231  .25967  .38811
MAINVOL 1.44502E-03 2.1919E-04 1.2933 .46928  .59294  .69313
MAIN        -1.3315       .2958 -.8859 .11599 -.40475 -.54874
(Constant)   1.5494       .8638

----------- in ------------

Variable           T  Sig T

SIDEVOL          .879  .3840
SPEED           2.887  .0059
MAINVOL         6.592  .0000
MAIN           -4.500  .0000
(Constant)      1.794  .0793
------------- Variables not in the Equation -------------

Variable      Beta In  Partial  Min Toler        T  Sig T

PCTRT          .38479   .60538     .20461    5.159  .0000

Equation Number 1    Dependent Variable..   LNDELAY

End Block Number   1   All requested variables entered.
```

Beginning Block Number 2. Method: Enter PCTRT

Variable(s) Entered on Step Number
5.. PCTRT

Multiple R	.87128		
R Square	.75913	R Square Change	.13934
Adjusted R Square	.73295	F Change	26.61068
Standard Error	.65184	Signif F Change	.0000

Analysis of Variance

	DF	Sum of Squares	Mean Square
Regression	5	61.59883	12.31977
Residual	46	19.54537	.42490

F = 28.99455 Signif F = .0000

--------------- Variables in the Equation ----------------

Variable	B	SE B	Beta	Cor	Part Cor	Partial
SIDEVOL	4.10989E-04	4.8095E-04	.068	-.1398	.0618	.1250
SPEED	.7013	.2278	.249	.4423	.2227	.4132
MAINVOL	1.29468E-03	1.7874E-04	1.158	.4692	.5241	.7299
MAIN	-1.2366	.2387	-.822	.1159	-.3747	-.6068
PCTRT	.0152	2.9461E-03	.384	.5462	.3732	.6053
(Constant)	1.1444	.6997				

----------- in ------------

Variable	T	Sig T
SIDEVOL	.855	.3972
SPEED	3.078	.0035
MAINVOL	7.243	.0000
MAIN	-5.179	.0000
PCTRT	5.159	.0000
(Constant)	1.636	.1086

Summary table

Step	MultR	Rsq	F(Eqn)	SigF		Variable	BetaIn
1					In:	SIDEVOL	-.1399
2					In:	SPEED	.4525
3					In:	MAINVOL	.5316
4	.7873	.6198	19.154	.000	In:	MAIN	-.8859
5	.8713	.7591	28.995	.000	In:	PCTRT	.3848

Residuals Statistics:

	Min	Max	Mean	Std Dev	N
*PRED	.9919	5.3557	3.2564	1.0990	52
*RESID	-.9934	1.2926	-.0000	.6191	52
*ZPRED	-2.0605	1.9101	-.0000	1.0000	52
*ZRESID	-1.5240	1.9831	-.0000	.9497	52

Total Cases = 52

Durbin-Watson Test = 1.28189

Outliers - Standardized Residual

Case #	*ZRESID
42	1.98305
1	1.81680