NDOT Research Report

Report No. 224-14-803

Task Order 7

Right-Turn Traffic Volume Adjustment in Traffic Signal Warrant Analysis

May 2016

Nevada Department of Transportation
1263 South Stewart Street
Carson City, NV 89712

Disclaimer

Final Report
May 6, 2016

Right-turn Traffic Volume Adjustment in Traffic Signal Warrant Analysis

SOLARIS Consortium, Tier 1 University Transportation Center
Center for Advanced Transportation Education and Research
Department of Civil and Environmental Engineering
University of Nevada, Reno
Reno, NV 89557

Zong Tian
University of Nevada, Reno

***DISCLAIMER*:**

Abstract

This report was based on the research project, Right-Turn Traffic Volume Adjustment in Traffic Signal Warrants, sponsored by the Nevada Department of Transportation (NDOT) and SOLARIS. Right-turn traffic does not affect intersection performance in the same magnitude as through or left-turn traffic. Therefore, it is necessary to apply an adjustment to the right-turn volume when conducting signal warrant analysis. Without any reduction, an intersection with heavy right-turn volume might mislead the signal warrant analysis result, and could make a difference in whether a signal is deemed warranted or not.

Firstly, a comprehensive literature review was conducted focusing on the state-of-the-practice handling of minor-street right-turn volumes while conducting signal warrant studies. Further, an agency survey through the Institute of Transportation Engineers (ITE) community discussion was performed to acquire valuable information from practicing engineers. It was found that the limited guidance in the Manual on Uniform Traffic Control Devices (MUTCD) does not provide a clear direction on determining whether or how much right turns impact the signal warrant analysis. In reality, most traffic engineers have done the reduction based on engineering judgments by incorporating key factors such as geometry and main street volume. Sometimes agencies develop and adopt internal procedures but do not necessarily publish them.

Based on the lack of an adequate guideline, a new one is needed to estimate the reduction factor for right-turn traffic on the minor street when conducting a traffic signal warrant study. The proposed guideline is based on the delay equivalent relationship between right-turn and through traffic. The right-turn volume equals an equivalent number of through vehicles, which would produce the same control delay on the minor street. The equivalent factor is defined as the measurement of the reduction of right turns. Because equivalent factors are calculated based on delay, it incorporates major impact factors of the right-turn and through traffic inherently, such as flow rates, conflicting flow rates, capacity, critical headways, and follow-up headways. Especially, the volume ratio in the two directions of the main street is considered. The research found that uneven volume distribution has a greater impact on the right-turn movement on the minor street. Therefore, just considering the main street volume can cause over- or under-estimation of the impact of the main street traffic on the minor street.

Further, regression models were developed for all the configurations with calibrated regression coefficients. The advantage of these models is that they could give an equivalent factor for a specific volume scenario. The proposed guidelines were tested at three intersections and the results indicated that they are convenient to use and easily help to determine right-turn volume equivalents.

Lastly, pedestrian impact on right-turn traffic adjustment was discussed. Usually,

pedestrians crossing the main street would block right-turn vehicles on the minor street, and on the other hand, the through vehicles on the minor street can use this gap to cross an intersection. A Monte Carlo model was built to simulate the real operation of two-way stop-control (TWSC) intersections, and further validated with field data that was collected at one intersection near the University of Nevada, Reno (UNR) campus. With this model, minor-street through capacity considering pedestrian crossings was estimated. Using this enlarged capacity allows more accurate calculation of equivalent factors by considering the counter impact of pedestrians on right turns.

In summary, this research focused on the right-turn adjustment in the signal warrant. The decision on reducing the right turns on the minor street is somewhat subjective. Therefore, this study developed a practical guidance for determining the percentage of right turns to be considered in the signal warrant analysis. Based on the data analysis and case study results, statewide uniform guidelines were developed for implementation in the State of Nevada. However, the recommendations reached in this research could be applied in other states as well.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Chapter 1 Introduction

1.1 Background

Traffic signals are signaling devices positioned at roadway intersections, pedestrian crossings, and other locations to control conflicting flows of vehicles and pedestrians. Installation of traffic signals usually does not have a detrimental effect on both operations and safety. However, unwarranted signals might cause some problems. Traffic control signals that are properly located, operated and maintained typically have one or more of the following advantages:

- Signals provide for the orderly movement of traffic by assigning right-of-way to conflicting traffic movements.
- Signals can increase intersection capacity by permitting conflicting traffic movements to share the same intersection.
- Signals usually reduce the frequency of right-angle (broadside) collisions.
- Signals can provide a progression of traffic through a series of intersections by coordinating adjacent traffic signals.
- Signals will interrupt heavy traffic to allow both lighter vehicular and pedestrian traffic to cross the heavy traffic movement.

Traffic control signals may have one or more of the following disadvantages:

- Signals can increase delay, both for the overall intersection delay and/or delay of a specific movement.
- To avoid signals, drivers sometimes use alternate routes that are less adequate.
- Signals might increase traffic on minor street approaches when drivers wish to use the signal that will interrupt heavy main street traffic.
- Signals might encourage disregard of traffic control devices. When drivers on the minor street approaches have excessive wait times with very little main street traffic, they might “run” the red display.
- Signals tend to increase in the frequency of rear-end collisions.

Traffic signals are the most restrictive type of control at intersections that require conflicting movements to take turns using the intersection. However, traffic signals are the most expensive intersection control, costing between $250,000 and $500,000, depending on the complexity of the intersection and characteristics of the traffic using it. Besides, signals tend to increase accidents, delay, congestion and disobedience of signals.

Therefore, traffic signals should be only installed when they will alleviate more problems than they will create. The decision for traffic signals should be based on competent engineering studies and field observations to ensure that the signal is warranted and will enhance the safety and efficiency of the intersection. Before installation, less restrictive and less expensive control measures should be considered, such as widening the approach, removing roadside parking, adding turn lanes, and roundabouts.

Signal warrant analysis is the first and most important step in the signal installation process to avoid the unnecessary use of signals. There are three vehicle-volume related signal warrants in the MUTCD: Warrant 1) Eight-hour vehicular volume, Warrant 2) Four-hour vehicular volume, and Warrant 3) Peak hour vehicular volume. It is customary to adjust minor street right-turn volumes to allow for the fact that a certain percentage of vehicles can make a right turn without the aid of a traffic signal when a signal is being considered for capacity reasons. High volumes of right-turn vehicles from the minor street can skew a signal warrant analysis and indicate an incorrect need for a signal; consequently, how right-turn volumes are utilized can be important in signal warrant analysis. MUTCD as a guideline clearly states that the study should consider the effects of the right-turn vehicles from the minor-street approaches and engineering judgment should be used. This provides justification for reducing right-turn traffic volumes. However, MUTCD does not offer any clear direction on this matter. Pure “engineering judgment” is subjective in nature and will likely vary from engineer to engineer. Specific guidelines would be helpful when considering the right-turn traffic during signal warrant studies.

The current practice in Nevada involves two different approaches applied in two broad areas. Area 1 encompasses Clark County and its cities and there is a letter of agreement between NDOT and Clark County that, when conducting traffic signal warrant studies, 25% of minor-street right-turn vehicles may be included in the minor-street volume. There is no supporting documentation that identifies how this percentage was developed. Area 2 covers all the remaining areas in the state. Within this broad area, right-turn volumes are entirely removed from the minor-street volume with an underlying assumption that right-turn vehicles can make turns without affecting the intersection performance. Again, no supporting documentation justifies this methodology.

As can be seen from the above discussion, limited information tends to focus on using “engineering judgment” when applying right-turn volume reductions. Limited research on this subject has not led to any identifiable method in developing a standard procedure to deal with minor-street right-turn traffic volumes. In this report, a standard practice for right-turn traffic reduction in signal warrant analysis will be developed for Nevada.

1.2 Objective and Scope

Currently, the specific guidelines are based on engineering judgment at NDOT and local agencies regarding right-turn volume reductions in signal warrant studies. No documentation could be found to support the current practice and limited research on this subject has not identified a method to deal with minor-street right-turn traffic volumes.

The primary objectives of this research are:

- To develop guidelines when right-turn traffic volumes should be reduced during traffic signal warrant analysis in both urban and rural settings in two-way stop control intersection;
- To develop software tool(s) or tables that can assist agencies in conducting signal warrant analysis.

1.3 Organization of Report

The report documents all the findings and conclusions pertaining to right-turn traffic volume adjustment in traffic signal analysis. Chapter 1 introduces the background, objectives and scope of this research. Chapter 2 is a comprehensive literature review which documents current practices across the U.S. Chapter 3 presents the proposed methodology. Chapter 4 presents the regression equation for equivalent factors. Chapter 5 introduces three case studies at intersections in Nevada. Chapter 6 focuses on the pedestrian impact on right-turn traffic volume adjustment. Finally, Chapter 7 summarizes the major findings and contributions of this research.

Chapter 2 Literature Review

Over the years, studies conducted in the United States have advanced several guidelines helping governments decide under what conditions right-turn traffic could be reduced. Basically, the literature on right-turn reduction methods can be organized as engineering judgment, field observation, and an accepted right-turn adjustment methodology.

2.1 Existing Guidelines

The portion of the right turning traffic that is able to make a movement without experiencing significant delay should be reduced during signal warrant studies. However, if queued vehicles prevent right-turning traffic from flowing freely or if mainline volumes are high enough that even right turning vehicles experience significant delay, the reduction should be used carefully and full right-turn volumes might be used in the warrant analysis.

Section 4C.01 [1] of MUTCD serves as the general guideline and indicates the following:

"The study 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 signal warrants listed in Paragraph 2."

Even though MUTCD states that the right-turn traffic should be subtracted from the minor street, it fails to provide clear guidance. Due to the lack of specific unified guidance for the entire country regarding this matter, several individual states have been developing their own guidelines as seems appropriate according to their situation.

Mozdbar et al. [2] indicated that including the appropriate portion of the right-turn volume in the signal warrant study was critical, as it could make the difference in whether a signal is deemed warranted or not. The City of Austin in Texas had developed the guideline that considered the right-turn volume adjustment based on the application of one of three conditions: accident experience, sight distance obstruction and delay. The highest adjusted right-turn volume would be used in a combination of left-turn and through traffic to carry on signal warrant analysis. These guidelines are only based on engineering experience and practice in signal warrant studies, but not developed through theoretical studies. However, this guideline is able to provide a frame of reference for including the appropriate portion of the right-turn volume.

McDonald [3] examined the methods of DOTs in two states. The State of Illinois DOT was divided into nine districts. Districts One, Two and Four used a process called the "Pagones Theorem", to be discussed below, to reduce the number of right turns on the minor street; District Seven just left the right-turn reduction to the judgment of engineers; Districts Three, Five, Six, Eight and Nine did not reduce any right turns from the minor street when performing signal warrant analysis. The State of Tennessee DOT was divided into four regions and all of them used engineering judgment to perform the right-turn reduction. If the approach had one lane or no right-turn lane, the approach volume was generally not reduced. Reductions were based on traffic volume, storage capacity, and geometrics. In many cases, the assumption was made that the geometry of the approach could be modified to handle an exclusive right-turn lane if the lane would help reduce the need for a signalized intersection. The author also concluded that the engineer should be aware of interstate and intra-state variations in determining right-turn reduction.

The Manual of Traffic Signal Design (MTSD) published by ITE suggested that all right turns might be excluded in the analysis if the approach had a separate right-turn lane and a large-radius curb return. This exclusion could also apply when right turns were made from the through lane and only a small-radius curb return was available.

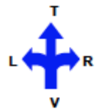
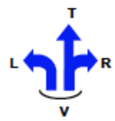
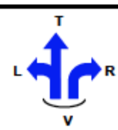
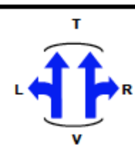
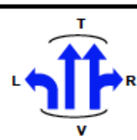
A formal right-turn adjustment methodology has been developed by IIIinois DOT [4] and also been used by Alabama DOT [5]. It is a two-step methodology called Pagones Theorem that uses a minor street equivalent factor and a mainline congestion factor to estimate the portion of right-turn volumes. The adjusted right-turn volume is calculated as follows:

$$R_{adj} = R\times[1-(f_{minor}-f_{main})] \tag{2.1}$$

where R_{adj}= adjusted right turn volume; R = original right turn volume; f_{minor}= minor street adjustment factor; f_{main}= mainline congestion factor. Note: if $f_{minor}-f_{main}<0$, then$R_{adj}=R$.

The minor street adjustment factor reflects whether minor street geometry and traffic volumes permit the free movement of right turns and reduce right-turn volumes accordingly. The mainline congestion factor adjusts to account for the amount of congestion on the mainline. In essence, f_{minor} considers what portion of vehicles could get to the intersection to make a right-turn without delay while f_{main} determines whether there are enough gaps in the mainline traffic to permit them to actually make that right-turn. The suggested values for f_{minor} and f_{main} are listed in Tables 2.1 and 2.2 according to lane configuration and volume condition. For the mainline right-turn reduction, if there is no mainline right-turn lane, mainline right-turn volumes are added to the through volumes for the lane volume calculations; if a right-turn lane is present, mainline right turn volumes are excluded from the calculation.

Table 2.1 Pagones Theorem Right-turn Adjustment Factors

Minor Street Adjustment Factor (f_{minor})			
Case	**Lane Configuration**	**Volume Condition**	f_{minor}
1		R > 0.7V	0.60
		0.7V ≥ R > 0.35V	0.40
		R ≤ 0.35V	0.20
2		R > 3T	0.60
		3T ≥ R > T/3	0.40
		R ≤ T/3	0.20
3		Any configuration with an exclusive right turn lane ≥ 500 ft. long. (See note* for shorter right turn lanes)	0.75
4		R > (T+L)	0.65
		L > (T+R)	Use Case 2
		L ≈ T ≈ R (±10 veh)	0.40
		L ≈ T > 3R	0.20
		R ≈ T > 3L	0.50
		all other conditions	0.30
5		R > T	0.75
		T ≥ R > T/2	0.50
		T/2 ≥ R > T/4	0.30
		R < T/4	0.15

Note: Originally shown in Ref [5].

Table 2.2 Pagones Theorem Mainline Congestion Factors

Mainline Congestion Factor (f_{main})			
Mainline volume per lane (veh/hr/lane)	f_{main}	**Mainline volume per lane (veh/hr/lane)**	f_{main}
0 - 399	0.0	1100 – 1199	0.40
400 – 499	0.05	1200 – 1299	0.45
500 – 599	0.10	1300 – 1399	0.50
600 – 699	0.15	1400 – 1499	0.55
700 – 799	0.20	1500 – 1599	0.60
800 – 899	0.25	1600 – 1699	0.65
900 – 999	0.30	1700 – 1799	0.70
1000 – 1099	0.35	1800 - 1899	0.75

Note: Originally shown in Ref [5].

NCHRP report 457 [6] uses the following method to determine right-turn volumes in signal warrant analysis, which was originally developed by Utah DOT. In this method, the actual right-turn volume is reduced on the basis of consideration of the major-road volume that conflicts with the right-turn movement, the number of traffic lanes serving the conflicting volume, and the geometry of the subject minor-road approach. The relationship between these factors is illustrated in Figure 2.1.

Erika Mar
Deleted:

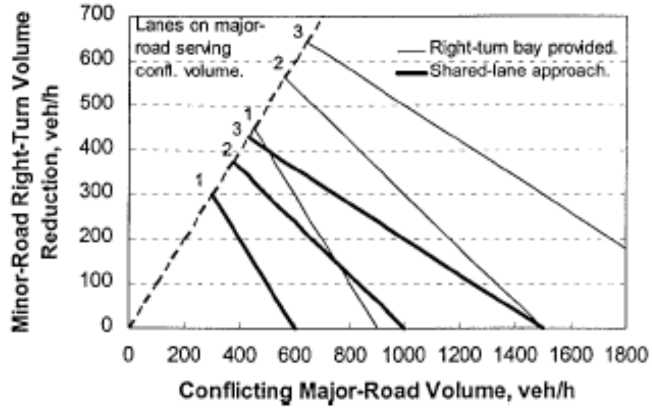


Figure 2.1 Minor Street Right-turn Volume Reduction in NCHRP method
Note: Originally shown in Ref [6].

To determine if a heavy right-turn volume might mislead the signal warrant analysis, the following adjusted minor street volume would be performed. The adjusted volume is computed as follows:

$$\text{Adjusted minor street volume} = \text{Max}\begin{bmatrix} V_7 + V_8 + V_9 - V_{r9} \\ V_{10} + V_{11} + V_{12} - V_{r12} \end{bmatrix} \tag{2.2}$$

$$V_{c9} = 0.5V_3 + \frac{V_2}{N_2}$$

$$V_{c12} = 0.5V_6 + \frac{V_5}{N_5}$$

$$V_9 - V_{r9} \geq 0$$

$$V_{12} - V_{r12} \geq 0$$

where V_i= volume for movement i (movement numbers are shown in Figure 2.2), N_i= number of approach lanes serving through movement i; V_{r9} (V_{r12}) = right-turn volume reduction for movement 9(12), obtained from Figure 2.2 using conflicting major street volume; $V_{c9}(V_{c12})$ = conflicting major street volume for movement 9(12).

Also, the “Right-turn bay provided” case in Figure 2.1 could be used for shared-lane approaches when the shared lane functions as a de facto right-turn lane. If this warrant check yields different conclusions than the original warrant check (i.e., with unadjusted volumes), then right-turn volumes might be enough to affect the accuracy of the warrant check. In this situation, it is recommended that the effect of right-turns be fully examined during the warrant study.

Erika Mar
Deleted:

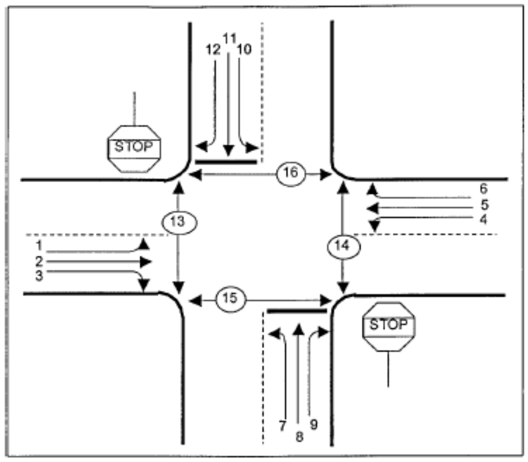


Figure 2.2 Movement Numbers for Right-turn Volume Adjustment

State of Wisconsin DOT [7] suggested that before evaluating traffic volumes against the warrant criteria, inclusions of right-turn vehicles shall be considered. The number of right-turn vehicles included in the intersection analysis played an important role in the overall operation of the intersection. The traffic control for the right turning vehicles should be known prior to determining the percentage of volume inclusion. The department used three right-turn inclusion percentages based on the impact of the right turns on the operation of the intersection. Figure 2.3 shows lane configurations and corresponding percentages.

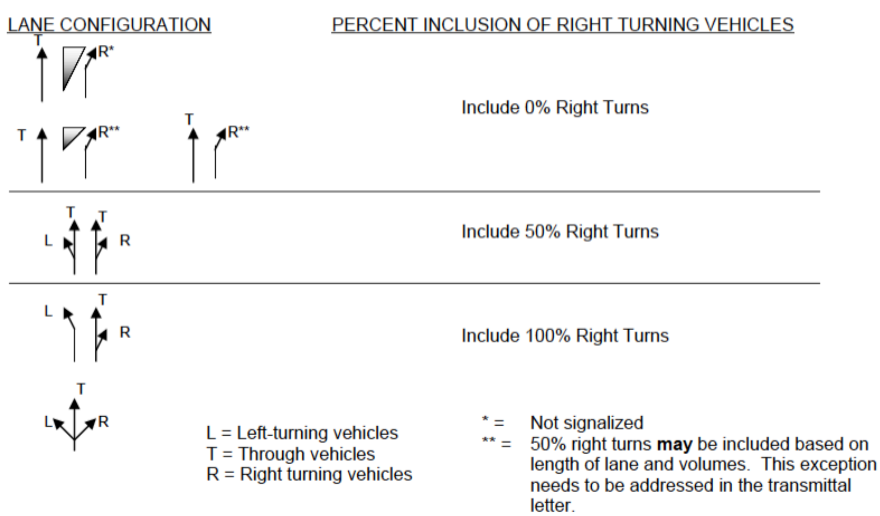


Figure 2.3 Right-turn Inclusion Percentages (WDOT)
Note: Originally shown in Ref [8].

Los Angeles DOT [8] indicated that if the right turning traffic was delayed less than 45 seconds under stop sign control, the right turning volume should be subtracted from the side-street volume. The rationale for this subtraction was that side-street traffic that waits less than 45 seconds would likely turn right on red; thus, it would not benefit from traffic signal control. Therefore, for Warrants 1, 2, and 3, if right-turning vehicles were delayed less than 45 seconds under stop control and there were no more than two right-turn collisions in the most recent 12-month period, then those vehicles shall be subtracted from the side street volume.

Oregon DOT [9,10] suggested that 85% of the right-turn lane or shared lane capacity was subtracted from the right-turn volume. If the value of 85% of the lane capacity (measured in vph) exceeded the right-turn volume, no right-turn volumes were included in the analysis. If the right-turn volumes were greater, they were reduced by 0.85% of the lane capacity. This method takes into account not only traffic volumes on the minor street, but also traffic condition on the main street which affect the ability of vehicles to turn right from the minor street. Arizona DOT [11] recommends that the adjusted right-turn volume equals the total right-turn volume minus the right-turn volume experiencing a stopped-delay measurement of five seconds or less on the higher volume minor-street approach.

In an existing signal warrant report [12], the authors advised that when there was an exclusive right turn, the right-turn volume could be subtracted from the total volume of the approach. When the road consisted of a single approach lane for all movements, there was no reduction in volume for right-turn vehicles.

From the available literature, we found that most states simply follow the MUTCD recommendation for adjusting right-turn volumes and roughly base it on engineering experience.

2.2 ITE Community Discussion

To collect more information regarding the reduction of right-turn vehicles in signal warrant analysis in current practice, this topic was posted in the ITE community discussion section in October 2013. Eight responses were received in this survey. The corresponding responses are presented and summarized below.

Traffic engineers from Wisconsin DOT and Illinois DOT mentioned that their states had written policies about right-turn reduction and provided their states' methods. A transportation planner from a consulting firm said his office used the recommendations based on NCHRP 457, which attributed its methodology to the Utah DOT. Detailed introduction of all these three methods is presented in the former section.

For DOTs that have no written policy on this matter, traffic engineers have their own consideration procedures. A traffic engineer from the City of Federal Way said, "I don't include right-turn volume at all if the LOS for that movement is A, but otherwise include all of it." A senior engineer from Lee County DOT mentioned that if there was a right-turn only lane, he would deduct the number of left turns from the right-turn volume using the justification that if there was enough of a gap for a left turn, then there was a gap for a right turn. Another traffic engineer from Lee County said that if there was no right-turn lane, he used the entire approach volume for the warrant analysis and he didn't consider the effect of a small right turn channel. Particularly, if there were a lot of U-turns that conflict with the right turns, then he might want to consider a greater percentage of the right-turning traffic in the count. An area traffic engineer working at Virginia DOT said that when he was researching the same question, he discovered the Pagones Theorem used by Illinois DOT District 1 and found it useful in reduction calculations.

Particularly, the President of Yarger Engineering, Inc. responded with an extensive post about the right-turn inclusion in the signal warrant analysis that highlights the discrepancies of practice:

"*This area is a real gray area and I would hate to see any hard and fast rules, but more guidance would be very helpful at least for some consistency and reasonableness. The all or none call seems to be unreasonable and there should be some guidance that says under x conditions, reduce #%, and under y conditions, reduce #+1%... Why should an extra second of delay on a right turn from 9.5 second of delay to 10.5 seconds flip from reducing all to none? If LOS A is all, shouldn't LOS be more like 80% to 90%, with LOS C being 60% to 70% and so on? I have seen numerous approaches. Indiana DOT has a procedure, but they won't share it with the rest of us, so in the absence of something better, I run the question backwards. I calculate what percentage can be excluded and still warrant the signal, and then see if that looks reasonable given the unsignalized levels of services for the right turn and also compare it with Synchro's RTOR flow rate from the signalized analysis. The issue is that in most cases, we are not talking about reducing the right turns in the peak hours, but in the eighth highest hour in order to satisfy the 8 hour warrants. The right turn during the 8th hour typically is LOS A or B if in an exclusive lane.*

I believe if there is an exclusive right turn and RTOR would be permitted if there was a signal, then some of them need to be reduced. I have been overruled on this where the reviewer included 100%. I have also been on the other side where the reviewer didn't want the signal, and said to exclude all of the right turns. Recently, when in a TIS report I said that in 10 years with an assumed development and most of the vehicles being right turners, which it was too close to call, the reviewer said I had to make a decision now and send that portion of the study back for revision."

2.3 Problems in Existing Guidelines

All these methods are means of estimating the volume of right-turning traffic that would not benefit from the provision of a signal. However, most of them are based on engineering judgments and there are no theoretical supports behind them. Los Angeles DOT's guideline is based on field observation. It seems reasonable but is hard to use in reality. Pagones Theorem and the NCHRP method seem to be more robust, but no published literature was found to document the algorithms and theories behind these two methods. Besides, even though Pagones Theorem has considered the main street volume, it fails to take into account the uneven volume distribution in two directions. The NCHRP method works out the reduced right-turn volume purely based on the conflicting major-road volume and whether a right-turn bay is provided. It does not consider the through traffic in the minor street at all, but in the reality, the through traffic and right turn traffic often disturb each other. Further, this method does not provide the inherent relationship between minor-street right-turn volume reduction and conflicting major-street volume except for a graph. From the case study in a later chapter, this method tends to reduce right turns too much.

2.4 Chapter Summary

Unwarranted traffic signals are detrimental for several reasons not only to the flow of traffic but may also increase overall delay. Including all right-turn traffic volumes or an inappropriate portion of the right-turn traffic volume, could result in an erroneous traffic study and possible installation of an unwarranted traffic signal. In existing reports and guidelines, engineers and scholars generally agree on reducing right-turn volumes in signal warrant analysis, but as to right-turn traffic reduction percentages, there is no mature theoretical methodology and reduction factors are basically based on engineering judgment.

Currently, nearly all right-turn reductions are implemented on minor streets, and only Pagones Theorem has provided quantitative reduction guidance on main streets. Minor streets reduction factors normally relate to lane configuration, capacity, minor and main streets traffic volumes, right-turn traffic percentage, delay, crash experience and sight distance.

A specific and detailed right-turn volume reduction model, which considers important aspects of right-turn traffic, is important for signal warrant analysis. Proper right-turn traffic reduction is beneficial for evaluating the justification of signals.

Chapter 3 Proposed Methodology

Our approach for the right-turn volume adjustment focuses on traffic operation principles, i.e. delay or level-of-service. This LOS-based approach has been successfully applied to determining intersection control types [13,14]. The approach proposed is based on the principle of delay equivalence, i.e., to find the delay equivalent relationship between right-turn and through traffic under different conditions. The control delay estimation is based on the Highway Capacity Manual 2010 (HCM) [15] procedure for two-way stop-controlled intersections. To expedite the data analysis process, the HCM analysis procedure is implemented in Excel using Visual Basic, which allows quick analysis of multiple scenarios.

An isolated intersection shown in Figure 3.1 is used. The subject movements are northbound through and right turn. Both movements have a direct crossing or merging conflict with all of the major street movements, except the right turn into the subject approach. We assume that all the traffic on the main street is through movement from both directions. In the analysis, the volume distribution in the two directions of the main street is considered and defined in Equation 3.1.

Erika Mar
Deleted:

$$VR = \frac{V_1}{V_2} \tag{3.1}$$

where VR is the volume ratio of the main street; V_1 is the farther side of main street to the subject minor street and V_2 is the nearer side of main street to the subject minor street.

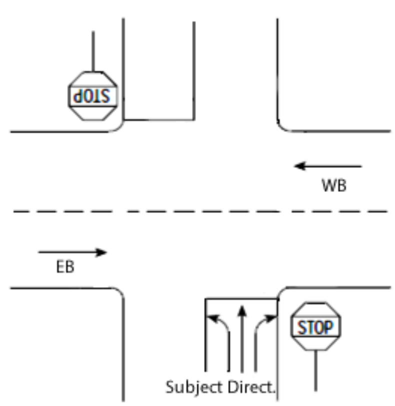
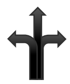
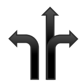
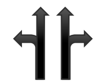
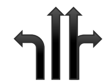


Figure 3.1 Study Intersection

For Figure 3.1, the volume ratio is the volume of the westbound divided by that of the eastbound. Furthermore, according to minor-street lane configurations, five configurations are discussed as shown in Table 3.1.

Erika Mar
Deleted:

Table 3.1 Minor Street Lane Configuration

Configuration	1	2	3	4	5
Lane Configuration					

3.1 Configuration 1

Configuration 1 depicts shared lane geometry on the minor street. The volume ranges covered in the analysis are listed in Table 3.2. These volume combinations yield a total of 12,096 (9×7×8×24) cases. Each main-street volume and a volume ratio work together as one study situation. There are 63 (9×7) study situations in total for Configuration 1 and the configurations thereafter.

In each study situation, there are 192 (8×24) combinations of minor-street volume scenarios. For the minor-street left and through movements, 20% left turns are assumed.

Table 3.2 Scenarios Evaluated in Configuration 1

Item	Range
Major Street (9)	400, 500, 600, 700, 800, 900, 1000, 1100, 1200 vph
Volume ratio (7)	1:1, 1:2, 1:3, 1:4, 2:1, 3:1, 4:1
Minor Street Right Turn (8)	50, 100, 150, 200, 250, 300, 350, 400 vph
Minor Street Left turn and Through (24)	40, 60, 80, 100, 120, 140, 160, 180, 200, 220, 240, 260, 280, 300, 320, 340, 360, 380, 400, 420, 440, 460, 480, 500 vph

Under one volume scenario shown in Table 3.2, for example, major street volume is 400 vph; volume ratio is 1:3 (i.e. the westbound is 100 vph and the eastbound is 300 vph); minor street right turn is 400 vph and minor street left turn and through is 500 vph (i.e. left turn= 500×20%=100 vph and through volume is 500-100=400 vph). The information is listed in Table 3.3.

In this condition, the shared lane delay is 232.1 sec/veh. The right turn traffic is eliminated and the through traffic volume increases until the control delay arrives at 232.1 sec/veh, which is illustrated in Figure 3.2.

Before After

Figure 3.2 Right-turn Conversion

For the analysis, the right-turn equivalent factor (equivalent factor for short) is defined

Erika Mar
Deleted:

Erika Mar
Deleted:

Erika Mar
Deleted:

in Equation 3.2. By means of "equivalence", it is to find out the amount of right-turn traffic that is equivalent to the amount of through traffic in order to yield the same control delay.

$$EF = \frac{T_2 - T_1}{R} \tag{3.2}$$

where EF is the right-turn equivalent factor; T_1 is through volume before equivalence; T_2 is through volume after equivalence without right-turn traffic; R is right-turn volume before equivalence.

The adjusted right-turn volume could be estimated by the equivalent factor as follows:

$$R_{adj} = R \times EF \tag{3.3}$$

For example, before reduction, the right-turn volume is 100 vph and the equivalent factor is 0.8, so the adjusted right-turn volume used for signal warrants should be 80 vph. It is obvious that the larger the equivalent factor, the more right-turn volume would be used for a warrant check.

Table 3.3 Equivalent factor Calculation Example for Configuration 1

Configuration 1 			Major Volume		400
			Volume Ratio		1/3
Before Equivalence					
Major Street		Subject Minor Street			Delay
EB	WB	LT	T	RT	
300	100	100	400	400	232.1
After Equivalence					
Major Street		Subject Minor Street			Delay
EB	WB	LT	T	RT	
300	100	100	688	0	232.1
Equivalent Factor	EF= (688-400)/400 = 0.72				

Figure 3.3 shows equivalent factors for one specific study situation where the mainline volume is 500 vph with a 1:1 volume ratio (a total of 192 scenarios). From the graph, we can see that when right-turn volumes increase, equivalent factors increase accordingly. Under the same right-turn volume (such as right turn volume of 300 vph, green line), when there are more left-turn and through vehicles on the minor street, equivalent factors would increase and it would use more right-turn traffic for a warrant check. In general, when the minor-street traffic increases, equivalent factors tend to converge to a fixed value (0.59 in this case). This value is the largest number of the entire equivalent factors in this study situation and defined as the situation equivalent factor.

Erika Mar
Deleted:

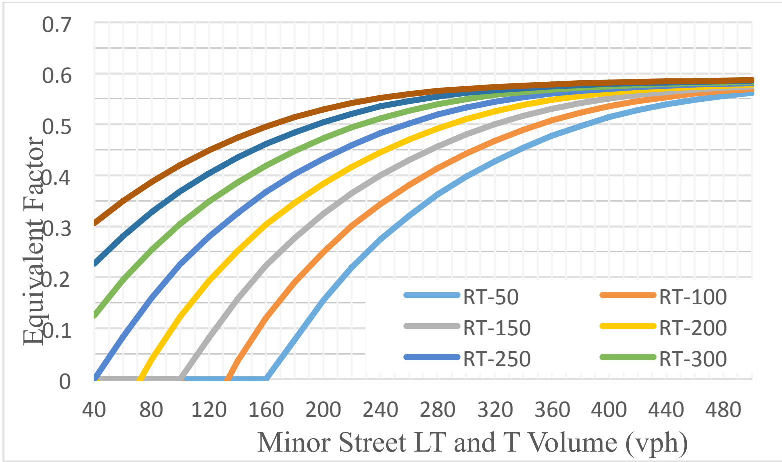


Figure 3.3 Equivalent Factor Graph for One Study Situation: main street volume= 500 vph; *VR*= 1:1

Table 3.4 and Figure 3.4 show equivalent factors under different main-street volumes and volume ratios. When mainline volume increases, the equivalent factor decreases. It can be explained by the fact that the main street volume affects more through vehicles than right turns; therefore, delay increases more for minor-street through vehicles than that of right turns. When the mainline volume is higher than 1200 vph, the same equivalent factors for the 1200 vph level will apply. In reality, the main-street volume may not fall exactly in the same values in the table; it is recommended using lower bound values.

Erika Mar
Deleted:
Erika Mar
Deleted:

Table 3.4 Situation Equivalent Factors for Configuration 1

Main Street Volume / Volume Ratio	400	500	600	700	800	900	1000	1100	1200
1:1	0.64	0.59	0.55	0.52	0.48	0.45	0.42	0.39	0.36
1:2	0.69	0.66	0.63	0.60	0.57	0.54	0.52	0.49	0.47
1:3	0.72	0.70	0.68	0.64	0.62	0.60	0.58	0.56	0.54
1:4	0.74	0.72	0.70	0.68	0.66	0.64	0.62	0.60	0.58
2:1	0.57	0.52	0.47	0.43	0.39	0.37	0.33	0.29	0.26
3:1	0.55	0.49	0.44	0.40	0.36	0.32	0.29	0.26	0.23
4:1	0.53	0.47	0.42	0.38	0.34	0.30	0.27	0.24	0.21

*When the main street volume is beyond 1200 vph, equivalent factors of 1200 vph are applied.

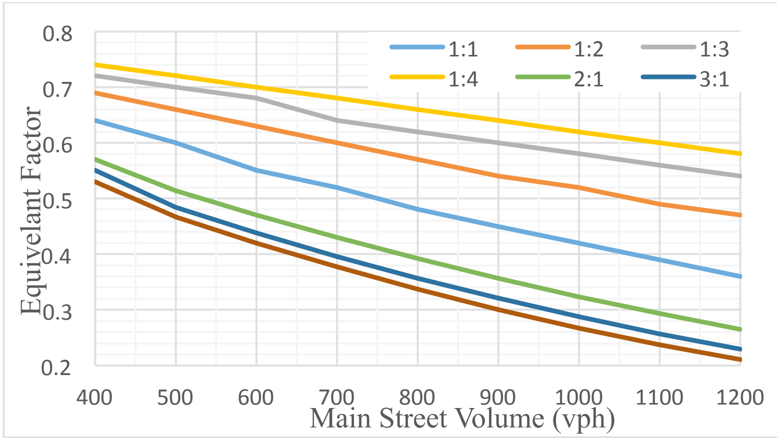


Figure 3.4 Situation Equivalent Factor Graph for Configuration 1

3.2 Configuration 2

Configuration 2 is a shared right-through lane with an exclusive left-turn lane. Using the same traffic volume scenarios, almost the same equivalent factors are obtained. So it is reasonable to treat Configurations 1 and 2 as one category. It also shows that the assumed left-turn percentage does not significantly affect the equivalent relation. This phenomenon could be explained by showing that the left-turn traffic has the same impact on the through and right-turn vehicles.

3.3 Configuration 3

The geometry of Configuration 3 is a shared left-through lane with exclusive right-turn lane shown in Table 3.1. Because there is a right-turn lane, the delay of the right-turn movement is irrelevant to the through movement, so it is assumed that the through movement is zero. Different traffic volume scenarios are listed in Table 3.5. Before the reduction, the right-turn movement is 50 vph to 510 vph with a 20-vph increment. After reduction, 20 left turns are assumed for the minor-street left movements. A total of 24 scenarios are considered in each study situation.

Table 3.5 Scenarios Evaluated in Configuration 3

Item	Range
Major Street (9)	400, 500, 600, 700, 800, 900, 1000, 1100, 1200 vph
Volume ratio (7)	1:1, 1:2, 1:3, 1:4, 2:1, 3:1, 4:1
Minor Street Right Turn (24)	50, 70, 90, 110, 130, 150, 170, 190, 210, 230, 250, 270, 290, 310, 330, 350, 370, 390, 410, 430, 450, 470, 490, 510 vph
Minor Street Left turn and Through (0)	0

Under one volume scenario shown in Table 3.5, for example (listed in Table 3.6), the

Erika Mar
Deleted:

Erika Mar
Deleted:

major street volume is 400 vph; volume ratio is 1:3 (i.e. the westbound is 100 vph and the eastbound is 300 vph); minor street right turn is 510 vph and minor street left turn is 20 vph. Under this condition, the right-turn lane delay is 19.7 sec/veh. The right turn traffic is then eliminated and the through traffic volume is increased until the left and through lane control delay arrives at 19.7 sec/veh. Using Equation 3.2, the equivalent factor is 0.55.

Table 3.6 Equivalent factor Calculation Example for Configuration 3

Configuration 3 		Major Volume		400	
		Volume Ratio		1/3	
Before Equivalence					
Major Street		Subject Minor Street			Right-turn lane Delay
EB	WB	LT	T	RT	
300	100	20	0	510	19.7
After Equivalence					
Major Street		Subject Minor Street			Left and through lane Delay
EB	WB	LT	T	RT	
300	100	20	283	0	19.7
Equivalent Factor	*EF*= 283/510 = 0.55				

Equivalent factors for these 63 study situations are shown in Table 3. It is easy to observe that the effect of volume ratio is obvious. If the volume distribution in the two directions of the main street is not considered, it may often reduce too many right turns with exclusive right-turn lanes. Most agencies are inclined to exclude all the right-turn traffic in this geometry.

Table 3.7, equivalent factors vary from 0.09 to 0.60 with different volume ratios when the main street volume is 400 vph. This phenomenon tells us that it is not proper to reduce all the right-turn volume when there is more traffic near the minor street.

Because right-turn vehicles have a separate lane, their movement may not be affected by the through and left-turn conducting signal warrant analysis. There are two ways to consider the minor street volume and the lane number, which is introduced in MUTCD 3C.01. 13. For the first approach, the minor street has two lanes (a shared through lane and a right-turn lane). The minor street volume is the sum of adjusted right-turn, and through and left-turn traffic volumes. For the other approach, the minor street has one lane. Under this configuration, the minor street volume is the maximum volume of adjusted right-turn traffic, and through and left-turn traffic, which is defined as the critical volume.

Erika Mar
Deleted:

Table 3.7 Situation Equivalent factors for Configuration 3

Main Street Volume / Volume Ratio	400	500	600	700	800	900	1000	1100	1200
1:1	0.36	0.33	0.30	0.29	0.28	0.27	0.26	0.25	0.24
1:2	0.49	0.48	0.48	0.47	0.46	0.45	0.44	0.42	0.40
1:3	0.55	0.55	0.55	0.55	0.54	0.53	0.52	0.50	0.48
1:4	0.60	0.60	0.60	0.60	0.59	0.58	0.56	0.55	0.53
2:1	0.21	0.20	0.11	0.07	0.03	0	0	0	0
3:1	0.14	0.07	0	0	0	0	0	0	0
4:1	0.09	0.02	0	0	0	0	0	0	0

*When the main street volume is beyond 1200 vph, equivalent factors of 1200 vph are applied.

3.4 Configuration 4

The lane geometry in Configuration 4 is two lanes with shared right-turn and left-turn as shown in Table 3.1. Traffic volume scenarios are listed in Table 3.8. For the minor-street left and through movements, 20% left turns are assumed. In each study situation (total of 63), 204 (34×6) cases are evaluated.

Table 3.8 Scenarios Evaluated in Configuration 4

Item	Range
Major Street (9)	400, 500, 600, 700, 800, 900, 1000, 1100, 1200 vph
Volume ratio (7)	1:1, 1:2, 1:3, 1:4, 2:1, 3:1, 4:1
Minor Street Right Turn (6)	50, 100, 150, 200, 250, 300 vph
Minor Street Left turn and Through (34)	40, 60, 80, 100, 120, 140, 160, 180, 200, 220, 240, 260, 280, 300, 320, 340, 360, 380, 400, 420, 440, 460, 480, 500, 520, 540, 560, 580, 600, 620, 640, 660, 680, 700 vph

Under one volume scenario shown in Table 3.8, for example, major street volume is 400 vph; volume ratio is 1:1 (i.e. the westbound is 200 vph and the eastbound is 200 vph); minor street right turn is 300 vph and minor street left-turn and through volume is 520 vph (i.e. left turn= 520×20%= 104 vph and through volume is 520 - 104=416 vph). Under this condition, the right-turn lane delay is 23.5 sec/veh. All the right turn traffic is eliminated and the through traffic volume is increased until the left and through lane control delay arrives at 23.5 sec/veh. The equivalent factor calculated by Equation 4 is 0.60. Detail information is listed in Table 3.9.

Erika Mar
Deleted:

Erika Mar
Deleted:

Erika Mar
Deleted:

Erika Mar
Formatte
grammar

Erika Mar
Deleted:

Table 3.9 Equivalent factor Calculation Example for Configuration 4

Configuration 4		Major Volume		400	
		Volume Ratio		1/1	
Before Equivalence					
Major Street		Subject Minor Street			Right-turn Delay
EB	WB	LT	T	RT	
200	200	104	416	300	23.5
After Equivalence					
Major Street		Subject Minor Street			Through Delay
EB	WB	LT	T	RT	
200	200	104	595	0	23.5
Equivalent Factor	*EF*= (595-416)/300 = 0.60				

Figure 3.5 depicts the equivalent factor when the mainline volume is 500 vph and the right-turn volume is 400 vph. From the picture, the maximum equivalent factor is not in the highest minor-street left-turn and through volume, but corresponds to a certain middle level. It is mainly because the capacity of through traffic is relatively large in this geometry.

Erika Mar
Deleted:

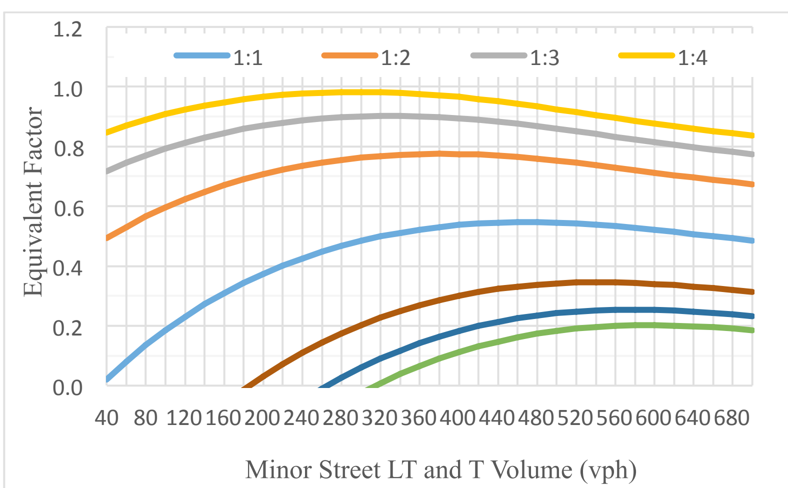


Figure 3.5 Equivalent factor Graph for Mainline Volume 500 vph

Table 3.10 Situation Equivalent factors for Configuration 4

Main Street Volume / Volume Ratio	400	500	600	700	800	900	1000	1100	1200*
1:1	0.60	0.55	0.51	0.48	0.46	0.44	0.42	0.40	0.38
1:2	0.80	0.78	0.76	0.75	0.74	0.73	0.73	0.71	0.70
1:3	0.91	0.90	0.90	0.90	0.91	0.91	0.91	0.90	0.90
1:4	0.98	0.98	0.99	1.00	1.00	1.00	1.00	1.00	1.00
2:1	0.42	0.35	0.29	0.25	0.22	0.20	0.17	0.15	0.13
3:1	0.34	0.25	0.19	0.15	0.12	0.10	0.07	0.05	0.03
4:1	0.29	0.20	0.14	0.09	0.06	0.04	0.02	0	0

*When the main street volume is beyond 1200 vph, equivalent factors of 1200 vph are applied.

3.5 Configuration 5

Configuration 5 has three lanes: exclusive left turn, exclusive through and a shared through and right-turn lane. Traffic volume scenarios are listed in Table 3.11. Because there is an exclusive left-turn lane, left-turn traffic is not considered in this condition.

Erika Mar
Deleted:

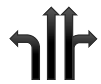
Table 3.11 Scenarios Evaluated in Configuration 5

Item	Range
Major Street (9)	400, 500, 600, 700, 800, 900, 1000, 1100, 1200 vph
Volume ratio (7)	1:1, 1:2, 1:3, 1:4, 2:1, 3:1, 4:1
Minor Street Right Turn (8)	50, 100, 150, 200, 250, 300, 350, 400 vph
Minor Street Through (24)	40, 60, 80, 100, 120, 140, 160, 180, 200, 220, 240, 260, 280, 300, 320, 340, 360, 380, 400, 420, 440, 460, 480, 500 vph

Under one volume scenario shown in Table 3.11, for example, the major street volume is 600 vph; volume ratio is 1:2 (i.e. the westbound is 200 vph and the eastbound is 400 vph); minor street right turn is 250 vph and minor street through volume is 260 vph. Under this condition, the right-turn lane delay is 19.96 sec/veh. All the right turn traffic is eliminated increase the through traffic volume is increased until the left and through lane control delay arrives at 19.96 sec/veh. The equivalent factor is 0.196. The equivalent factors when the main-street volume is 500 vph with 400 vph right turns are illustrated in Figure 3.6. Table 3.13 gives the suggested equivalent factors in all study situations.

Erika Mar
Deleted:

Table 3.12 Equivalent factor Calculation Example for Configuration 5

Condition 5 		Major Volume		600	
		Volume Ratio		1/2	
Before Reduction					
Major Street		Subject Minor Street			Right-turn and Though lane Delay
EB	WB	LT	T	RT	
400	200	0	260	250	19.96
After Reduction					
Major Street		Subject Minor Street			Through lane Delay
EB	WB	LT	T	RT	
400	200	0	309	0	19.96
Equivalent Factor	*EF* = (309-260)/250 = 0.196				

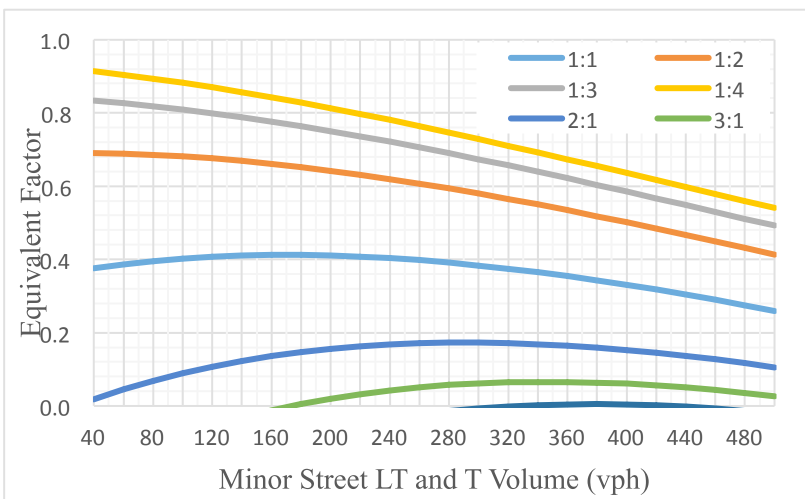


Figure 3.6 Equivalent Factor Graph for Mainline Volume 500 vph

Table 3.13 Situation Equivalent Factors for Configuration 5

Man Street Volume / Volume Ratio	400	500	600	700	800	900	1000	1100	1200*
1:1	0.45	0.41	0.38	0.37	0.36	0.35	0.35	0.34	0.34
1:2	0.71	0.69	0.68	0.68	0.68	0.68	0.68	0.67	0.66
1:3	0.83	0.83	0.84	0.84	0.84	0.84	0.84	0.83	0.82
1:4	0.91	0.91	0.92	0.93	0.93	0.93	0.93	0.93	0.92
2:1	0.24	0.17	0.13	0.11	0.09	0.08	0.07	0.07	0.07
3:1	0.14	0.07	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4:1	0.04	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

*When the main street volume is beyond 1200 vph, equivalent factors of 1200 vph are applied.

3.6 Discussion of Proposed Method

The proposed method is based on the relative delay relationship between through and right-turn traffic. When the main street volume increases, minor-street through traffic suffer from more conflicting volume than right turns. So the delay of through traffic increases much more than the right turns in the minor street, which explains why equivalent factors decrease when the main street volumes increase. However, this phenomenon is not fit for reality. In reality, the right-turn traffic experiences more difficulty entering the intersection when the main street traffic volume increases.

For Warrant 1 eight-hour vehicular volume, the threshold volumes are fixed. It does not consider the relationship between main-street and minor-street traffic. It is not proper just to converge right turns to through traffic. Therefore, to amend the proposed method, it is recommended to apply the equivalent factors for the main street volume of 400 vph for all main street volume conditions.

For Warrant 2 four vehicular volume and Warrant 3 peak hour, the required minor street volume decreases with the increase of the main street volume. It considers the relationship between the main street and minor street volumes. So in the reality, it is proper to converge the right turns to through traffic.

Chapter 4 Regression Equations for Equivalent Factors

In Chapter 3, the proposed reduction method was introduced in detail. The equivalent factor is the maximum value of the entire volume scenario and the volume range is relatively wide to consider different conditions. For a specific case, the equivalent factor may be not exact but tends to be conservative. Even though equivalent factor graphs (such as Figure 3.3) can give the equivalent results for the covered volume conditions, equivalent factors are not continuous and it is not easy to extract other conditions. Therefore, in this section, regression models are developed by a statistical method. Equations and regression coefficients for all these four configurations and are provided.

Erika Mar
Deleted:

4.1 Configuration 1 and 2

The equivalent factors are primarily calculated based on delay. The regression models are inspired by a two-way stop-control delay function. Thus, the equations for Configurations 1 through 5 are similar. The regression equation for Configurations 1 and 2 is shown as follows:

$$f = a\left(1 - \frac{1}{(bV_{T+L} + cV_R)^d}\right) \tag{4.1}$$

where f is the equivalent factor; V_{T+L} is the volume of through and left-turn traffic; V_R is the right-turn traffic volume; a, b, c, d are the regression coefficients.

To calculate the regression factors, the MATLAB R2013a [16] curve fitting toolbox is used.

Figure 4.1 is the toolbox interface. There are four steps to complete the fitting:

(1) Input fitting data: through and left-turn volume (X data); right-turn volume (Y data); equivalent factor (Z data);

(2) Choose Custom Equation and input Equation 4.1.

(3) Change setting in Fit Option (Figure 4.2),

Robust: off; Algorithm: Trust Region;

Specify starting conditions and define lower and upper bounds

(4) Read results: coefficients and goodness of fit.

Erika Mar
Formatte
0.17"

Erika Mar
Formatte

Erika Mar
Deleted:

Erika Mar
Delete

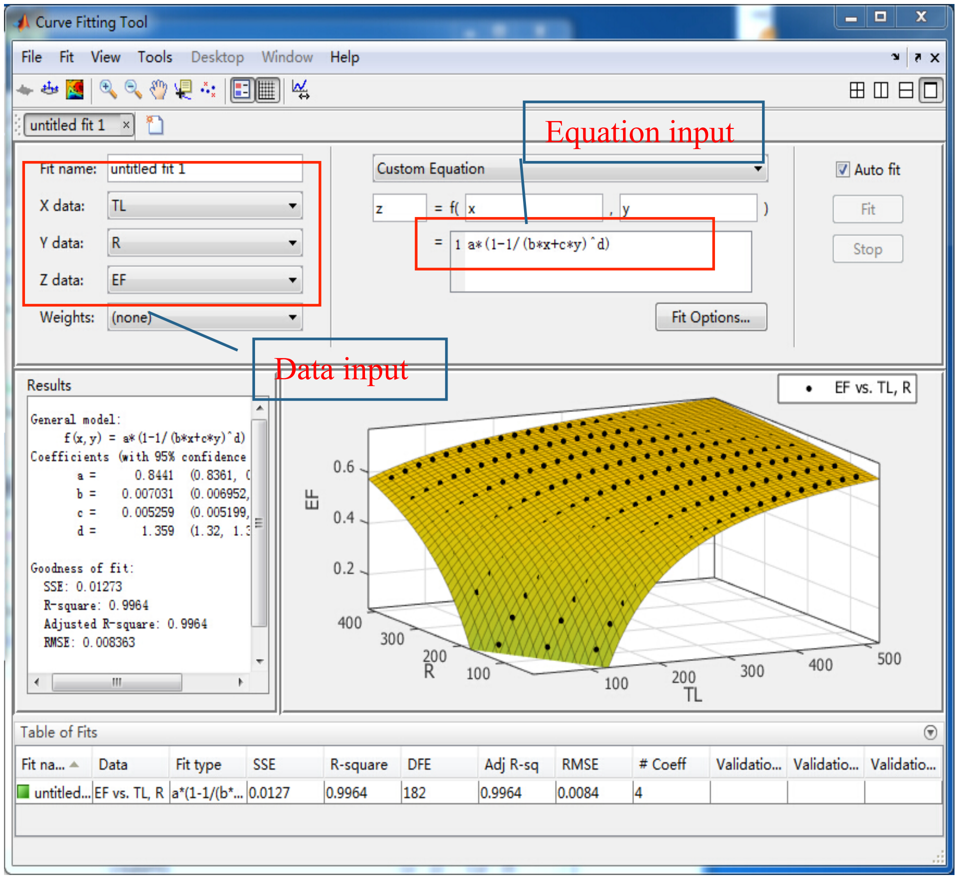


Figure 4.1 MATLAB Curve Fitting Interface: main street volume =400 vph; *VR*=1:4

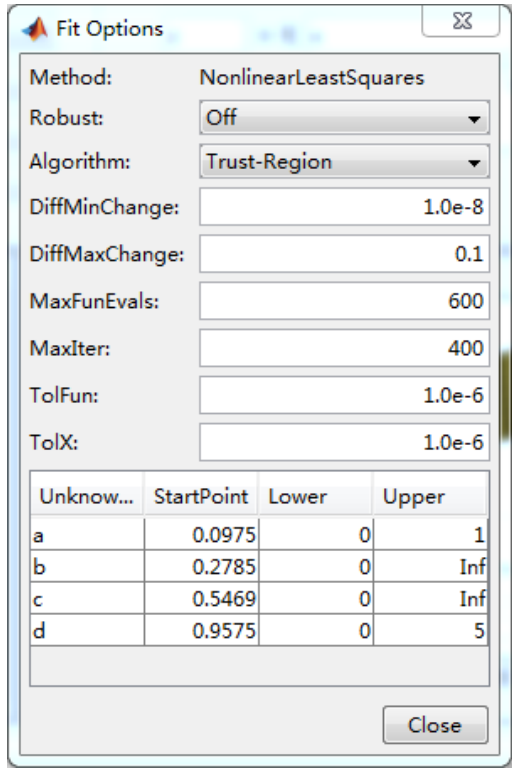


Figure 4.2 Fit Option Setting: main street volume =400 vph; VR=1:4

From Figure 4.1, we can see that all points scatter around the fitting surface. The coefficient of determination R^2 reaches up to 0.9964 and the sum of square error (SSE) is only 0.01273. Therefore, the proposed regression model could describe the equivalent factors almost perfectly for this scenario. It should be noticed that if the volume of through and left-turn traffic V_{T+L} and the right-turn traffic volume V_R are smaller than certain values, the equivalent factor will fall below zero, which is meaningless. In this condition, the equivalent factor should be reset to zero. Table 4.1 lists 63 study situations' regression coefficients and R^2 for Configurations 1 and 2. Three significant figures are shown. This method is applicable to all other configurations.

Table 4.1 Regression Coefficients and R^2 for Configuration 1 and 2

	400						500				
	a	b	c	d	R^2		a	b	c	d	R^2
1:1	0.754	0.00498	0.00321	1.52	0.997	1:1	0.671	0.00503	0.00304	1.83	0.995
1:2	0.806	0.00593	0.00413	1.42	0.997	1:2	0.741	0.00605	0.00404	1.65	0.995
1:3	0.829	0.00660	0.00483	1.37	0.988	1:3	0.780	0.00680	0.00483	1.54	0.994
1:4	0.844	0.00703	0.00526	1.36	0.996	1:4	0.803	0.00742	0.00540	1.48	0.994
2:1	0.706	0.00436	0.00259	1.61	0.997	2:1	0.607	0.00437	0.00239	2.01	0.996
3:1	0.678	0.00410	0.00234	1.69	0.997	3:1	0.574	0.00411	0.00215	2.13	0.996
4:1	0.661	0.00396	0.00222	1.74	0.998	4:1	0.555	0.00397	0.00202	2.21	0.996
	600						700				
	a	b	c	d	R^2		a	b	c	d	R^2
1:1	0.602	0.00516	0.00294	2.16	0.994	1:1	0.543	0.00539	0.00288	2.49	0.993
1:2	0.687	0.00625	0.00405	1.86	0.993	1:2	0.636	0.00655	0.00403	2.15	0.991
1:3	0.730	0.00703	0.00487	1.77	0.992	1:3	0.687	0.00743	0.00497	1.98	0.989
1:4	0.758	0.00772	0.00550	1.68	0.992	1:4	0.713	0.00791	0.00554	2.00	0.989
2:1	0.527	0.00449	0.00227	2.43	0.995	2:1	0.462	0.00465	0.00220	2.87	0.995
3:1	0.491	0.00421	0.00202	2.61	0.996	3:1	0.426	0.00439	0.00194	3.03	0.995
4:1	0.472	0.00409	0.00189	2.68	0.996	4:1	0.403	0.00425	0.00179	3.24	0.996
	800						900				
	a	b	c	d	R^2		a	b	c	d	R^2
1:1	0.495	0.00566	0.00288	2.78	0.994	1:1	0.454	0.00607	0.00293	2.95	0.994
1:2	0.593	0.00683	0.00407	2.44	0.991	1:2	0.559	0.00722	0.00435	2.56	0.991
1:3	0.650	0.00777	0.00508	2.22	0.989	1:3	0.617	0.00815	0.00522	2.47	0.990
1:4	0.689	0.00864	0.00604	2.01	0.987	1:4	0.659	0.00910	0.00630	2.22	0.988
2:1	0.412	0.00496	0.00215	3.13	0.995	2:1	0.366	0.00521	0.00213	3.52	0.996
3:1	0.373	0.00465	0.00188	3.41	0.996	3:1	0.328	0.00489	0.00186	3.81	0.996
4:1	0.350	0.00444	0.00174	3.74	0.996	4:1	0.309	0.00479	0.00173	3.78	0.995
	1000						1100				
	a	b	c	d	R^2		a	b	c	d	R^2
1:1	0.417	0.00719	0.00296	3.11	0.995	1:1	0.384	0.00758	0.00308	3.29	0.995
1:2	0.524	0.00850	0.00432	2.84	0.994	1:2	0.499	0.00942	0.00471	2.68	0.993
1:3	0.590	0.00976	0.00551	2.54	0.992	1:3	0.564	0.0103	0.00577	2.70	0.995
1:4	0.635	0.01096	0.00676	2.25	0.988	1:4	0.611	0.01162	0.00714	2.40	0.991
2:1	0.328	0.00613	0.00215	3.68	0.995	2:1	0.297	0.00661	0.00227	3.52	0.992
3:1	0.292	0.00586	0.00185	3.77	0.994	3:1	0.260	0.00620	0.00195	3.70	0.991
4:1	0.272	0.00561	0.00174	3.92	0.992	4:1	0.241	0.00603	0.00179	3.73	0.988
	1200*										
	a	b	c	d	R^2						
1:1	0.355	0.00833	0.00316	3.19	0.992						
1:2	0.471	0.01007	0.00480	2.84	0.995						
1:3	0.543	0.01168	0.00626	2.53	0.994						
1:4	0.589	0.01253	0.00723	2.54	0.995						
2:1	0.266	0.00716	0.00221	3.58	0.987						
3:1	0.231	0.00674	0.00195	3.56	0.985						
4:1	0.212	0.00645	0.00181	3.62	0.978						

*When the main street volume is beyond 1200 vph, equivalent factors of 1200 vph are applied.

4.2 Configuration 3

The regression equation for Configuration 3 is shown in Equation 4.2. Because there is an exclusive right-turn lane, left-turn and through volumes are not considered. The same procedures are applied to calculate the regression coefficients as Configurations 1 and 2.

Figure 4.3 is an example of the regression model. The equation is good enough to explain the original data. Table 4.3 lists regression coefficients and coefficients of determination for 63 study situations in Configuration 3.

$$f = a\left(1 - \frac{1}{bV_R^c}\right) \tag{4.2}$$

where *f* is the equivalent factor; V_R is the right-turn traffic volume; *a*, *b*, *c* are regression coefficients.

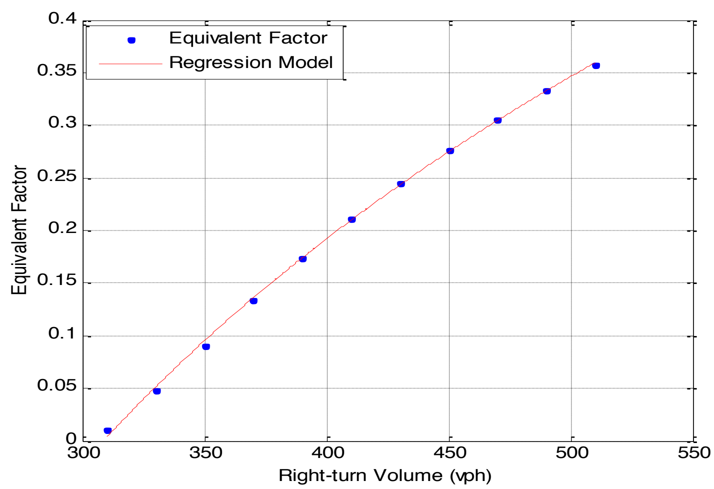


Figure 4.3 Regression Model Example for Configuration 3: main street volume =400 vph; *VR*=1:1

Erika
Dele

Table 4.2 Regression Coefficients and R^2 for Configuration 3

	400					500			
	a	b	c	R^2		a	b	c	R^2
1:1	2.79	0.208	0.274	0.999	1:1	2.92	0.200	0.278	0.999
1:2	1.54	0.058	0.518	1.000	1:2	1.13	0.0124	0.794	1.000
1:3	1.37	0.045	0.582	0.999	1:3	1.02	0.00746	0.911	1.000
1:4	1.31	0.042	0.606	0.999	1:4	1.01	0.00758	0.926	1.000
2:1	3.38	0.254	0.230	0.999	2:1	3.08	0.188	0.277	1.000
3:1	2.77	0.188	0.276	0.998	3:1	-	-	-	-
4:1	-	-	-	-	4:1	-	-	-	-
	600					700			
	a	b	c	R^2		a	b	c	R^2
1:1	3.006	0.197	0.278	0.998	1:1	3.14	0.194	0.279	0.996
1:2	1.010	0.00521	0.946	1.000	1:2	1.02	0.00540	0.939	0.998
1:3	0.940	0.00338	1.06	1.000	1:3	0.899	0.00223	1.13	0.999
1:4	0.915	0.00283	1.11	1.000	1:4	0.837	0.00102	1.31	1.000
2:1	-	-	-	-	2:1	-	-	-	-
3:1	-	-	-	-	3:1	-	-	-	-
4:1	-	-	-	-	4:1	-	-	-	-
	800					900			
	a	b	c	R^2		a	b	c	R^2
1:1	3.05	0.193	0.279	0.997	1:1	3.05	0.194	0.278	0.995
1:2	0.967	0.00371	1.01	0.996	1:2	0.769	0.0004981	1.37	0.997
1:3	0.754	0.000272	1.53	1.000	1:3	0.721	0.0002067	1.58	0.999
1:4	0.826	0.00108	1.30	0.999	1:4	0.738	0.000269	1.58	0.999
2:1	-	-	-	-	2:1	-	-	-	-
3:1	-	-	-	-	3:1	-	-	-	-
4:1	-	-	-	-	4:1	-	-	-	-
	1000				1100				
	a	b	c	R^2		a	b	c	R^2
1:1	2.93	0.192	0.280	0.992	1:1	1.53	0.0440	0.531	0.992
1:2	0.802	0.00150	1.18	0.991	1:2	0.665	0.000357	1.45	0.990
1:3	0.710	0.00036	1.49	0.995	1:3	0.612	2.94E-05	1.96	0.995
1:4	0.656	3.00E-05	2.00	0.999	1:4	0.642	5.74E-05	1.89	0.997
2:1	-	-	-	-	2:1	-	-	-	-
3:1	-	-	-	-	3:1	-	-	-	-
4:1	-	-	-	-	4:1	-	-	-	-
	1200*								
	a	b	c	R^2					
1:1	1.32	0.0367	0.563	0.989					
1:2	0.530	1.93E-05	1.93	0.992					
1:3	0.573	2.73E-05	2.00	0.994					
1:4	0.624	0.000116	1.78	0.991					
2:1	-	-	-	-					
3:1	-	-	-	-					
4:1	-	-	-	-					

*When the main street volume is beyond 1200 vph, equivalent factors of 1200 vph are applied.

4.3 Configuration 4

The regression equation for Configuration 4 is shown in Equation 4.3. The same procedures are applied to calculate the regression coefficients as Configurations 1 and 2.

$$f = \left(1 - \frac{V_{T+L}^{0.558}}{(bV_{T+L}^{-0.227} + cV_{R}^{0.062})^{4.65}}\right) \tag{4.3}$$

where f is the equivalent factor; V_{T+L} is the volume of through and left-turn traffic; V_R is the right-turn traffic volume and a, b, c, d are regression coefficients.

Figure 4.4 is an example of the regression model. Equivalent factors scatter around the surface closely. Table 4.3 lists regression coefficients and coefficients of determination for 63 study situations in Configuration 4.

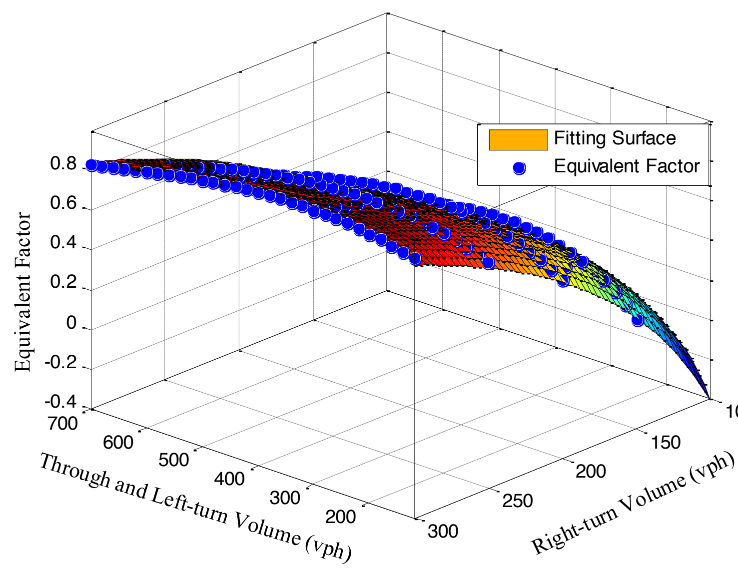


Figure 4.4 Regression Model Example for Configuration 4: main street volume =400 vph; *VR*=1:4

Table 4.3 Regression Coefficients and R^2 for Configuration 4

	400					500			
	a	b	c	R^2		a	b	c	R^2
1:1	0.754	0.00498	0.00321	0.997	1:1	0.671	0.00503	0.00304	0.995
1:2	0.806	0.00593	0.00413	0.997	1:2	0.741	0.00605	0.00404	0.995
1:3	0.829	0.00660	0.00483	0.988	1:3	0.780	0.00680	0.00483	0.994
1:4	0.844	0.00703	0.00526	0.996	1:4	0.803	0.00742	0.00540	0.994
2:1	0.706	0.00436	0.00259	0.997	2:1	0.607	0.00437	0.00239	0.996
3:1	0.678	0.00410	0.00234	0.997	3:1	0.574	0.00411	0.00215	0.996
4:1	0.661	0.00396	0.00222	0.998	4:1	0.555	0.00397	0.00202	0.996
	600					700			
	a	b	c	R^2		a	b	c	R^2
1:1	0.754	0.00498	0.00321	0.997	1:1	0.671	0.00503	0.00304	0.995
1:2	0.806	0.00593	0.00413	0.997	1:2	0.741	0.00605	0.00404	0.995
1:3	0.829	0.00660	0.00483	0.988	1:3	0.780	0.00680	0.00483	0.994
1:4	0.844	0.00703	0.00526	0.996	1:4	0.803	0.00742	0.00540	0.994
2:1	0.706	0.00436	0.00259	0.997	2:1	0.607	0.00437	0.00239	0.996
3:1	0.678	0.00410	0.00234	0.997	3:1	0.574	0.00411	0.00215	0.996
4:1	0.661	0.00396	0.00222	0.998	4:1	0.555	0.00397	0.00202	0.996
	800					900			
	a	b	c	R^2		a	b	c	R^2
1:1	0.495	0.00566	0.00288	0.994	1:1	0.454	0.00607	0.00293	0.994
1:2	0.593	0.00683	0.00407	0.991	1:2	0.559	0.00722	0.00435	0.991
1:3	0.650	0.00777	0.00508	0.989	1:3	0.617	0.00815	0.00522	0.990
1:4	0.689	0.00864	0.00604	0.987	1:4	0.659	0.00910	0.00630	0.988
2:1	0.412	0.00496	0.00215	0.995	2:1	0.366	0.00521	0.00213	0.996
3:1	0.373	0.00465	0.00188	0.996	3:1	0.328	0.00489	0.00186	0.996
4:1	0.350	0.00444	0.00174	0.996	4:1	0.309	0.00479	0.00173	0.995
	1000					1100			
	a	b	c	R^2		a	b	c	R^2
1:1	0.417	0.00719	0.00296	0.995	1:1	0.384	0.00758	0.00308	0.995
1:2	0.524	0.00850	0.00432	0.994	1:2	0.499	0.00942	0.00471	0.993
1:3	0.590	0.00976	0.00551	0.992	1:3	0.564	0.01033	0.00577	0.995
1:4	0.635	0.01096	0.00676	0.988	1:4	0.611	0.01162	0.00714	0.991
2:1	0.328	0.00613	0.00215	0.995	2:1	0.297	0.00661	0.00227	0.992
3:1	0.292	0.00586	0.00185	0.994	3:1	0.260	0.00620	0.00195	0.991
4:1	0.272	0.00561	0.00174	0.992	4:1	0.241	0.00603	0.00179	0.988
	1200								
	a	b	c	R^2					
1:1	0.355	0.00833	0.00316	0.992					
1:2	0.471	0.01007	0.00480	0.995					
1:3	0.543	0.01168	0.00626	0.994					
1:4	0.589	0.01253	0.00723	0.995					
2:1	0.266	0.00716	0.00221	0.987					
3:1	0.231	0.00674	0.00195	0.985					
4:1	0.212	0.00645	0.00181	0.978					

*When the main street volume is beyond 1200 vph, equivalent factors of 1200 vph are applied.

4.4 Configuration 5

The regression equation for Configuration 5 is shown in Equation 4.4. The same procedures are applied to calculate regression coefficients as Configurations 1 and 2.

$$f = a\left(1 - \frac{V_{T+L}^{0.493}}{(bV_{T+L}^{-0.199} + cV_{R}^{0.119})^{3.25}}\right) \tag{4.4}$$

where f is the equivalent factor; V_{T+L} is the volume of through and left-turn traffic; V_R is the right-turn traffic volume and a, b, c, d are regression coefficients.

Figure 4.5 is one example of the regression model in Configuration 5. The original data basically sit around the regression surface. Table 4.4 lists regression coefficients and coefficients of determination for 63 study situations in Configuration 5.

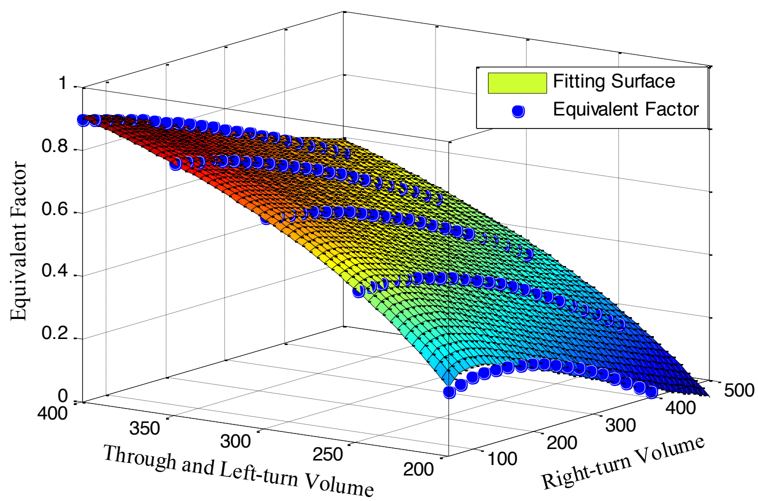


Figure 4.5 One Regression Model Example for Configuration 5: main street volume =400 vph; volume ratio=1:4

Table 4.4 Regression Coefficients and R^2 for Configuration 5

	400					500			
	a	b	c	R^2		a	b	c	R^2
1:1	0.754	0.00498	0.00321	0.997	1:1	0.671	0.00503	0.00304	0.995
1:2	0.806	0.00593	0.00413	0.997	1:2	0.741	0.00605	0.00404	0.995
1:3	0.829	0.00660	0.00483	0.988	1:3	0.780	0.00680	0.00483	0.994
1:4	0.844	0.00703	0.00526	0.996	1:4	0.803	0.00742	0.00540	0.994
2:1	0.706	0.00436	0.00259	0.997	2:1	0.607	0.00437	0.00239	0.996
3:1	0.678	0.00410	0.00234	0.997	3:1	0.574	0.00411	0.00215	0.996
4:1	0.661	0.00396	0.00222	0.998	4:1	0.555	0.00397	0.00202	0.996
	600					700			
	a	b	c	R^2		a	b	c	R^2
1:1	0.754	0.00498	0.00321	0.997	1:1	0.671	0.00503	0.00304	0.995
1:2	0.806	0.00593	0.00413	0.997	1:2	0.741	0.00605	0.00404	0.995
1:3	0.829	0.00660	0.00483	0.988	1:3	0.780	0.00680	0.00483	0.994
1:4	0.844	0.00703	0.00526	0.996	1:4	0.803	0.00742	0.00540	0.994
2:1	0.706	0.00436	0.00259	0.997	2:1	0.607	0.00437	0.00239	0.996
3:1	0.678	0.00410	0.00234	0.997	3:1	0.574	0.00411	0.00215	0.996
4:1	0.661	0.00396	0.00222	0.998	4:1	0.555	0.00397	0.00202	0.996
	800					900			
	a	b	c	R^2		a	b	c	R^2
1:1	0.495	0.00566	0.00288	0.994	1:1	0.454	0.00607	0.00293	0.994
1:2	0.593	0.00683	0.00407	0.991	1:2	0.559	0.00722	0.00435	0.991
1:3	0.650	0.00777	0.00508	0.989	1:3	0.617	0.00815	0.00522	0.990
1:4	0.689	0.00864	0.00604	0.987	1:4	0.659	0.00910	0.00630	0.988
2:1	0.412	0.00496	0.00215	0.995	2:1	0.366	0.00521	0.00213	0.996
3:1	0.373	0.00465	0.00188	0.996	3:1	0.328	0.00489	0.00186	0.996
4:1	0.350	0.00444	0.00174	0.996	4:1	0.309	0.00479	0.00173	0.995
	1000				1100				
	a	b	c	R^2		a	b	c	R^2
1:1	0.417	0.00719	0.00296	0.995	1:1	0.384	0.00758	0.00308	0.995
1:2	0.524	0.00850	0.00432	0.994	1:2	0.499	0.00942	0.00471	0.993
1:3	0.590	0.00976	0.00551	0.992	1:3	0.564	0.01033	0.00577	0.995
1:4	0.635	0.01096	0.00676	0.988	1:4	0.611	0.01162	0.00714	0.991
2:1	0.328	0.00613	0.00215	0.995	2:1	0.297	0.00661	0.00227	0.992
3:1	0.292	0.00586	0.00185	0.994	3:1	0.260	0.00620	0.00195	0.991
4:1	0.272	0.00561	0.00174	0.992	4:1	0.241	0.00603	0.00179	0.988
	1200*								
	a	b	c	R^2					
1:1	0.355	0.00833	0.00316	0.992					
1:2	0.471	0.01007	0.00480	0.995					
1:3	0.543	0.01168	0.00626	0.994					
1:4	0.589	0.01253	0.00723	0.995					
2:1	0.266	0.00716	0.00221	0.987					
3:1	0.231	0.00674	0.00195	0.985					
4:1	0.212	0.00645	0.00181	0.978					

*When the main street volume is beyond 1200 vph, equivalent factors of 1200 vph are applied.

Chapter 5 Case Studies

Three intersections were selected as case studies to introduce how to apply the proposed procedures. The first two cases were from the NDOT previous signal warrant analysis data. The third one at Blue Diamond Road and South El Capitan Way was a detailed case to apply the proposed method.

The first two intersections were chosen among 26 intersections with data provided by NDOT. At first, according to the eight-hour warrant in MUTCD, these intersections were divided into three categories:

(1) the minor-street through and left-turn volumes met eight-hour warrant without right turning movement, which included seven cases;

(2) all minor-street turning volumes did not meet the warrant, which included 12 cases;

(3) when considering all turning movements where the warrant was met, which included seven cases.

Among these three categories, the first two could be easily determined for warrants being met without considering the proper portion of right-turn volume reduction. The third category is the study focus, in which how much percentage of right-turn traffic utilized in the signal warrant influences the justified result. From the seven cases in the third category, two intersections are demonstrated here.

5.1 Lamoille Highway and Spring Creek Parkway

The first intersection located at Lamoille Highway and Spring Creek Parkway in the rural area is shown in Figure 5.1. Lamoille Highway is the main street (east and west bounds) with three lanes and Spring Creek Parkway is the minor street with shared through and a left-turn lane and an exclusive right-turn lane. For the eight-hour warrant verification, with all turning volume, Condition A was justified and the vehicular data is shown in Table 5.1. It is obvious that this intersection has very high right-turn traffic, which would skew the warrant study results. By means of the proposed method, the equivalent factor and adjusted right-turn volume are listed in Table 5.2.

In each time period, there were two ways to consider the number of lanes and minor-street volume, which was introduced in Chapter 3.3. For the first one, the minor street with two lanes was considered. The minor street volume was the sum of adjusted right-turn, through and left-turn traffic volumes. The required eight-hour vehicular volumes (Warrant 1 Condition A, MUTCD Table 4C-1) for main and minor streets were 420 vph and 105 vph. For the other one, the minor street had one lane. Under this condition, the minor street volume was the maximum volume of adjusted right-turn volume, and through and left-turn traffic. The eight-hour vehicular volumes for main and minor streets were 420 vph and 140 vph. By using the adjustment methodology, Warrant 1 Condition A was not justified.

Since Condition A was not satisfactory, Condition B and the combination of Condition A and B were calculated. As a result, neither condition was warranted and therefore, this intersection was not justified for Warrant 1.

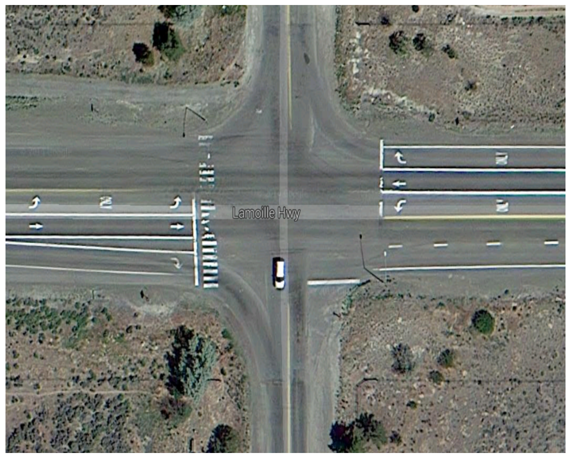


Figure 5.1 Intersection Picture at Lamoille Highway and Spring Creek Parkway

Table 5.1 Analysis Vehicular Volulme Before Adjustment (Lamoille Highway and Spring Creek Parkway)

Start Time	6:00	7:00	8:00	14:00	15:00	16:00	17:00	18:00	Requirements
Major Volume Total	454	556	480	572	582	880	1111	699	420
Minor Volume Total	290	280	268	181	178	186	202	152	105/140
Minor Through and Left Turns	38	37	62	57	56	59	63	38	N/A
Minor Right Turns	252	243	206	124	122	127	139	114	N/A

Table 5.2 Analysis Vehicular Volulme After Adjustment (Lamoille Highway and Spring Creek Parkway)

Start Time	6:00	7:00	8:00	14:00	15:00	16:00	17:00	18:00	Requirements
Equivalent factor*	0.36	0.30	0.36	0.30	0.30	0.27	0.25	0.30	N/A
Minor Through and Left Turns	38	37	62	57	56	59	63	38	N/A
Equivalent Minor Right Turns	91	73	75	38	37	35	35	35	N/A
Critical Volume (one lane)	91	73	75	57	56	59	63	38	105
Minor Volume Total (two lane)	129	110	137	95	93	94	98	73	140

*The volume ratio was not available, so assume the factor as 1:1.

5.2 US 395 and Airport Road

Another intersection is located at US 395 and Airport Road as shown in Figure 5.2. US 395 is the main street (northwest and southeast bounds) with two lanes and Airport Road is the minor street with the shared lane. The flared shared lane at the intersection was not considered for the proposed method. For the eight-hour warrant verification, with all the turning traffic, warrant Condition B was justified and the vehicular data is shown in Table 5.3. After reduction, the total minor-street volume still met the minimum requirement for warrant Condition B and detailed results are illustrated in Table 5.4.

Figure 5.2 Intersection Picture US 395 and Airport Rd

Table 5.3 Analysis Vehicular Volumes Before Adjustment (US395 and Airport Rd)

Start Time	9:00	11:00	12:00	13:00	14:00	15:00	16:00	17:00	Requirements
Major Volume Total	1562	1627	1671	1695	1934	1983	2053	2115	630
Minor Volume Total	74	110	100	78	106	207	171	154	53
Minor Through and Left Turns	43	65	59	46	63	123	100	91	N/A
Minor Right Turns	31	45	41	32	43	84	71	63	N/A

Table 5.4 Analysis Vehicular Volume After Adjustment (US395 and Airport Rd)

Start Time	9:00	11:00	12:00	13:00	14:00	15:00	16:00	17:00	Requirements
Equivalent factor*	0.36	0.36	0.36	0.36	0.36	0.36	0.36	0.36	N/A
Minor Through and Left Turns	43	65	59	46	63	123	100	91	N/A
Reduced Minor Right Turns	12	17	15	12	16	31	26	23	N/A
Adjusted Minor Volume Total	55	82	74	58	79	154	126	114	53

*The volume ratio was not available, so assume the factor as 1:1.

5.3 Blue Diamond Road and South El Capitan Way

Blue Diamond Road and South El Capitan Way in Las Vegas is the study intersection and the geometry picture is shown in Figure 5.3. The Blue Diamond Road is the main street (east and west bounds), and South El Capitan Way is the minor street (south and north bounds). The busier approach of the minor street is the south one. The minor street lane configuration is Configuration 3 with an exclusive right-turn lane. The sketch of the study intersection is shown in Figure 5.4.

Erika Mar
Deleted:

Erika Mar
Deleted:

Figure 5.3 Intersection Picture at Blue Diamond Road and South El Capitan Way

5.3.1 Right-turn Adjustment

The volume directional distribution at the intersection of Blue Diamond Road and South El Capitan Way was not available, but there was TRINA data at Site 0031094 collected downstream of the intersection on Blue Diamond Road. The locations of these two sites are shown in Figure 5.5. Traffic volume and the volume ratio at Site 0031094 from Monday to Sunday are shown in Table 5.5. From these data, volume ratios in each peak hour could be derived and are listed in
Table 5.6.

From Table 5.5, we can see the westbound direction of Blue Diamond Road is busier and does not directly affect the northbound approach, which is the subject minor street approach to be analyzed. This explains why the high volume of right-turn traffic can easily enter the intersection. Table 5.7 provides detailed information about the reduction process. After the adjustment, warrants Condition A, Condition B and the combination of Condition A and B at the 56% level were all not warranted.

Erika Mar
Deleted:

Erika Mar
Deleted:

Erika Mar
Deleted:

Erika Mar
Deleted:

Warrant 2 (Four-hour vehicular volume) and Warrant 3 (Peak hour) were also checked and neither warrant was met. The warrant volume requirement is shown in Figure 5.6.

Erika Mar
Deleted:

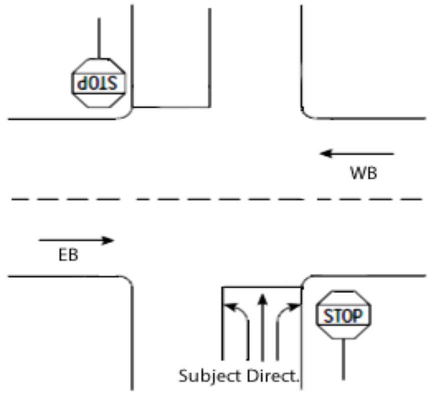


Figure 5.4 Intersection Sketch at Blue Diamond Road and South El Capitan Way

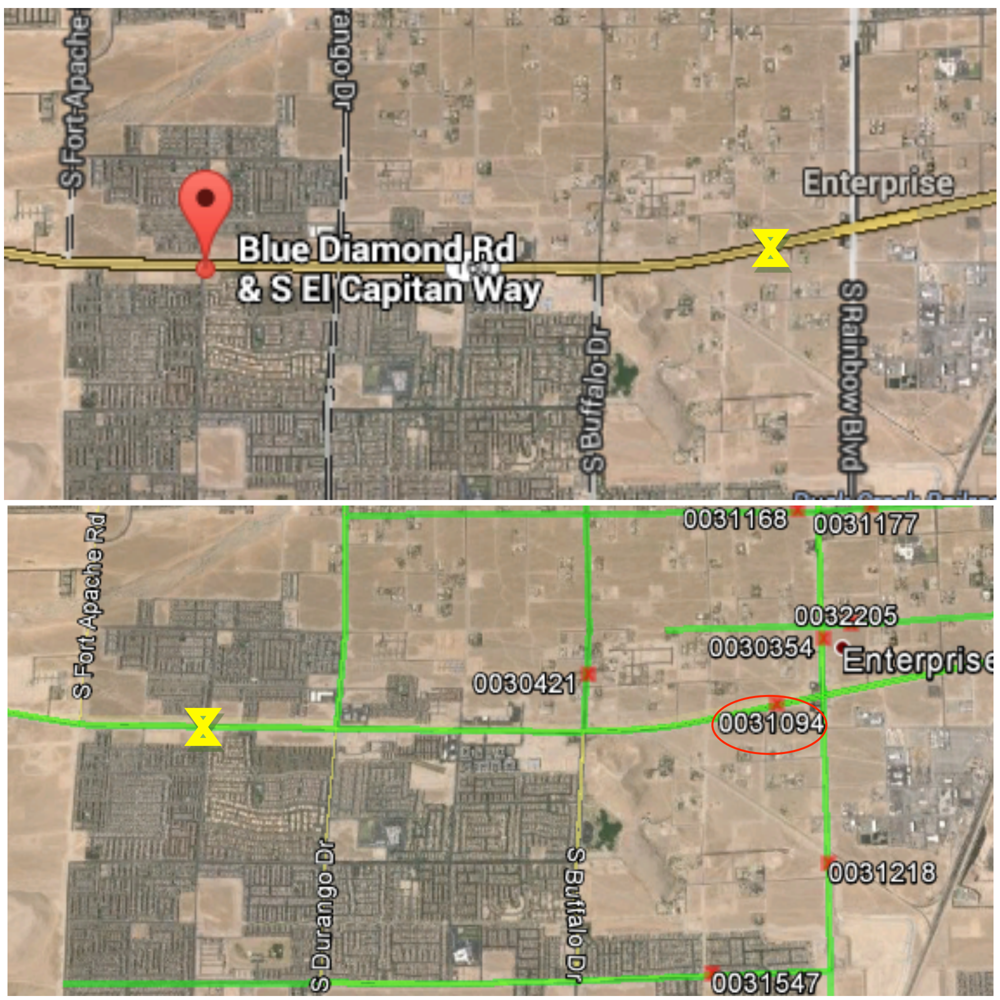


Figure 5.5 Relative Locations between Study Site and Site 0031094

Table 5.5 Traffic Volume and Volume Ratios at Site 0031094

	Direction	6:00	Ratio	7:00	Ratio	8:00	Ratio	9:00	Ratio	10:00	Ratio	1300	Ratio	14:00	Ratio	15:00	Ratio
Sun	W	359	1.5	550	1.8	760	1.7	990	1.7	1001	1.5	1137	1.2	1032	1.1	1058	1.0
	E	232		301		460		572		670		966		920		1010	
Mon	W	1171	1.9	1954	2.7	1557	2.0	1250	1.8	1069	1.5	1109	1.2	1099	1.0	1164	0.9
	E	626		735		797		690		720		933		1079		1248	
Sat	W	1209	1.9	1934	2.5	1578	2.0	1234	1.7	1065	1.4	1099	1.2	1097	1.1	1178	1.0
	E	647		770		772		719		742		921		1041		1193	
Wed	W	1159	1.6	1883	2.5	1585	2.0	1221	1.8	1059	1.4	1043	1.1	1029	1.0	1118	0.9
	E	704		747		795		691		738		919		1033		1216	
Thu	W	1271	2.5	1972	2.6	1982	2.7	1426	2	1106	1.6	1067	1.2	1089	1.0	1176	1.0
	E	500		755		734		723		693		923		1082		1234	
Fri	W	1043	1.9	1883	3	1642	2.2	1339	1.9	1234	1.7	1153	1.1	1307	1.2	1278	1.0
	E	536		623		732		720		715		1096		1130		1231	
Sat	W	516	1.1	858	1.6	1035	1.8	1139	1.7	1227	1.5	1265	1.3	1240	1.1	1257	1.1
	E	469		541		583		690		836		975		1118		1126	

Table 5.6 Suggested Volume Ratios at Each Peak Hour

Start Time	6:00	7:00	8:00	9:00	10:00	1:00	2:00	3:00
Volume Ratio	2:1	2:1	2:1	2:1	1:1	1:1	1:1	1:1

Table 5.7 Signal Warrant Analysis Based on Proposed Method

START TIME	6:00	7:00	8:00	9:00	10:00	13:00	14:00	15:00
MAJOR VOLUME	787	988	1060	946	983	1157	1192	1390
MINOR VOLUME	353	586	519	375	296	295	318	302
MINOR THROUGH & LEFT TURNS	56	128	101	60	47	47	51	48
MINOR RIGHT TURNS	297	458	418	315	249	248	267	254
Configuration 3								
Volume ratio	2:1	2:1	2:1	2:1	1:1	1:1	1:1	1:1
Equivalent factor	0.07	0	0	0	0.27	0.25	0.25	0.24
Reduced Right turns	21	0	0	0	67	62	67	61
Adjusted Minor volume	77	128	101	60	114	109	118	109
Warrant 1								
Condition A (70%)	0	0	0	0	0	0	0	0
Condition B (70%)	1	1	1	0	1	1	1	1
Condition A (56%) & Condition B (56%)	0	1	0	0	1	0	1	0
Warrant 2	0	0	0	0	0	0	0	0
Warrant 3	0	0	0	0	0	0	0	0

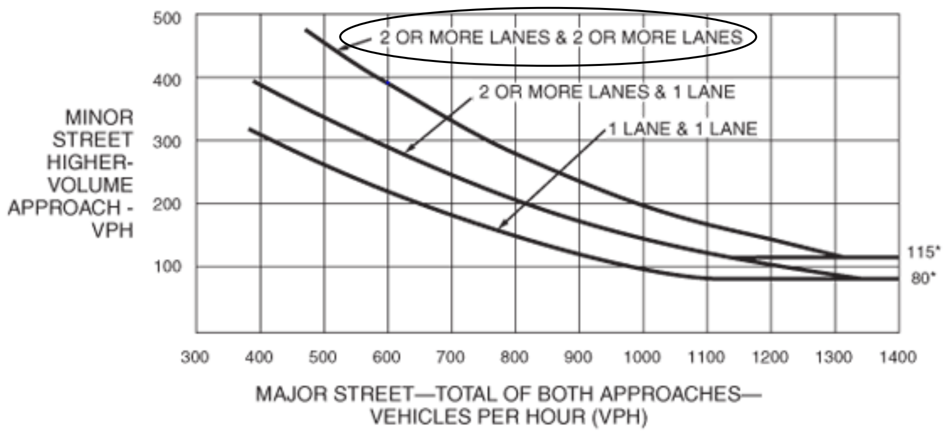
*In Warrant 1, 2 and 3, 0 represents that the volume is not warranted and 1 represents that the volume is warranted.

Warrant 1: Eight-Hour Vehicular Volume

Condition		Main Street	Minor Street
A	70%	420	140
	56%	336	112
B	70%	630	70
	56%	504	56

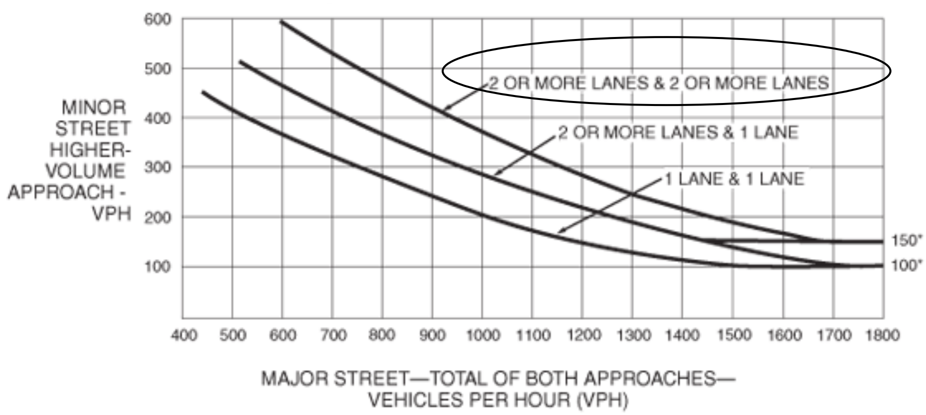
*The number of lanes for major street are 2 or more; the number of lanes for minor street are 2 or more

Warrant 2: Four-Hour Vehicular Volume



*Note: 115 vph applies as the lower threshold volume for a minor-street approach with two or more lanes and 80 vph applies as the lower threshold volume for a minor-street approach with one lane.

Warrant 3: Peak Hour



*Note: 150 vph applies as the lower threshold volume for a minor-street approach with two or more lanes and 100 vph applies as the lower threshold volume for a minor-street approach with one lane.

Figure 5.6 Signal Warrants 1, 2 and 3 Volume Requirements

5.3.2 HCM Delay

Synchro is used to calculate the delay at the study intersection for the eight peak hours. The purpose of this analysis is to determine if the intersection operates at an acceptable level of service. The LOS of each movement at peak hour periods are shown in Table 5.8. From the delay, we can see minor-street through traffic may have difficulty in crossing the intersection due to the high volume on the main street. The right-turn traffic can easily enter the intersection, even though the right-turn volume is very high at peak hours. Overall, the intersection operates at acceptable levels. The worst LOS is E, but the majority are C or better.

Erika Mar
Deleted:

Table 5.8 Intersection LOS at Blue Diamond Road and South El Capitan Way

LOS	Minor Street		
	Left turn	Through	Right turn
6:00	B	C	B
7:00	C	D	C
8:00	C	D	B
9:00	C	C	B
10:00	C	C	B
13:00	C	D	B
14:00	C	D	B
15:00	D	E	B

5.3.3 Other Reduction Methods

The Pagones Theorem and NCHRP 475 method were conducted for this case and further were compared with the proposed method.

For Pagones Theorem, the lane configuration was with an exclusive right-turn lane. Based on Table 2.1, the minor street adjustment factor was 0.75 and the mainline congestion factors were extracted from Table 2.2, shown in Table 5.9. There were two lanes in each direction of the main street.

From the reduction procedure, this intersection signal was warranted based on Warrant 1 Condition B, and Condition A and B.

Table 5.9 Signal Warrant Analysis Based on Pagones Theorem

START TIME	6:00	7:00	8:00	9:00	10:00	1:00	2:00	3:00
MAJOR VOLUME	787	988	1060	946	983	1157	1192	1390
MINOR VOLUME	353	586	519	375	296	295	318	302
MINOR THROUGH & LEFT TURNS	56	128	101	60	47	47	51	48
MINOR RIGHT TURNS	297	458	418	315	249	248	267	254
CASE 3								
MALINE VOLUME PER LANE	197	247	265	237	246	289	298	348
MAINLINE CONGESTION FACTOR	0	0	0	0	0	0	0	0
MINOR ADJUSTMENT FACTOR	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
REDUCTION FACTOR	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25
REDUCED RIGHT TURN	74	115	105	79	62	62	67	64
ADJUSTED MINOR VOLUME	130	243	206	139	109	109	118	112
Warrant 1								
Condition A (70%)	0	1	1	0	0	0	0	0
Condition B (70%)	1	1	1	1	1	1	1	1
Condition A (56%) & Condition B (56%)	1	1	1	1	1	1	1	1
Warrant 2	0	1	1	0	0	0	0	0
Warrant 3	0	0	0	0	0	0	0	0

*In Warrant 1, 2 and 3, 0 represents that the volume is not warranted and 1 represents that the volume is warranted.

Using the NCHRP 475 comparison method, the volume proportion of the main street was unavailable so 10% of the major volume was assumed as right turns and no left turn was assumed. It was reasonable and conservative since most people drove through Las Vegas. After calculation, it was found that the assumption was not important because the right-turn reduction was high enough to cover the total right-turn volume. As a result, the signal was not warranted.

For this specific case, the signal was not warranted from the proposed method or the NCHRP 475 method. However, the signal was warranted using the Pagones Theorem. From the operation perspective, this intersection was at the edge of installing a signal.

In the NCHRP 475 method, from conflicting major-road volume, minor-road right-turn volume reduction was calculated. From Table 5.10, the right-turn reduction volume was too high. It assumed all right turns operated freely, but in reality, the right lane may be blocked by through traffic.

Table 5.10 Signal Warrant Analysis Based on NCHRP 475 Method

START TIME	6:00	7:00	8:00	9:00	10:00	13:00	14:00	15:00
MAJOR VOLUME	787	988	1060	946	983	1157	1192	1390
MINOR VOLUME	353	586	519	375	296	295	318	302
MINOR THROUGH & LEFT TURNS	56	128	101	60	47	47	51	48
MINOR RIGHT TURNS	297	458	418	315	249	248	267	254
CONFLICTING MAJOR-ROAD	131	165	177	158	246	289	298	348
RIGHT-TURN REDUCTION	821	801	794	806	753	727	721	692
REDUCED RIGHT TURN	297	458	418	315	249	248	267	254
ADJUSTED MINOR VOLUME	56	128	101	60	47	47	51	48
Warrant 1								
Condition A (70%)	0	1	1	0	0	0	0	0
Condition B (70%)	0	1	1	0	0	0	0	0
Condition A (56%) & Condition B (56%)	0	1	0	0	0	0	0	0
Warrant 2	0	0	0	0	0	0	0	0
Warrant 3	0	0	0	0	0	0	0	0

*In Warrant 1, 2 and 3, 0 represents that the volume is not warranted and 1 represents that the volume is warranted.

Chapter 6 Pedestrian Impact on Right-turn Vehicle Adjustment

It is commonly seen that pedestrians crossing the main street block the minor-street right-turn vehicles. Such occurrence creates large main-street gaps, which can significantly affect the minor street capacity. Minor-street though vehicles often take this opportunity and cross with the pedestrian. In this chapter, a Monte Carlo model was built to simulate the real operation of TWSC intersections. This model can calculate the minor-street though vehicles' capacity enhancement due to the pedestrian crossing, which would increase minor-street right turn vehicle equivalent factors and account for pedestrian blocking impact on right turns.

6.1 Introduction

The current edition of the HCM [15] uses gap acceptance theory for estimating the capacity and delay for various traffic movements at TWSC intersections. The HCM capacity estimation method assumes that a traffic movement with a lower priority yields to the right-of-way of higher order movements. Accordingly, each traffic movement at a TWSC intersection is assigned a specific ranking in a priority hierarchy. The through and right-turn traffic on the major street is considered as a Rank 1 movement over other traffic streams and has the highest priority; that is, all other movements with lower ranks have to yield to them. Even pedestrians crossing the main street must yield to the main street traffic. Given this assumption, the capacity of through and right-turn traffic lanes on the major street at a TWSC intersection is estimated as the saturation flow rate, and experienced delay is usually considered to be zero. If vehicles obey these rules strictly, the estimation of the minor-street capacity is relatively exact in this context.

However, in reality, this assumption is not so reasonable considering the fact that in many countries, motorists are legally required to yield to pedestrians under most circumstances. So in practice, drivers often yield to pedestrians crossing the main street. The pedestrian crossing the main street has an impedance effect on the main street traffic, which will decrease the capacity and increase the delay. On the contrary, it may increase the minor street capacity. Generally speaking, when there is one or more pedestrians crossing the main street, minor-street traffic can use the gap created by pedestrians to enter the intersection. This situation is especially obvious when main street traffic is very high and minor-street drivers have difficult to find an adequate gap.

The underlying reason for such capacity change on the minor street is due to major-street drivers' yielding behavior. Turner et al. [17] conducted a national study about motorists yielding to pedestrians at unsignalized intersections. The research team collected extensive data at 42 study sites in different regions of the country and found the actual motorist yielding rate ranges from 17% to 99%. This research was also documented by HCM. The wide ranges of the yielding rate are influenced by street and traffic characteristics, e.g., the posted speed limit, the number of lanes, traffic volumes and so on.

Bonneson [18,19] proposed two models to account for delays to major-street through drivers. The first model was to predict the delay caused by vehicles turning right from the outside-through traffic lane on the major street. The second model was to predict the delay that was incurred when major-street left-turn demand exceeds the available storage area and blocks the adjacent through lane. A recent study conducted by Wei et al. [20] developed an analytical model to estimate the expected vehicular delay as a function of the traffic volume, pedestrian volume, and vehicle-yielding rate for the minor-street through movement at TWSC intersections. Yang et al. [21] further enhanced the model by considering pedestrians arriving on both sides of the street and results from the model were tested using field-measured data. Their research results suggested that the capacity of the major-street through movement would decrease with an increase of the pedestrian arrival rate. It was also found that, with the same pedestrian arrival rate, the capacity would decrease as the motorist-yielding rate becomes higher, and the magnitude of the capacity reduction would increase with an increase of the pedestrian arrival rate.

Li and Deng [22] studied TWSC intersections' capacity affected by upstream-signalized intersections including interactions in the gap acceptance, platoon dispersion, and signalized intersection control systems. Better capacity estimations were achieved compared to the HCM method.

Previous researchers mainly focused on major-street vehicular delay models based on turning vehicles and pedestrians, but they ignored the fact that the minor-street capacity may also change due to these factors. The major objective of this chapter is to develop a simulation model using Monte Carlo sampling to calculate the minor-street capacity with the impact of pedestrian crossing and using this capacity to estimate the right-turn equivalent factors under pedestrian impact.

6.2 Theoretical Background

6.2.1 Headway distributions

Researchers [23] have demonstrated that the headway process will be a Poisson process at some reasonably large distance from the entry point. Cowan [24] mentioned that for vehicular headway, significant errors would be introduced if the headway model used does not exclude very small headways and vehicles arriving as a platoon should be specifically addressed. Cowan's M3 model addressed these issues and produces a more realistic model than exponential distribution. The equation for its probability density function is as follows:

$$f(t) = \begin{cases} 0 & t < \Delta \\ \theta\lambda e^{-\lambda(t-\Delta)} & t \geq \Delta \end{cases} \tag{6.1}$$

where θ is proportion of free vehicles; Δ is minimum headway, and λ is model parameter in vehicle per seconds determined on the basis of Equation 6.2.

$$\lambda = \frac{\alpha V}{3600 - \Delta V} \tag{6.2}$$

This model states that a proportion $1-\alpha$ of vehicles are tracking their predecessor at headway Δ while a proportion α are traveling freely at some headway greater than Δ.

The pedestrian arrival model is assumed as the Poisson model, which is reasonable since this is no minimum safety gap for pedestrian flow.

6.2.2 Gap acceptance

At unsignalized intersections, minor-street drivers alone must decide when it is safe to enter the intersection. The driver looks for a safe gap in the traffic to enter the intersection [25]. This process is called gap acceptance. Gaps are measured in time and equal to headways. The minimum gap that all drivers in the minor stream are assumed to accept at all similar locations is the critical gap, t_c. In a very long gap, it is further assumed that several drivers will be able to enter the intersection from a minor road. Usually, vehicles following the vehicle ahead enter the intersection in the long gaps at headways; this is referred to as the follow-up time, t_f. Based on these two terms, it is easy to calculate how many minor stream vehicles can take one gap. Further, the capacity for the minor stream can be integrated over the whole range of major stream gaps:

$$C = V\int_0^\infty f(t)g(t)dt \tag{6.3}$$

where C = the capacity in the minor stream, i.e. maximum traffic volume departing from the stop line; V = major stream volume; $f(t)$ = density function for the distribution of gaps in the major stream, and $g(t)$ = the number of minor stream vehicles which can enter into a major stream gap of size, t. The continuous linear function for $g(t)$ has first been used by Siegloch [26] as follows:

$$g(t) = \begin{cases} 0 & t < t_0 \\ \dfrac{t-t_0}{t_f} & t \geq t_0 \end{cases} \tag{6.4}$$

where,

$$t_0 = t_c - \frac{t_f}{2}$$

If the distribution of gaps in the major stream is known, it is easy to calculate the minor stream capacity. However, when pedestrians are considered, the gap distribution of the mainstream has changed and it is not easy to derive theoretically. A Monte Carlo method is used here to estimate the gaps and further gap acceptance is employed to calculate the capacity of the minor stream.

6.3 Model Assumptions

Certain assumptions are made regarding the yielding behavior, traffic flow, and pedestrian crossing behavior so the problem can be simulated.

At each location, a certain yielding rate is assumed for each vehicle. If vehicles decide to yield to a pedestrian, they need to wait at the yielding line until pedestrians exit the crosswalk. By doing so, major vehicles will not block the intersection. In reality, vehicles tend to leave early, especially when a pedestrian has passed their lanes. This effect is considered by randomizing pedestrian crossing time. Pedestrian crossing time is assumed as truncated Gaussian distribution. If one pedestrian arrives at the intersection while another pedestrian is crossing the crosswalk, he or she will cross the intersection immediately. The total crossing time will be extended by one pedestrian crossing time. This is reasonable because vehicles on the major street are stopped and the later pedestrian can easily catch the gap.

Major-street vehicles' arrival pattern is assumed to follow Poisson distribution, shifted Poisson distribution or Cowan's M3. Major-street right-turn vehicles' characteristics are not considered and treated as through vehicles. If there is more than one lane on the major road, all the vehicles are treated as if they are using one lane. The capacity calculated on the minor street is assumed as through vehicle capacity.

When pedestrians cross the street, major-street vehicles may form a queue. After pedestrians exit the crosswalk, the queue begins to discharge and will not be interrupted by other incoming pedestrians. This assumption simplifies the simulation and may reduce the enhancing effect created by pedestrians.

6.4 Probabilistic Model

As shown in Equation 6.3, the minor-street capacity depends on the net gap distribution, which is related to many practical factors, such as the distribution of headway, crossing pedestrians, and left-turn vehicles on the major street. In order to accurately model the complex interactions within these parameters, a Bayesian network model is applied, which can incorporate all uncertainties in parameters and capture their conditional relationships in a graphical manner. For an introduction to the Bayesian network, please refer to [27].

The Bayesian network model, built by a causal relationship to estimate the distribution of net gap G for minor-street vehicles, is shown in Fig. 6.1, in which each circle represents a random variable and each arrow represents the conditional dependency. The meaning of each random variable and its associated probability distribution are described below, and the way to construct their conditional dependencies is introduced thereafter.

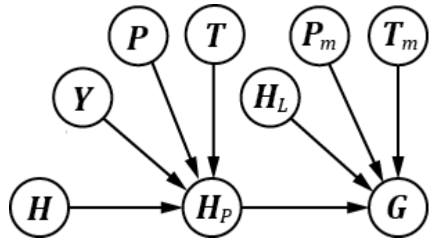


Figure 6.1 Probabilistic Model for Minor-street Capacity Estimation

The random variables P and P_m represent minor-street and major-street pedestrian headways, respectively. Because there is no minimum safety gap for pedestrian flow, they are both modeled by the exponential distribution but with different parameters. Minor-street pedestrians may block major-street vehicles, thus increase the minor-street capacity, but major-street pedestrians have the opposite effect on the minor-street capacity.

The crossing times T and T_m involving pedestrians are also important parameters influencing the net gap distribution. According to field observations and research studies, pedestrians have a wide range of needs and abilities [28]. An Australian Institute of Transportation study [29] of intersection signalized crossing sites indicates that the average pedestrians crossing speed is 5.35 ft/s. The MUTCD standard specifies a normal walking speed as 4 ft/s for calculating pedestrian clearance intervals for traffic signals. Another research [30] states that the majority of pedestrians walk at a speed that is slower and that 15% of pedestrians walk at speeds less than 3.5 ft/s. Considering the randomness of pedestrian crossing speed, truncated Gaussian distributions are assumed here.

The motorist-yielding rate Y is a dominating parameter when considering the influence of pedestrians on major-street vehicles. If there is no yielding as idealized in HCM, the problem is highly simplified, but motorists in the United States are legally required to yield to pedestrians, under most circumstances, in both marked and unmarked crosswalks. In reality, a range of factors influences the yielding behavior. These include roadway geometry, travel speed, pedestrian crossing treatments, local culture, and law enforcement practices. Specific yielding rate or range should be provided based on field measurement or estimate.

The random variables H and H_L denote the major-street through/right-turn and left-turn vehicles arrival headways, respectively. The right-turn volume is combined with the through volume, but the left-turn vehicles are treated separately since they have different effects on the net gap distribution. If multiple lanes exist in the major street, they are counted together. Three kinds of distributions including Poisson, shifted Poisson and Cowan's M3 distributions are formulated for the major-street vehicles arrival pattern, and one can choose any of them in minor-street capacity estimation.

The major-street through vehicles are interrupted by the minor-street pedestrians, resulting in a new random variable H_P, which can be interpreted as the major-street through-vehicle departure time. Although major-street left-turn vehicles also interact with pedestrians – minor-street pedestrians create gaps for these vehicles while the major-street pedestrians conflict with the flow of left-turn vehicles – their conditional relations are not explicitly modeled in our model because they have negligible influence on the net gap distribution.

In the Bayesian network, the conditional distributions of random variables H_P and G need to be specified. Due to their complicated interactions, closed forms of distributions are not available for these random variables. Instead, a simulation-based approach is applied to provide these distributions by samples.

First, we consider the conditional distribution of H_P given random variables P, T, Y and H. In order to simulate the interactive behaviors of vehicles and pedestrians, action rules between these random variables need to be specified. The two core aspects are the minor-street pedestrians' crossing behavior and the major-street vehicles' yielding decision.

The minor-street pedestrians' crossing behavior highly depends on their critical headway, which is defined as the maximum duration in seconds below which a pedestrian will not attempt to begin to cross the street. The critical headway is computed as

$$t_{pc} = \frac{L}{S_p} + t_{ps} \tag{6.5}$$

where t_{pc} is the critical headway for a single pedestrian, S_p is the average pedestrian walking speed, L is the crosswalk length, and t_{ps} is the sum of pedestrian start-up time and end clearance time. Note that the pedestrian crossing time T is the actual time that a pedestrian uses to cross the intersection, and it is not necessarily equal to the critical headway.

If the available headway on the major street is greater than the critical headway, it is assumed that the pedestrian will cross; otherwise, they will not unless major-street vehicles yield. In some circumstances, there may be series of pedestrians coming subsequently. Then, once the major-street vehicles decide to yield, they need to keep waiting until all pedestrians exit the crosswalk, and the total crossing time will be extended accordingly. It is reasonable because vehicles on the major street are stopped and the later pedestrian can easily catch the gap.

The yielding behavior of major-street vehicles is totally determined by their yielding rate Y. When vehicles arrive as a platoon, if the heading vehicle yields, all following vehicles must yield as a consequence; if the heading vehicle does not yield, the yielding decision of the following vehicle is made according to the yielding rate. Besides, when pedestrians are crossing, a queue may be formed on the major street, which needs time to discharge, especially in the case of a long queue. To consider this effect, a deterministic lost time for discharging the queue is applied, and it will be deducted from the first several gaps. For the first queuing vehicle, the lost time is assumed to be 2 s, and following queuing vehicles' lost time is 1 s, resulting in total lost time $2+1\times(n-1)$ s, where n is the number of vehicles in the queue. It is further assumed that other incoming pedestrians will not interrupt vehicles on the queue once it begins to discharge.

In reality, vehicle-yielding time is the actual time that will be used for minor-street vehicles, which is estimated by the pedestrian crossing time in the model. However, vehicles tend to leave early when pedestrians have passed their lanes, especially when the major-street traffic volume is high. To alleviate this effect, pedestrian start-up time and end clearance time is not added to the pedestrian crossing time. The assumed truncated Gaussian distribution for crossing time can also compensate this effect.

Then, the conditional distribution of G given H_P, H_L, P_m and T_m, which is the core distribution in our problem, needs to be determined. Even though minor-street pedestrian crossing enlarges the net gaps, other factors may reduce this effect. The major-street left-turn vehicles are the Rank 2 movement and have higher priority than minor-street through movements. If a gap exists in the major street, it will be filled first by these left-turn vehicles. In addition, as mentioned before, pedestrians crossing the minor street will block minor-street vehicles as well. Therefore, these gaps conquered by the left-turn vehicles and major-street pedestrians should be deducted in calculating the net gap.

Based on the above rules, six steps are summarized for computing both conditional distributions using sampling:

(1) Generate the arrival time of major-street through/right-turn vehicles, left-turn vehicles, minor-street pedestrians (cross the major street), and major-street pedestrians (cross the minor street);

(2) Calculate major-street vehicle departure time considering the impact of pedestrians. According to vehicles and pedestrians arrival time, two conditions are considered:

(a) Vehicles arrive without present pedestrians, so their departure time is equal to the arrival time;

(b) Vehicles arrive within pedestrian critical headway. If vehicles choose to yield, pedestrians can cross the intersection, vehicles departure time is equal to the arrival time plus the crossing time. If vehicles decide not to yield, their departure time is equal to the arrival time or plus the possible decision time;

(3) Eliminate possible queue influence;

(4) Determine all possible major-street pedestrians blocking duration;

(5) Calculate the conquered gaps by major-street left-turn vehicles;

(6) Compute the distribution of net gaps, and obtain the minor-street capacity based on Equation 6.3.

The proposed Bayesian network model can efficiently process the measured information. For example, if one can provide a point or an interval measurement on any one or more of the random variables, a narrower distribution of net gap G will be obtained, thus resulting in a more accurate estimation of the minor-street capacity. If only a few measurements are available, the model still gives a reasonable estimation by averaging all considered uncertainties.

The calculation flow chart is provided in Figure 6.2, which gives a clear view.

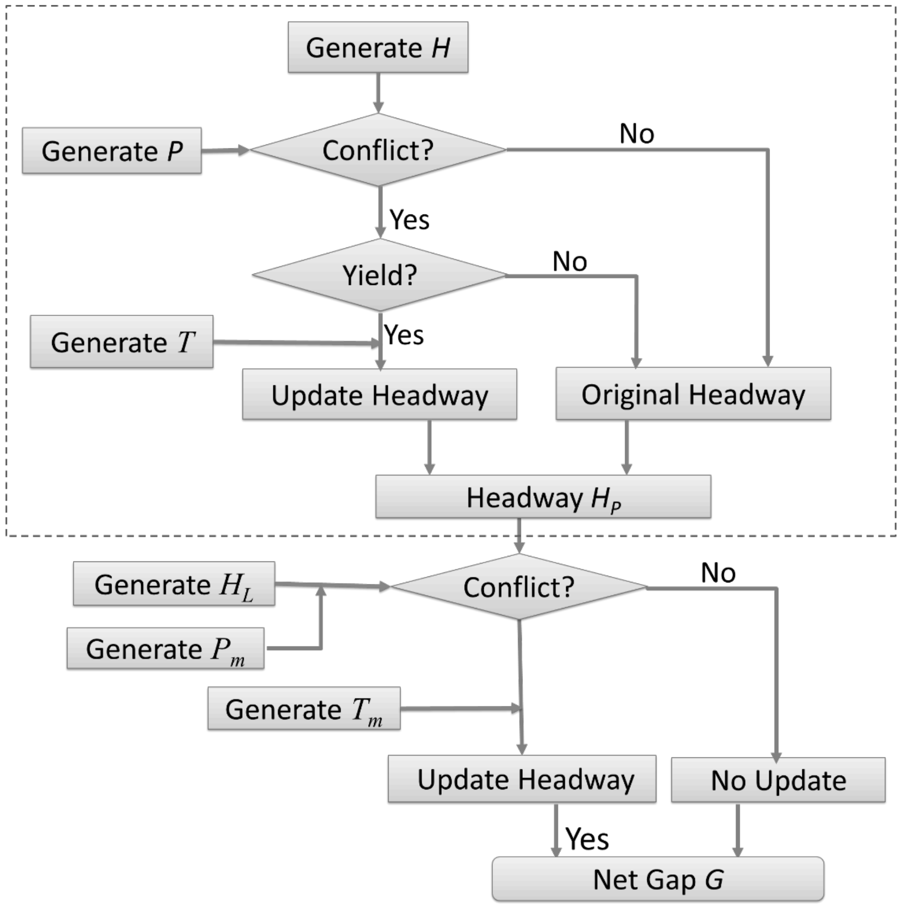


Figure 6.2 Calculation Flow Chart

6.5 Case Study

In order to test the validation of our established model, data collection was conducted at a TWSC intersection near the campus of University of Nevada, Reno. The intersection picture and sketch are shown in Figure 6.3. The major street is north-south bound and has one through lane with a short left-turn bay in each direction. This intersection has a marked crosswalk and pedestrian actuated lighting barrier. This intersection is the access to the campus from the residential area and two small campus parking lots are located nearby. Every day, many students and university personnel cross this intersection from both directions. The upstream-signalized intersections are far away and the platoon is dispersed when vehicles arrive here. The minimum headway between vehicles may be nearly zero, since the departure time of the two directions is recorded together. Therefore, major-street vehicle gaps follow exponential distribution.

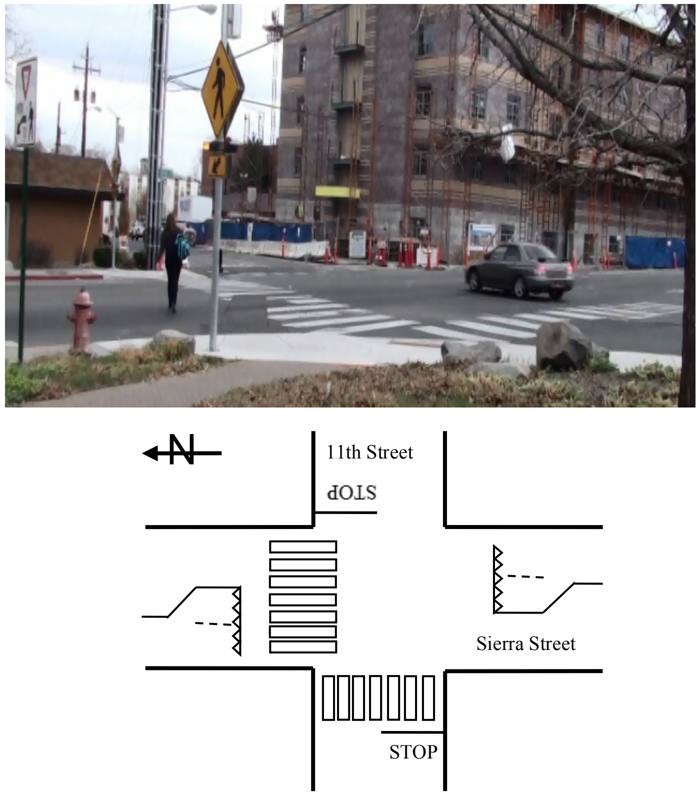


Figure 6.3 Intersection Picture and Sketch

A video camera was set up in the field for recording the intersection operation. The recordings were scheduled on March 9, 2015 from 10:55 a.m. to 11:55 a.m. and March 11, 2015 from 8:00 a.m. to 9:00 a.m. Necessary information was extracted from the recorded videos:

(1) Record major-street through and right-turn vehicles departure time. The reference line is the yield line;

(2) Count minor-street pedestrians;

(3) Record time of main-street pedestrians entering and exiting;

(4) Record time of main-street left-turn vehicles entering and exiting;

(5) Deduct the main-street pedestrians and left-turn crossing intervals from major-street departure time;

(6) Calculate the gaps and capacity based on gap acceptance.

The motorist-yielding rate was calculated for each individual pedestrian attempting to cross the street. If there was no vehicle arriving at the site when a pedestrian crossed the street, this crossing event was not included in the calculation, as there were no motorists who should have yielded. If there was a vehicle arriving at the site when a pedestrian crossed the street, this crossing event was included. If the driver decelerated or stopped to yield, the yield rate was one; if the driver did not choose to yield, the yield rate was zero. The motorist-yielding rate was calculated as the ratio of the number of motorists who decelerated or stopped to yield to a pedestrian to the total number of motorists who should have decelerated or stopped. Fifty-seven vehicles were recorded in total. Among them, 49 drivers decelerated to yield to a pedestrian. The estimated yielding rate was around 0.90. Based on observation, this intersection had a very good yielding behavior. During low volume periods, almost all drivers chose to yield. However, during peak hours, some drivers chose not to yield to avoid delay.

Pedestrian crossing time was also measured. In addition, the occurrence of drivers leaving before pedestrians exited the intersection was observed. However, if a minor-street vehicle waited for the pedestrian to cross, early departure was not probable. Sixty pedestrians were recorded for obtaining an average crossing time. The average crossing time was 9.4 s and standard derivation was 1.6 s.

Table 6.1 lists input information and calculation results. The capacity estimated from the model was 36,000 samples, which were repeatedly run 10 times. Figure 6.4 shows 50 vehicles' arrival and departure time. If there is no pedestrian influence, the arrival time equals the departure time and two dots overlap. Otherwise, departure time is greater than arrival time. The time difference can be seen as the mainstream vehicular delay.

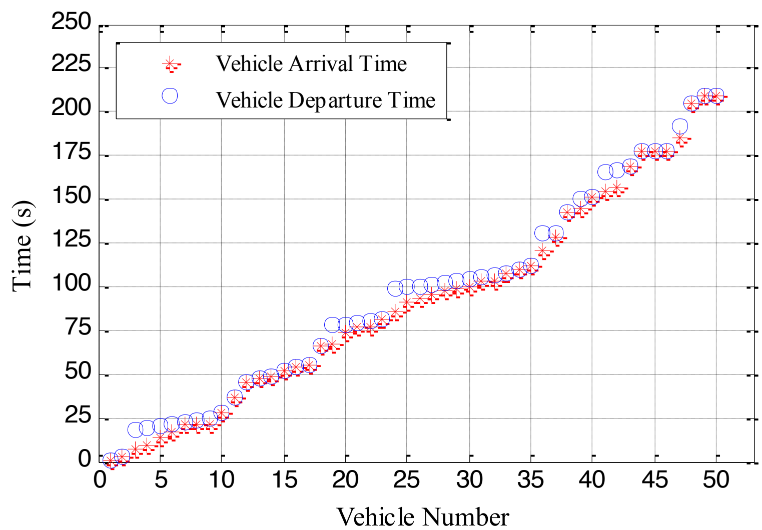


Figure 6.4 Vehicle Arrival and Departure Time

Table 6.1 Comparisons between Field Fata and Model Estimation

Motorist Yielding Rate		0.90	
Pedestrian Crossing Time and Critical headway (s)			
Mean	9.4	SE	1.6
Count		Record 1	Record 2
Major-street Trough/Right Turn (vph)		532	700
Major-street Left Turn (vph)		30	28
Major-street Pedestrian Volume (pph)		14	16
Minor-street Pedestrian Volume (pph)		98	154
		Record 1	Record 2
Capacity from field (vph)		428	352
Capacity from model (vph)		424	367
Deviation		4	15

Deviation in percentage	4/428=0.93%	15/352=4.26%

From Table 6.1, it was observed that model results were very close to field observation. It was not surprising because the model was built by the real operation of the intersection. Capacity without pedestrian impact was also calculated based on gap acceptance. The capacity for Video 1 was 412 vph and 332 vph for Video 2. The capacity had been increased by 4% and 14%. The pedestrian effect was obvious especially when the major vehicle volume and minor pedestrian volume were high.

6.6 Sensitivity Analysis

Main factors that influenced the minor-street capacity are major-street vehicular volume (without left-turn volume), major-street pedestrian volume, minor-street pedestrian volume, motorist-yielding rate, and crossing time. These parameters are analyzed further based on the proposed model. In each section, the relationship between one parameter and capacity is calculated with other factors fixed. Except for the section of gap distribution, exponential distribution is used for all other analysis.

6.6.1 Major Street Vehicular Volume

Major street vehicular volume and gap distribution determine the gaps on the major road. When the volume is low, the chance of large gaps is high. So the chance of a pedestrian crossing the intersection with a large gap is high. Therefore, there is little chance that pedestrians interrupt the vehicular flow in order to cross the street. The pedestrian blocking effect is not obvious. Similarly, when the volume is relatively high, the gap in the major road tends to be small. Pedestrians must interrupt the traffic flow to pass through the street and minor-street vehicles can use this gap. It is especially hard for minor-street traffic to cross the street if the major-street traffic volume is high, causing long delays. Minor-street pedestrians can be seen as timesavers for them, decreasing the delay extensively. The major-street vehicular volume mentioned here is the through and right-turn volume, except left-turn traffic. The major-street left-turn traffic is a second rank movement, which also needs gaps to make a turn.

Figure 6.5 shows how the capacity will change with different major-street vehicular volume. In order to get a smooth curve, the result was the average of 20 runs.

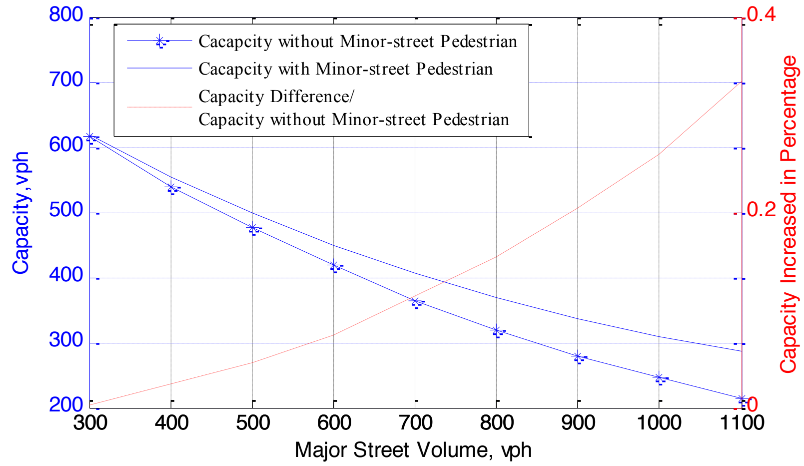


Figure 6.5 Comparison of Capacity with and without Pedestrians
Note: MotoristYieldRate = 0.8; MinorPedVol = 150 pph; MajorPedVol = 0; LeftturnVol = 0; PedCrossingMajorTime = 9 s and PedCrossingMinorTime = 6 s

6.6.2 Gap Distribution

Different gap distributions influence minor-street capacity greatly. There are three common gap distributions in traffic flow theory, namely exponential distribution, shifted exponential distribution and Cowan M3. Table 6.2 lists six situations' results, namely exponential, shifted exponential ($t_m = 2$), Cowan M3 ($t_m = 2$, $u = 0.8$), Cowan M3 ($t_m = 2$, $u = 0.5$), Cowan M3 ($t_m = 0$, $u = 0.8$) and Cowan M3 ($t_m = 0$, $u = 0.5$). The capacity enhanced effect is great if the original capacity is low. It is easy to understand because pedestrian crossing increases useful gaps for minor-street traffic.

6.6.3 Motorist-Yielding Rate

As mention before, the motorist-yielding rate is influenced by a couple of factors, such as local culture, speed limit, and roadway geometry. At one specific location, the motorist-yielding rate also varies due to some factors. Drivers do not tend to yield to pedestrians if the vehicular volume is high during peak hours. However, if pedestrians aggressively enter the crosswalk, most drivers will slow and stop their vehicles. There are lighted posts at locations with high pedestrian volume. The lighting helps drivers notice pedestrians, making the yielding rate high. Figure 6.6 depicts that capacity increases with motorist-yielding rates. Similar increasing rates are obtained with different major vehicle volumes.

Table 6.2 Capacity Changes Considering Different Gap Distributions

Major Volume	Exponential			Shifted Exponential (t_m=2)		
	Capacity without Ped (vph)	Capacity Ped (vph)	Difference in Percentage (%)	Capacity without Ped (vph)	Capacity with Ped (vph)	Difference in Percentage (%)
300	617	618	0.2	580	590	1.7
400	540	553	2.4	484	508	5.0
500	476	498	4.6	392	438	11.7
600	418	449	7.4	308	374	21.4
700	365	407	11.5	232	321	38.4
800	320	369	15.5	167	278	66.5
900	280	337	20.4	109	241	121.1
1000	246	310	26.0	66	213	222.7
1100	215	287	33.5	34	192	464.7
Major Volume	Cowan M3 (t_m =2, u =0.8)			Cowan M3 (t_m =2, u =0.5)		
	Capacity without Ped (vph)	Capacity with Ped (vph)	Difference in Percentage (%)	Capacity without Ped (vph)	Capacity with Ped (vph)	Difference in Percentage (%)
300	610	618	1.3	662	662	0.0
400	521	537	3.1	584	588	0.7
500	436	468	7.3	509	523	2.7
600	354	404	14.1	434	461	6.3
700	280	349	24.6	365	405	10.9
800	209	301	44.0	296	354	19.5
900	148	263	77.7	231	309	33.9
1000	97	229	136.1	173	268	54.6
1100	56	204	264.3	118	235	98.8
Major Volume	Cowan M3 (t_m =0, u=0.8)			Cowan M3 (t_m =0, u =0.5)		
	Capacity without Ped (vph)	Capacity with Ped (vph)	Difference in Percentage (%)	Capacity without Ped (vph)	Capacity with Ped (vph)	Difference in Percentage (%)
300	665	666	0.3	743	741	0.1
400	600	611	1.9	697	698	0.2
500	539	559	3.7	659	661	0.4
600	489	520	6.5	616	630	2.2
700	441	480	8.8	576	599	3.9
800	397	451	13.6	543	573	5.3
900	357	423	18.5	506	550	8.6
1000	319	401	25.5	476	526	10.5
1100	289	381	31.9	448	507	13.3

Note: MotoristYieldRate = 0.8; MinorPedVol = 150 pph; MajorPedVol = 0; LeftturnVol = 0; PedCrossingMajorTime = 9 s and PedCrossingMinorTime = 6 s

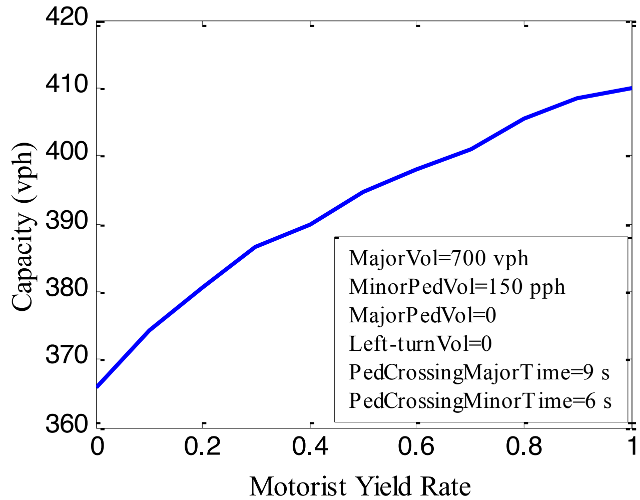


Figure 6.6 Relationship between Capacity and Motorist-Yielding Rate

6.6.4 Pedestrian Volume on Minor Street

Minor-street pedestrians are the focus of this research and they are the reason of the enlarging effect. Obviously, the more minor-street pedestrian volume, the greater impact on the major-street gap, which further influences the capacity. Figure 6.7 shows capacity increases with minor-street pedestrian volume. The relationship is approximately linear. The increasing rates become larger when major vehicular volumes increase.

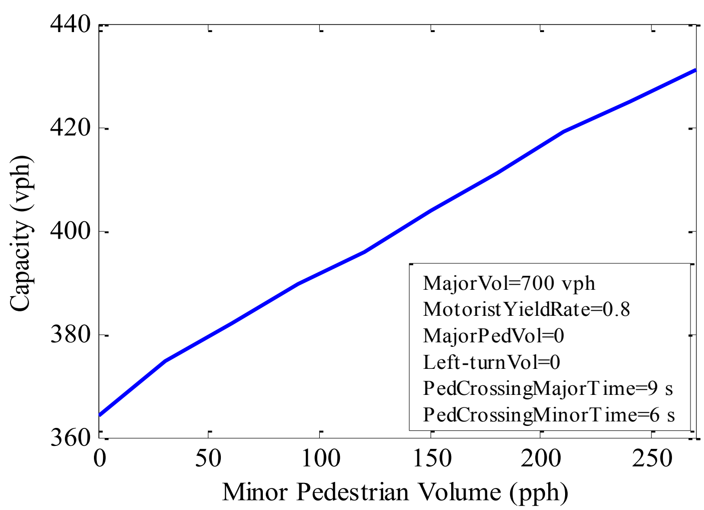


Figure 6.7 Relationship between Capacity and Motorist-Yielding Rate

6.6.5 Pedestrian Crossing Time on Major Street

While Pedestrians cross the major street, they block the major street traffic and make a gap for major-street left-turn and minor-street vehicles. It is obvious that the longer the crossing time, the larger the gap. Table 6.3 lists capacity under different crossing times and zero crossing time means a situation without the consideration of pedestrians. In the model, pedestrians are not homogenous and the pedestrian crossing time is considered as truncated Gaussian distribution.

The proposed capacity model adds all lane vehicles together. In this way, the crossing time increases with the number of lanes. Generally, six seconds crossing time stands for two lanes on the major street; nine seconds accounts for three lanes and 12 seconds means four lanes.

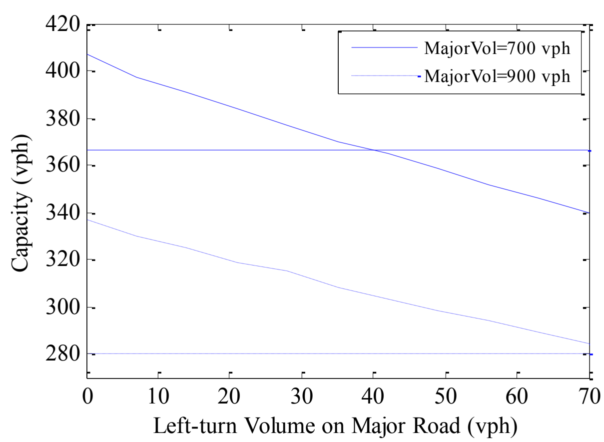
Table 6.3 Capacity Changes Considering Different Pedestrian Crossing Time

Major Volume	500		700		900	
Average Crossing Time	Capacity (vph)	Difference in Percentage (%)	Capacity (vph)	Difference in Percentage (%)	Capacity (vph)	Difference in Percentage (%)
0	476	0	365	0	280	0
6	477	0.2	374	2.6	308	9.9
9	498	4.6	407	11.8	337	20.4
12	519	9.0	441	21.5	385	37.3

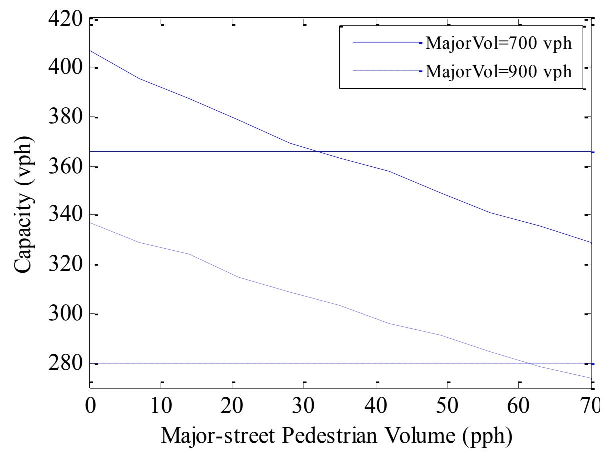
Note: MotoristYieldRate = 0.8; MinorPedVol = 150 pph; MajorPedVol = 0; LeftturnVol = 0; PedCrossingMajorTime = 9 s and PedCrossingMinorTime = 6 s

6.6.6 Left-turn Traffic on Major Street and Pedestrian on Major Street

Major-street left-turn traffic has a higher rank than minor-street traffic (expect minor-street right-turn, not considered here). Therefore, they will use the gap first. If the major-street left turn traffic is very high, the enlarging gaps will not benefit minor-street vehicles. Figure 6.8 (a) shows capacity change with left-turn volume. The two horizontal lines stand for the capacity without pedestrians. We can see that when the major street volume is relatively low, most of the enlarging gaps will be used by left-turns.



(a) Capacity and Major-street Left-turn Volume



(b) Capacity and Major-street Pedestrian Volume

Figure 6.8 Capacity Relationship with Major-street Left-turn Volume Major-street Pedestrian Volume

Note (a): MotoristYieldRate = 0.8; MinorPedVol = 150 pph; MajorPedVol = 0; PedCrossingMajorTime = 9 s, and PedCrossingMinorTime = 6 s
Note (b): MotoristYieldRate = 0.8; MinorPedVol = 150 pph; LeftturnVol = 0; PedCrossingMajorTime = 9 s, and PedCrossingMinorTime = 6 s

Pedestrians crossing the minor street are the first rank movement. They will enter the crosswalk after arriving at the intersections. Minor-street vehicles must wait at the stop bar until pedestrians exit. However, in the field, minor-street vehicles tend to leave early. The crossing time of the minor street is also assumed as truncated Gaussian distribution, which accounts for different situations. Compared with Figure 6.8 (b), the capacity is lower, which is because the pedestrian crossing time is longer than left-turn critical headway. The average of pedestrian crossing time for Figure 6.8 (b) is six seconds and the critical headway for left turns is 4.1 seconds.

6.7 Pedestrian Impact on Right-turn Adjustment

Minor-street right turns usually have a conflict with adjacent pedestrian crossing, which is shown in Figure 6.9. However, minor-street through vehicles can use this gap to cross the street. The Monte Carlo simulation model mentioned before is used to capture the capacity increase of through vehicles due to this situation. Further, this new capacity is employed to calculate equivalent factors. The underlying assumption is that through and right-turn vehicles do not interrupt each other and right-turn capacity reduction due to pedestrian blockage is ignored. Therefore, this method just works for Configuration 3 with an exclusive right-turn lane. Table 6.4 shows the through movement capacity increment with different main-street volume and pedestrian volume. It is obvious that the increment grows with the increasing main-street and pedestrian volumes. When calculating the capacity increment, other factors like main-street pedestrian, left turns on the main street, yielding rate, and crossing time and are the same.

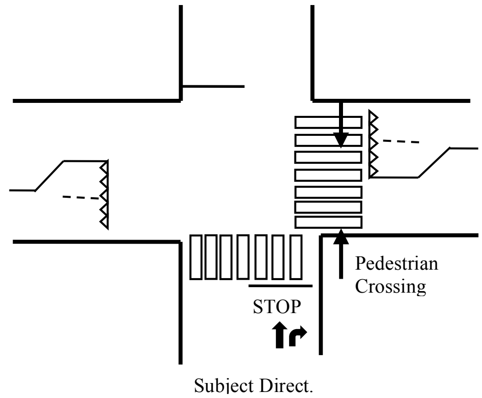


Figure 6.9 Minor Street Traffic and Conflicting Pedestrian

Table 6.4 Through Movement Capacity Increment in Percentage

Pedestrian Volume \ Main Street Volume	400	500	600	700	800	900	1000	1100	1200*
50	0	0.1	0.6	1.4	2.0	3.8	4.8	7.5	10.0
100	0.1	0.9	1.7	3.6	4.5	7.9	10.2	15.0	20.0
150	0.7	1.5	3.4	5.1	8.1	11.6	16.5	23.4	29.5

*When the main street volume is beyond 1200 vph, equivalent factors of 1200 vph are applied.

Three pedestrian levels, i.e. 50 pph, 100 pph and 150 pph, are considered to capture low, medium, and high pedestrian volume impacts. Results are shown in Tables 6.5, 6.6 and 6.7. We can see that equivalent factors increase significantly compared to Table 3.7 (without pedestrian impact). Due to the capacity increment trend, the equivalent factors of heavy main-street volume increase more. Therefore, the value difference of equivalent factors among different main-street volumes decreases. In medium and high pedestrian levels, the smallest equivalent factor is not in the highest main-street volume.

Table 6.5 Situation Equivalent factors for Configuration 3 with 50 PPH

Volume Ratio \ Main Street Volume	400	500	600	700	800	900	1000	1100	1200*
1:1	0.36	0.33	0.31	0.29	0.29	0.28	0.28	0.28	0.28
1:2	0.49	0.48	0.48	0.48	0.48	0.48	0.46	0.46	0.45
1:3	0.55	0.55	0.56	0.56	0.56	0.56	0.54	0.54	0.53
1:4	0.59	0.60	0.60	0.61	0.60	0.60	0.59	0.59	0.59
2:1	0.21	0.16	0.11	0.07	0.04	0	0	0	0
3:1	0.14	0.07	0	0	0	0	0	0	0
4:1	0.09	0.02	0	0	0	0	0	0	0

*When the main street volume is beyond 1200 vph, equivalent factors of 1200 vph are applied.

Table 6.6 Situation Equivalent factors for Configuration 3 with 100 PPH

Main Street Volume / Volume Ratio	400	500	600	700	800	900	1000	1100	1200*
1:1	0.36	0.33	0.32	0.31	0.30	0.31	0.30	0.31	0.31
1:2	0.49	0.49	0.49	0.50	0.49	0.50	0.49	0.49	0.49
1:3	0.56	0.56	0.57	0.58	0.57	0.58	0.58	0.58	0.58
1:4	0.59	0.60	0.61	0.62	0.62	0.63	0.63	0.64	0.64
2:1	0.21	0.16	0.12	0.08	0.05	0	0	0	0
3:1	0.14	0.07	0	0	0	0	0	0	0
4:1	0.10	0.02	0	0	0	0	0	0	0

*When the main street volume is beyond 1200 vph, equivalent factors of 1200 vph are applied.

Table 6.7 Situation Equivalent factors for Configuration 3 with 150 PPH

Main Street Volume / Volume Ratio	400	500	600	700	800	900	1000	1100	1200*
1:1	0.42	0.34	0.33	0.32	0.32	0.32	0.33	0.34	0.35
1:2	0.56	0.49	0.50	0.51	0.52	0.52	0.53	0.54	0.54
1:3	0.62	0.57	0.58	0.59	0.60	0.60	0.61	0.63	0.64
1:4	0.66	0.61	0.63	0.64	0.65	0.65	0.67	0.69	0.70
2:1	0.27	0.17	0.13	0.09	0.07	0	0	0	0
3:1	0.19	0.07	0	0	0	0	0	0	0
4:1	0.15	0.02	0	0	0	0	0	0	0

*When the main street volume is beyond 1200 vph, equivalent factors of 1200 vph are applied.

6.8 Conclusions

The proposed capacity model considering pedestrian crossing based on the Monte Carlo method shows the reliability of estimating the actual capacity for TWSC intersections. From model results and field observations, pedestrian crossing will increase minor-street capacity at significant levels.

This model is employed to consider the pedestrian impact on the right-turn traffic adjustment. Usually, pedestrian crossing would block adjacent minor-street right-turn vehicles, but benefits the through movement. According to calculation, equivalent factors increase significantly especially when the main-street volume and pedestrian volume are high. However, this estimation just considers the through movement capacity increment, but ignores the right-turn capacity reduction. Therefore, equivalent factors should be a little higher than these results. Basically, equivalent factors at high main-street volumes will not exceed those at 400 vph. It is recommended to use the results of main-street volume at 400 vph for all main-street volumes.

Chapter 7 Summary and Conclusions

Unwarranted traffic signals are detrimental not only to the flow of traffic but also to the overall delay. Including all right-turn traffic volume or an inappropriate portion could result in an erroneous traffic study and possible installation of an unwarranted traffic signal. A review of previous studies and existing guidelines revealed that engineers and scholars generally agree on reducing right-turn volumes in signal warrant analysis. However, determining the reduction percentage has primarily been done based on engineering judgments. Although limited guidelines exist on right-turn volume reduction, no detailed quantitative evaluations or theoretical methodologies were found in the literature. This research serves the purpose of filling out such methodological gaps by theoretically developing right-turn volume equivalent guidelines for the signal warrant analysis that do not exist presently. In general, research objectives were attained. The following conclusions can be drawn based on this research:

- The guidelines are essentially based on what is so-called the delay equivalence methodology, i.e. to equal the right-turn volume to an equivalent number of through vehicles, which would produce the same control delay on the minor street. Equivalent factors are defined as the measurement of reducing level.
- According to the geometry of the minor street, five configurations are classified, which cover the most common geometry. Under each configuration, a variety of volume distributions are considered which yielded a total of 63 study scenarios. Configuration 1 and Configuration 2 used the same equivalent factors based on calculation.
- Tables and figures are produced to indicate the right-turn traffic volume equivalent levels based on various combinations of volume values.
- Especially, the volume ratio in the two directions of the main street is important. From the analysis, a large volume ratio has a great impact on the right-turn movement. Therefore, ignoring this ratio can greatly affect the influence of the main-street traffic on the minor street.
- Regression equations were built based on a statistical method for all the configurations and regression coefficients are provided. The equation gives the equivalent factor for specific volume scenarios.
- The guidelines were tested with three intersection case studies. The results indicated that the proposed guideline could easily help engineers determine appropriate right-turn volume reductions.

- After adjustment, if the warrant results yield different conclusions than the original one, then right-turns volumes might be enough to affect the accuracy of the warrant. In this situation, it is recommended that the effect of right-turns be fully examined during the signal warrant study.
- Pedestrian impact on right-turn traffic adjustment was discussed. Basically, pedestrian crossing would block adjacent minor right-turn vehicles. A Monte Carlo model was built to calculate minor-street capacity considering pedestrian crossing. Due to the complicated relationship between the through and right-turn vehicles, the calculation just works for Configuration 3. It is observed that equivalent factors increase significantly due to pedestrian impact, especially at high main-street volume conditions.
- Field studies are crucial to adjusting right-turn traffic in signal warrant analysis, because every intersection's situation is usually unique due to its location, traffic volume, and geometry. Engineering judgment must be employed to check whether the reduction is appropriate. The final right-turn adjustment should include the observation of the actual operation of the intersection during the peak periods.

Reference

[1] Federal Highway Administration. Manual on Uniform Traffic Control Devices, 2009 Edition. Washington, DC: U.S. Department of Transportation, 2009.

[2] Mozdbar, A. A., Gerard, D. G., and Lammert, J. K. Guidelines for Inclusion of Right-turn Traffic in Signal Warrant Studies. ITE Journal, 2009, pp. 65-67.

[3] David R. McDonald Jr.. Traffic Signal Warrants: Two Agencies' Preferences, *ITE Journal*, January 2001, pp. 36-44.

[4] Illinois Department of Transportation Bureau of Traffic, District One. Signal Warrant Worksheet Procedures. Facsimile to David McDonald, 1997.

[5] Alabama Department of Transportation. Traffic Signal Design Guide and Timing Manual. 2007

[6] Bonneson, J. A., and Fontaine, M. D. Engineering Study Guide for Evaluating Intersection Improvements. NCHRP report 457, Transportation Research Board, Washington, DC, 2001

[7] State of Wisconsin Department of Transportation. Traffic Signal Design Manual. Bureau of Highway Operations, 2011. Available at: http://www.wisdot.info/microsimulation/images/3/3e/Tsdm.pdf

[8] Los Angeles Department of Transportation. Guidelines for traffic signals. 2008.

[9] Oregon Department of Transportation. Analysis Procedures Manual. Transportation Development Division Planning Section, 2006. Available at: http://www.oregon.gov/ODOT/TD/TP/APM/APMv1.pdf

[10] Steven L. Jones, Jr, Andrew J. Sullivan Shinde, Virginia P. Sisiopiku. Statewide Traffic Signal Design and Timing Manual. UCTA Report 04407, 2006.

[11] Arizona Department of Transportation. ADOT Traffic Engineering Policies, Guidelines, and Procedures. 2001. Available at: http://www.azdot.gov/business/engineering-and-construction/traffic/policies-guidelines-and-procedures-(pgp)

[12] Alliance Transportation group. Traffic Signal Warrant Analysis: Villa Maria Road at Autumn Lake Drive & Kingsgate Drive. 2013.

[13] Chilukuri, B. and Laval, J. Traffic Signal Volume Warrants – A Delay Perspective. *ITE Journal*, March 2012, pp. 36-41.

[14] Marek, J., Kyte, M., Tian, Z., Lall, K., Voigt, K. Determining Intersection Traffic Control Type Using the 1994 Highway Capacity Manual. *ITE Journal*, Vol. 67(12), 1997, pp. 22-26.

[15] Highway Capacity Manual, 2010 Edition. Washington DC: Transportation Research Board, 2010.

[16] MATLAB version 7.10.0. Natick, Massachusetts: The MathWorks Inc., 2010.

[17] Turner, S., K. Fitzpatrick, M. Brewer, and E. S. Park, Motorist Yielding to Pedestrians at Unsignalized Intersections. *Transportation Research Record: Journal of the Transportation Research Board*, 1982, pp. 1-12.

[18] Bonneson, J. A. Delay to Major-street Through Vehicles due to Right-turn

Activity. *Transportation Research Part A: Policy and Practice*, vol. 32, Issue 2, 1998, pp. 139-148.
[19] Bonneson, J. A., and J. W. Fitts. Delay to Major Street Through Vehicles at Two-way Stop-controlled Intersections. *Transportation Research Part A: Policy and Practice*, vol. 33, Issues 3-4, 1999, pp. 237-253.
[20] Wei D. L., J. Kumfer, H. Liu, Z. Tian, and C. Yuan. An Analytical Delay Model to Yielding Vehicles at Unsignalized Pedestrian Crossings. *Proceedings of the Transportation Research Board 92nd Annual Meeting*, Washington, DC, USA, January 2013.
[21] Yang, Z., Y. Y. Zhang, R. W. Zhu, X. F. Ye, and X. H. Jiang. Impacts of Pedestrians on Capacity and Delay of Major Street Through Traffic at Two-way Stop-controlled Intersections, *Mathematical Problems in Engineering*, vol. 2015, Article ID 383121, 2015.
[22] Li, H.Y., W. Deng. Determination of Capacity at Two-way Stop-controlled Urban Intersections *Proceedings of the IEEE International Conference on Automation and Logistics*, Jinan, China, August 2007.
[23] Breiman, L. The Poisson Tendency in Traffic Distribution. *Ann. Mark Slats*, vol. 34, 1963, pp. 308-311.
[24] Cowan, R. J. Useful Headway Models. *Transportation Research Record*, vol. 9, No. 6, 1975, pp. 371–375.
[25] Federal Highway Administration. Revised Monograph on Traffic Flow Theory. Available at: http://www.fhwa.dot.gov/publications/research/operations/tft/chap8.pdf.
[26] Siegloch, W. Ein Richtlinienvorschlag Zur Leistungsermittlung an Knotenpunkten Ohne Lichtsignalsteuerung (Capacity Calculations for Unsignalized Intersections). *Strassenverkehrstechnik*, vol. 1, 1974.
[27] Koller D., N. Friedman. *Probabilistic Graphical Models: Principles and Techniques*. The MIT Press, 2009.
[28] Brewer, M., P. Carlson, K. Fitzpatrick, et al. *Improving Pedestrian Safety at Unsignalized Crossing*. NCHRP REPORT 562, TRB, 2006.
[29] Bennett, S. A. Felton, and R. Akçelik. Pedestrian Movement Characteristics at Signalized Intersections. *23rd Conference of Australian Institutes of Transportation Research*, Monash University, Melbourne, Australia, December 10-12, 2001.
[30] Kirschbaum, J. B., P. W. Axelson, P. E. Longmuir, et al. *Designing Sidewalks and Trails for Access Part Ⅱ of Ⅱ: Best Practices Design Guide*, September 2001. Available at:
http://www.fhwa.dot.gov/environment/bicycle_pedestrian/publications/sidewalk2/sidewalks208.cfm

Nevada Department of Transportation
Rudy Malfabon, P.E. Director
Ken Chambers, Research Division Chief
(775) 888-7220
kchambers@dot.nv.gov
1263 South Stewart Street
Carson City, Nevada 89712