NDOT Research Report

Report No. 236-11-803

Signal Timing and Coordination Strategies Under Varying Traffic Demands

July 2012

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

Disclaimer

This work was sponsored by the Nevada Department of Transportation. The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the State of Nevada at the time of publication. This report does not constitute a standard, specification, or regulation.

SIGNAL TIMING AND COORDINATION STRATEGIES UNDER VARYING TRAFFIC DEMANDS

Final Report

Prepared for

NEVADA DEPARTMENT OF TRANSPORTATION

Prepared by

Rasool Andalibian

Zong Tian, Ph.D., P.E. (PI)

Center for Advanced Transportation Education and Research

Department of Civil & Environmental Engineering

University of Nevada, Reno, NV 89557

Email: zongt@unr.edu

July 2012

1. EXECUTIVE SUMMARY

Current practice for signal timing and signal coordination is to develop and operate a limited number of predetermined time-of-day plans. Coordination plans are commonly developed for and based on weekday morning, mid-day, evening, and weekend peak periods. During the remaining time periods signals are operating either in fully/semi-actuated modes or fixed modes. Engineers often face a dilemma to decide when signals should be coordinated. Numerous studies have been conducted to develop practical guidelines by considering volume, signal spacing, platoon dispersion, and signal timing parameters. Although some guidelines recognize the need of coordination when traffic demand is high, none of them explicitly quantifies the term “high”. Therefore, most decisions are still made based on engineering judgment. This project aims at developing more quantitative guidelines.

One of the most significant benefits of signal coordination is the reduction of number of stops which has received less attention in previous studies. For instance, none of the developed guidelines define stop thresholds when determining signal coordination. This research specifically focuses on this aspect. First, a probabilistic model that predicts the expected number of stops on non-coordinated arterials was developed. The model is a function of the effective green to the cycle length (g/C) ratio that considers the effect of traffic demand and capacity indirectly. It is also assumed that vehicle arrival is random which deems reasonable when demand is low in actuated operational mode. The model was validated through VISSIM simulation of a real signalized arterial, Sparks Blvd in Sparks, Nevada. The results confirmed that the model was highly reliable in estimating the number of stops.

Based on the probabilistic model, a practical signal timing guideline was developed according to the percentage of stops on non-coordinated arterials. The guideline states that:

- If the percentage of stops along a non-coordinated arterial exceeds 50, coordination is recommended for the arterial.
- If the percentage of stops along a non-coordinated arterial is 20 or lower, an actuated operation is recommended for the arterial.

- If the percentage of stops falls between 20 and 50, an engineering judgment should be applied to decide whether to coordination is necessary.

Finally, case study was conducted to demonstrate the application of the guideline. The same Sparks Blvd. arterial was used in the case study but with various traffic demand conditions. The case study confirmed the practicality of guideline.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

2. INTRODUCTION

Current practice for signal timing and signal coordination is to develop and operate a limited number of predetermined time-of-day plans. For instance, coordination plans are commonly developed for and based on weekday morning, mid-day, evening, and weekend peak periods. Timing plans are developed in a way to fit specific traffic patterns. During the remaining time periods, signals operate either in a fully/semi-actuated mode or in a coordinated mode selected based upon engineering judgment. While coordination generally benefits main street traffic, minor street traffic may incur longer delays which indicates traffic coordination is not beneficial at all times; it is more a trade-off between motorists who benefit from it and those who do not.

A common practice for signal timing is to coordinate signals when traffic demand is sufficiently high, e.g. during peak hours. The expression “sufficiently high” is a qualitative value that needs to be quantified since its meaning varies from agency to agency and situation to situation. On the other hand, when traffic demand is not high enough, it may be beneficial to operate signals in free modes. In an aim to develop a practical guidelines research has been conducted to determine at what time of day coordination plans should be implemented. In this research factors affecting signal coordination such as signal spacing, platoon dispersion, cycle length, and offsets have been studied. Nevertheless, traffic engineers still decide to coordinate signals based on experience. Moreover, in current guidelines the number of stops, which is considered as one of the significant measures of effectiveness, does not address the number of stops that are acceptable with respect to timing plans.

This research approaches signal timing by developing a new guideline based on the percentage of stops for non-coordinated arterial streets considering drivers’ thresholds over the number of stops. Correspondingly, a probabilistic model that predicts the number of stops along non-coordinated arterials based on signal timing parameters will be developed. The developed probabilistic model and proposed guideline would be validated through simulating a real case study in VISSIM. The probabilistic model would be examined by analyzing the number of stops obtained from the simulation model and the guideline would be examined through considering the improvement of travel time along an arterial and excessive delays inferred by side street traffic due to the operation of coordination plans.

This report is organized as follows. After this introduction, chapter 3 deals with the literature review of signal coordination. Chapter 4 presents the developed probabilistic model that predicts number of stops along non-coordinated arterials and the proposed signal timing guideline. Chapter 5 summarizes a case study simulation model to assess the developed probabilistic model and it also validates the proposed signal timing guideline. Finally, Chapter 6 presents the conclusion and practical recommendations for signal timing and signal coordination.

3. LITERATURE REVIEW

Signal coordination has been a subject of attention for many years and several methods have been developed to produce optimal coordination plans. Whether two adjacent signals should be coordinated and at what time of day coordination plans should be implemented depends on the amount of benefit that can be obtained. Most traffic engineers decide to coordinate signals based on experience mainly because recommendations in manuals concerning time of a day signal coordination plans do not deal with the issue explicitly. Several parameters influence signal coordination plans such as signal spacing, cycle length, offsets, and so on, thus research by the aim of providing operational guidelines for implementing better signal coordination plans have been provided.

3.1 Signal Coordination: Advantageous and Disadvantageous

The primary benefits of signal coordination are the reduction in delays, travel time, and number of stops for the coordinated directions which result when a platoon of vehicles is allowed to move continuously through several intersections. Additionally, a good progression plan results in other benefits that include:

- Reduction in fuel consumption and vehicle emissions
- Maintenance of preferred travel speed
- Reduction in vehicular accidents
- Eliminating road widening
- Reduction in motorists' frustration and road rage.

However, achieving a coordinated system is not possible at all times. There are several factors impeding the benefits of signal coordination such as: inadequate roadway capacity, existence of side frictions like parking or multiple driveways, complicated intersections that involve multiphase controls, wide variability in traffic speeds, very short signal spacing, and heavy left-turn movements.

In contrast to the benefits signal coordination brings, it has its own drawbacks such as:

- Increase of delays and stops for minor streets
- Attraction of additional traffic through the corridor
- Maintenance and equipment costs may be high based on the type of hardware and software used
- Requirement of qualified staff for maintenance and monitoring of daily operations.

3.2 Signal Coordination and Influential Factors

Achieving an optimal and reasonable coordination plan is not possible without recognition of the factors affecting signal coordination. Traffic signal coordination plans are strongly influenced by parameters such as corridor speeds, signal spacing, traffic volumes, traffic congestion, pedestrian volumes, cycle lengths, phasing sequences, offsets, and safety. In this section the major aforementioned parameters will be discussed.

3.2.1 Signal Spacing

Signal spacing also known as link length has an effect on the amount of platoon dispersion. Platoon dispersion becomes more significant as the distance between signals increases. There is some research dealing with appropriate signal spacing in terms of signal coordination.

The Signal Timing Manual recommends coordinating signals when signals are in close proximity of one another and there is a large amount of traffic on the coordinated street [1]. However, it does not provide a quantitative value for "close proximity". MUTCD's, chapter 4D Traffic Control Signal Features, states that traffic signals within 0.5 mile of each other should be coordinated, preferably with interconnected controller units [2]. In a study conducted for FHWA, entitled signal timing on a shoestring, it states that when intersections are close together e.g., within ¾ mile of each other, it is advantageous to coordinate them. At greater distances, e.g. ¾ mile and greater, the traffic volume and potential for platoon dispersion should be reviewed for coordination operations [3]. Neither too far apart signals nor too closely spaced signal are recommended for coordination. Basically, signals with less than 300 feet (0.06 mile) distance are not recommended to be coordinated [4]. The Traffic Control Systems Handbook states that for good progression the ratio of travel time to cycle length should fall between 0.4 and 0.6 [5].

Change and Messer showed that ideal progression spacing is approximately the travel time of one-third to one-half the cycle length times the design speed in any generalized arterial street [6].

3.2.2 Signal Timing Parameters

Three fundamental parameters that distinguish a coordinated signal system are: cycle length, phase split, and offset. These settings are necessary inputs for coordination. Additionally, a phasing sequence is another important component that influences the quality of progression.

The selection of cycle lengths for a coordinated system depends upon the network optimization criteria. Generally, there are two methods to generate coordination plans for arterial streets: (1) maximization of the bandwidth of the progression, (2) minimization of the overall delay, number of stops, or other measure of disutility. Bandwidth optimization methods result in longer cycle lengths since those methods are attempting to provide as wide of a progression band as possible. Koshi attempted to determine an optimum cycle length considering both delay and the number of stops for a two-way coordinated system. In the study it was assumed that: (1) cycle length, green splits, and saturation flow rates were the same for two adjacent signals, (2) through traffic only, no turning movements, (3) no speed distribution which means no platoon distribution, and (4) green time was set 50% of the cycle length. At the end, it was concluded that both the delay and stops of a link are minimized when the cycle time is equal to T/n (n=1, 2, 3 …), where T is the travel time of a round trip of the link [7].

Once the cycle length is determined, then the phase split, phasing sequence, and offset for each intersection should be determined. The phase split is determined based on the approach demand. Unlike phase splits, which have not been an issue, offsets have received a lot of attention. Little set up the computation of optimal offsets among the traffic signals of arterial systems, aiming at the maximization of bandwidth, as a mixed-integer linear programming, and extended this formulation to the case of signalized networks [8]. Hillier and Whiting developed the Combination Method, a procedure which calculates optimal offsets for series-parallel signalized networks using discrete delay-offset functions [9]. Gartner et al. approached the problem of the computation of optimal offsets, and of the simultaneous optimization of all control variables including offsets, cycle length, and green splits by means of mixed-integer linear programming models [10].

3.2.3 Traffic Volume

It is true to state the most influential factor in signal coordination is traffic volume. Characteristics of traffic flow are substantially affected by traffic volume. For instance, the higher the traffic volume, the more condense the platoon dispersion. Not only does traffic volume impacts signal coordination, but also signal coordination directionality of traffic, percentage of left-turns and amount of traffic entering, exiting, or crossing from side streets have an effect on signal coordination. There is a general belief stating that traffic coordination plans are reasonable and justifiable when traffic volumes between adjacent intersections are very high i.e. during peak hours. As a rule of thumb there is no point in signal coordination after 9:00 p.m. Even, signal timing manuals states that signal coordination is easily justifiable when signals are in close proximity of each other and traffic demand is high.

Heavy left-turn movements play an important role in signal coordination. When the percentage of left-turn movements in comparison to through movements is high, achieving signal coordination might be difficult.

3.2.4 Traffic Flow Characteristics

Flow arrival patterns have great impact on the quality of progression. While controlled flow in platoons that arrives during green time enhances the quality of coordination, uniform arrival or arrival during red time decreases the quality of progression. Arrival patterns are a function of offsets, signal spacing, and platoon dispersion. Empirical observations of platoon dispersion were first reported by Pacey [11]. Later, Hillier and Rothery attempted to ascertain whether or not neighboring intersections can be effectively coupled on the basis of vehicular platoons [12]. They combined the concept of a cyclic platoon profile with a delay model. Robertson integrated the delay minimization concept with a formalized platoon dispersion model that formed the basis of TRANSYT [13]. Manar and Baas showed that at low traffic volume conditions dispersion is low, but it increases as traffic volumes increase up to the maximum at volumes of 60 to 80 percent of the link capacity. Dispersion approaches zero near the capacity of the link. Their findings prove that signal coordination during off-peak hours needs further analysis and research since the platoon dispersion reaches its peak during off-peak hours [14]3.2.5 Pedestrians

Pedestrians require more time to cross and clear an intersection than vehicles because of their lower operating speeds. This time difference has a great impact on the amount of green time that a phase must provide. Considering pedestrian volumes, two strategies are applied by traffic engineers for signal timing: (1) timing based on pedestrian minimums, and (2) timing based on vehicle minimums. Timing based on pedestrian minimums requires accommodation of pedestrian crossing times in the controller phase splits. The major advantage of this strategy is that the signal will remain in coordination regardless of whether there is a pedestrian phase activation; thus it is a preferred alternative from the point of view of system operations. Timing based on vehicle minimums may become problematic when the pedestrian crossing time is greater than the vehicle crossing time. In this case, the presence of pedestrians interrupts the coordination performance and causes the signal to work out of coordination for a short while. Despite the existence of signal timing strategies pertaining to level of pedestrian, there is a lack of recommendations and guidelines on pedestrian volumes to determine which strategy should be applied.

3.3 Current Practice and Techniques

Current practice for signal timing is to coordinate signals during peak hours, e.g. AM, midday, and PM peaks and to run signals either fully/semi actuated or fixed timed during off-peak hours. This strategy comes from the belief stating that signals should be coordinated when traffic demand is sufficiently high. However, the term "sufficiently high" has not been quantified. In an attempt to determine at what time of day signal coordination plans should operate, several numeric heuristic techniques have been developed. In fact, these techniques determine when two adjacent signals should be coordinated.

Yagoda introduced the concept of the coupling index (CI) which is a ratio of the link volume (V) to distance (D) [15]:

$$CI = \frac{V}{D} \qquad (1)$$

A variant of this concept is the so-called gravity model, which considers the weight of the distance squared as follow [16]:

$$CI = \frac{V}{D^2} \quad (2)$$

Later, the model was further developed by considering the formation and dispersion of a platoon on links through a network. Strength of Attraction is one of the models that embedded platoon interference (I) along with signal spacing (D), traffic volume (V), and travel speed (S).

$$AF = I \times V \times (\frac{S}{D})^2 \quad (3)$$

Platoon interference is a unitless value describing the interference of the platoon as it progresses down the street. For instance, a platoon interference factor of 2.0 can be used for roadways without parking, 1.5 for roadways with parallel parking, and 1.0 for roadways with angled parking.

Synchro has an internal methodology to calculate the coordinatability factor (CF) based on travel time ($CF1$), signal spacing ($CF2$), link volume (Av), vehicle platooning (Ap), vehicle queuing, and natural cycle length (Ac). The natural cycle length is defined as the cycle at which the intersection would run in an isolated mode. The factor ranges from 0 to 100 or more [17].

$$CF = Max(CF1, CF2) + Ap + Av + Ac \quad (4)$$

Table 3.1 presents a summary of the three coordinatability factors including coupling index, strength of attraction, and Synchro coordinatability factor [16].

Table 3.1 Coordinability Methodologies and Recommended Breakpoints

Methodology	**Coordination NOT desirable**	**Coordination Desirable**	**Coordination Critical**
Coupling Index	< 1.0	1 to 50	> 50
Strength of Attraction	< 0.5	0.5 to 2	> 2
Synchro	< 20	20 to 80	> 80

Hook and Albers studied and compared the effectiveness of the improved coupling index, strength of attraction, and coordinatability factor in determining the potential of coordination between two adjacent signals. All the indices were tested on five randomly chosen links. In their study, they arrived to the conclusion that there is no absolute best factor for determining when signals should be coordinated (or where progression breaks should occur). They believed that each method gave about the same result; the simpler methods were just as valid as the complicated ones. Finally, they stated that, as in every aspect of engineering, judgment and experience are the best tools [16].

In supporting Hook and Albers conclusion, the Traffic Control Systems Handbook states that "while coordination of adjacent signals often provides benefits, the traffic systems engineer must decide, in each case, whether better performance will be achieved with coordinated or isolated operations". It is also mentioned that "when a platoon of vehicles is released from a traffic signal, the degree to which this platoon has dispersed at the next signal (difference from profile at releasing signal) in part determines whether significant benefits can be achieved from signal coordination" [4].

3.4 Coordination Software

A number of computer programs are available to assist traffic engineers in analyzing and coordinating traffic signals. All of them are based on the abstraction of reality and naturally have their own strengths and weaknesses. For instance, none of these tools can handle saturated and over-saturated conditions well. Computer programs for signal timing and coordination plans are classified into two models; (1) traffic simulation models also called traffic models and (2) optimization models or analytical models.

- **Traffic Simulation Models**

Traffic simulation models take traffic volumes, geometric information, and traffic control plans as inputs. Then, they simulate the defined scenarios and report various measures of effectiveness (MOESs) as outputs. Typical MOES include: average intersection delay, number of stops, average queue, maximum queue, corridor speed, fuel consumption, total bandwidth, bandwidth efficiency, bandwidth attainability, and system throughput. Most traffic simulation models provide an estimate of several, if not all, MOEs. In fact, simulation models are able to tell how good or bad a given scenario is. The most widely-used traffic simulation models are CORSIM, VISSIM, and SimTraffic. Among these three simulation models CORSIM and VISSIM are more reliable and have less abstraction from reality.

- **Traffic Optimization Models**

Optimization models are able to generate an optimal scenario based upon defined network optimization criteria. The most widely-used programs are TRANSYT-7F, PASSER II, PASSER V, and Synchro. TRANSYT-7F and Synchro are based-delay models but PASER is a bandwidth-based program. Generally, Synchro, PASERII, and PASSER V produce good timing plans depending on the objective of coordination.

Texas Department of Transportation, Austin, recommends using PASSER V if the primary objective is maximizing arterial progression bandwidths. Bandwidth optimization result in maximum throughput and minimum stops. On the other hand, if the primary purpose is to implement a timing plan that produces minimum delay, Synchro is recommended. However, it should be noted that Synchro does not account queue spillback especially for short links.

3.5 Summary and Conclusions

Whether signals should be coordinated and at what time of day coordination plans should operate depends on the amount of benefit the coordination plan brings. Most traffic engineers have an opinion based on experience about when to coordinate signals. There is a common belief among traffic engineers considered as a rule of thumb stating that, when traffic demand is sufficiently high signals should be coordinated. However, signal coordination not only is a function of traffic demand but also it is a function of several other parameters including traffic volume, signal spacing, platoon dispersion, cycle length, phase split, offsets, and so on. Numerous studies by the aim of providing operational guidelines have been conducted over platoon dispersion, signal spacing, cycle length, and optimal offsets. Despite these studies, there still is a lack of consistency over traffic demand and the definition “high traffic volume” as it has not been quantified in the literature. In addition, none of the studies and guidelines analyze the coordination plans from the number of stops point of view.

4. STOP PROBABILISTIC MODEL AND SIGNAL TIMING STRATEGY

One of the most important MOEs in evaluating a signal timing plan for main arterials is the number of stops. Generally, the lower the number of stops the better the timing plan. When there are a couple of intersections along an arterial, the arrival pattern of vehicles at each intersection highly depends on the signal timing pattern. When signals run in free mode it means the arrival pattern at each intersection is independent from the upstream signal which means the arrival pattern is random. On the contrary, when signals are coordinated the arrival pattern is not random anymore and it follows the platoon dispersion models. Using the fact that when signals run in free mode the arrival pattern is random, it would be possible to predict the probability of stops and consequently the expected number of stops of a vehicle along an arterial. Considering the random arrival pattern of vehicles when signals are run free, this section attempts to develop a probabilistic model that predicts the number of stops along non-coordinated arterials. Afterwards, a practical guideline stipulating a signal timing strategy according to the predicted number of stops would be recommended.

4.1 Probabilistic Model

4.1.1 Assumptions

The probability of going through (hitting green at) an intersection with random arrival flow would be the ratio of green time to the cycle length and consequently, the probability of making a stop (hitting red) would be the ratio of red time to the cycle length as follow:

$$P_g^i(a) = g^i(a)/\ C(a) \tag{5}$$

$$P_r^i(a) = r^i(a)/\ C(a) \tag{6}$$

$$C(a) = g^i(a) + r^i(a) \tag{7}$$

$$P_g^i(a) + \ P_r^i(a) = 1 \tag{8}$$

Where,

$P_g^i(a)$ = probability of going through at intersection a, in direction i,

$P_r^i(a)$ = probability of making a stop at intersection a, in direction i,

$g^i(a)$ = effective green time at intersection a, in direction i,

$r^i(a)$ = red time at intersection a, in direction i,

$C(a)$ = cycle length at intersection a.

When there are a couple of intersections along an arterial two cases occur. A simple case is when all the signals are “homogenous” which means the probability of making a stop at each intersection is the same. A more realistic case is the “non-homogenous” case when the probability of stop at each intersection differs from the other intersections.

Besides arrival assumptions, additional assumptions are made, listed below, in order to develop a probabilistic model for estimating the number of stops.

1) Probability of making a stop at each intersection is independent from the other intersections.
2) The state of traffic flow is under saturated.

4.1.2 Homogenous Case

In this case the probability of making a stop at each intersection is the same. Therefore, when there is an array of n intersections, the probability of making a stop at each intersection is P_r^i.

$$P_g^i(a) = P_g^i(b) = \cdots = P_g^i(n) = \boldsymbol{P_g^i} \qquad (9)$$

$$P_r^i(a) = P_r^i(b) = \cdots = P_r^i(n) = \boldsymbol{P_r^i = 1 - P_g^i} \qquad (10)$$

Since there are only two conditions either hitting green or red and the number of signals is limited, the probability of making x stops out of n signals follows the binomial distribution:

$$SPF_x^i = \binom{n}{x}(1 - P_g^i)^x\,(P_g^i)^{n-x} \qquad (11)$$

Where,

SPF_x^i = probability of making x stops in direction i,

n = total number of signals in direction i,

P_r^i = probability of making a stop at each intersection in direction i,

Therefore, the expected number of stops (Ex^i) in direction i is calculated by:

$$EX^i = \sum_{x=0}^{n}(x.SPF_x^i) = n(1- P_g^i) \qquad (12)$$

4.1.3 Non-homogenous Case

Under this condition, the probability of making a stop at each intersection varies from intersection to intersection. The probability of making x stops out of n intersections can be calculated by applying a joint probability distribution.

$$SPF_0^i = \bigcap_{j=1}^{n} P(j)_g^i = \prod_{j=1}^{n} P(j)_g^i \qquad (13)$$

$$SPF_1^i = \sum_{j=1}^{n}\left\{\left[\prod_{j_1=1,\#j}^{n}(P(j_1)_g^i)\right]\left(P(j)_r^i\right)\right\} \qquad (14)$$

$$SPF_2^i = \sum_{j=1}^{n}\langle \sum_{j_1=j+1}^{n}\left\{\left[\prod_{j_2=1,\#j,k}^{n}(P(j_2)_g^i)\right]\cdot P(j_1)_r^i \cdot P(j)_r^i\right\}\rangle \qquad (15)$$

$$SPF_x^i = \sum_{j=1}^{n}\left(\sum_{j_1=j+1}^{n}\left(\sum_{j_2=j_1+1}^{n}\left(\dots \sum_{j_{x-1}=j_{x-2}+1}^{n}\left(\left[\prod_{j_x=1,\#j_1,\dots,j_{x-1}}^{n}(P(j_x)_g^i)\right]\cdot P(j)_r^i \cdot P(j_1)_r^i \cdot \dots \cdot P(j_{x-1})_r^i\right)\right)\right)\right) \qquad (16)$$

The expected number of stops is calculated by:

$$EX^i = \sum_{x=0}^{n}(x.SPF_x^i) = n - \sum_{a=1}^{n} P_g^i(a) \qquad (17)$$

Equation (17) can be proven as follow:

If $n = 2$

$SPF_0^i = P_g^i(1).P_g^i(2)$

$SPF_1^i = P_g^i(1).(1 - P_g^i(2)) + (1 - P_g^i(1)).P_g^i(2)$

$SPF_2^i = (1 - P_g^i(1)).(1 - P_g^i(2))$

$\boldsymbol{EX^i} = \sum_{x=0}^{2}(x.SPF_x^i) = 0 \times P_g^i(1).P_g^i(2) + 1 \times P_g^i(1).(1 - P_g^i(2)) + (1 - P_g^i(1)).P_g^i(2) +$
$2 \times (1 - P_g^i(1)).(1 - P_g^i(2)) = \mathbf{2} - [\boldsymbol{P_g^i}(\mathbf{1}) + \boldsymbol{P_g^i}(\mathbf{2})]$

If $n = 3$

$SPF_0^i = P_g^i(1).P_g^i(2).P_g^i(3)$

$SPF_1^i = P_g^i(1).P_g^i(2).(1 - P_g^i(3)) + P_g^i(1).(1 - P_g^i(2)).P_g^i(3) + (1 - P_g^i(1)).P_g^i(2).P_g^i(3)$

$$SPF_2^i = P_g^i(1).\left(1 - P_g^i(2)\right).\left(1 - P_g^i(3)\right) + \left(1 - P_g^i(1)\right).P_g^i(2).(1 - P_g^i(3)) + (1 - P_g^i(1)).\left(1 - P_g^i(2)\right).P_g^i(3)$$

$SPF_0^i = (1 - P_g^i(1)).(1 - P_g^i(2)).(1 - P_g^i(3))$

$\boldsymbol{EX^i} = \sum_{x=0}^{2}(x.SPF_x^i) = 0 \times P_g^i(1).P_g^i(2).P_g^i(3) + \cdots + 3 \times (1 - P_g^i(1)).(1 - P_g^i(2)).(1 - P_g^i(3)) =$
$\mathbf{3} - [\boldsymbol{P_g^i}(\mathbf{1}) + \boldsymbol{P_g^i}(\mathbf{2}) + \boldsymbol{P_g^i}(\mathbf{3})]$

Then, it could be confirmed that for n intersections the expected number of stops is:

$$\boldsymbol{EX^i} = \sum_{x=0}^{n}(x.SPF_x^i) = \boldsymbol{n} - \sum_{\boldsymbol{a=1}}^{\boldsymbol{n}} \boldsymbol{P_g^i}(\boldsymbol{a})$$

- **Simplified Non-homogenous Case**

In terms of estimating the expected number of stops it would be possible to simplify the non-homogenous case to a homogenous case by using the average of the probabilities. This is demonstrated below:

$$EX^i = \sum_{x=0}^{n}(x.SPF_x^i) = n - \sum_{a=1}^{n} P_g^i(a) = n\left(1 - \frac{1}{n}\sum_{a=1}^{n} P_g^i(a)\right) = n(1 - P_g^i(*))$$

Therefore,

$$EX^i = n(1 - P_g^i(*)) \tag{18}$$

$$P_g^i(*) = \frac{1}{n}\sum_{a=1}^{n} P_g^i(a) \tag{19}$$

It should be noted that this simplification is not true for the stop probability distribution.

4.1.4 Numerical Examples

To demonstrate how these probabilistic models function, a hypothetical arterial with five intersections is considered as an example. First, it is assumed the network is homogenous then it is assumed the network is non-homogenous. Finally, the results are compared with one another.

- **Homogenous Network**

In this case the probability of hitting green at any intersection is 0.80 for northbound and 0.65 for southbound. Figure 4.1 illustrates the stop probability distribution for both northbound and southbound directions. The expected number of stops for NB and SB directions is 1.0 and 1.7, respectively.

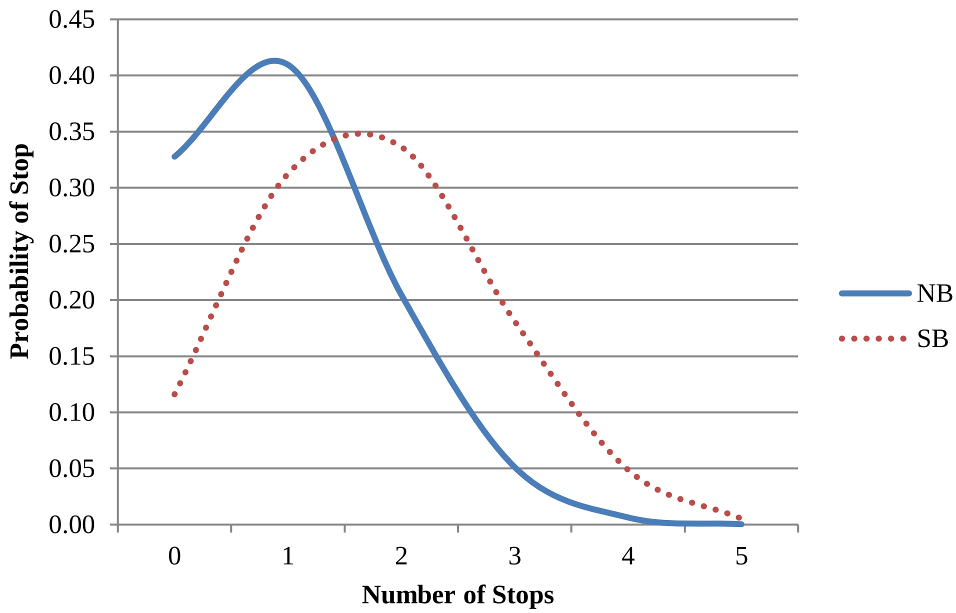

Figure 4.1 Stop Probability Distribution, Homogenous Case

- **Non-homogenous Network**

In this case the probability of hitting green at each intersection is different but the average probability of going through is 0.80 for northbound and 0.65 for southbound. Table 4.1 presents the probability of hitting green at each intersection. It should be noted that all these values were generated randomly but objectively in a way to have the same mean as the homogenous example.

Table 4.1 Probability of Hitting Green at Hypothetical Non-homogenous Network

Intersection #	NB	SB
1	0.68	0.70
2	0.75	0.60
3	0.82	0.56
4	0.92	0.63
5	0.83	0.78
AVE	**0.80**	**0.65**

Figure 4.2 demonstrates the stop probability distribution for this example, as it can be seen from this figure, the behavior of homogenous and non-homogenous networks are very similar.

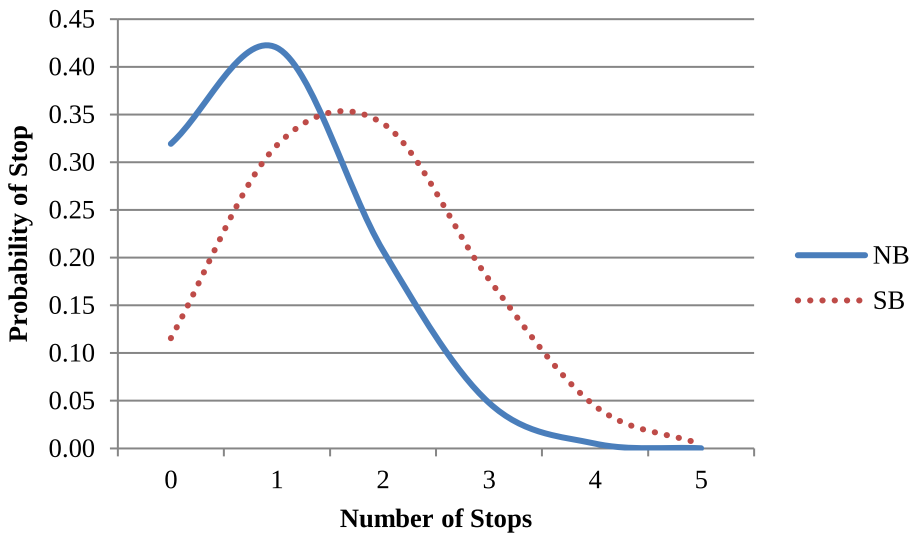

Figure 4.2 Stop Probability Distribution, Non-homogenous Case

In this example expected number of stops for NB and SB direction is 1.0 and 1.7 respectively.

- **Example Conclusion**

Comparing the results shows that the expected number of stops from the two cases is the same, although there is a difference in the stop probability distribution. Therefore, the non-homogenous case could be simplified to the homogenous case by using the average of the probabilities.

4.2 Guideline Development

The minimum expectation from a signal coordination plan is "one stop, one go" which means once a vehicle makes a stop at one intersection it should not make a stop at the next one, then it is allowed to make a stop at the third intersection. Figure 4.3 illustrates the concept of "one stop, one go".

From Figure 4.3 it is inferred that a poor signal coordination plan provides 50% stops, in other words for a coordination plan a maximum of 50% stops are allowable. It can be concluded that when signals are run free they bring about more than 50% stops, thus consecutive stops are unavoidable. Therefore, it is recommended switching to a coordination plan so as to decrease the percentage of stops at least to 50%.

On the other hand, and based upon engineering judgment, a signal timing plan that provides 20% stops and lower is a good plan. 20% stops means in every 5 intersections there is only one stop. Therefore, when running signals free brings about 20% or less stops, coordination should be excluded from the timing plan.

4.2.1 Percentage of Stops

The probabilistic model predicts the number of stops for non-coordinated arterials. Nevertheless, percentage of stops provides better insight into the performance of the network and corresponding signal timing plan. Percentage of stops defined as the expected number of stops over total number of signalized intersections is calculated as follows:

$$\%SP^i = 100 \times \frac{EX^i}{n} \tag{20}$$

Since the expected number of stops is a function of g/C ratio, percentage of stops can be written in terms of g/C ratio for both homogenous and non-homogenous cases.

- Homogenous Case

$$\%SP^i = 100\frac{EX^i}{n} = 100\frac{n(1-P_g^i)}{n} = 100(1-P_g^i)$$

- Non-homogenous Case

$$\%SP^i = 100\frac{EX^i}{n} = 100\frac{n-\sum_{a=1}^{n} P_g^i(a)}{n} = 100\frac{n(1-P_g^i(*))}{n} = 100(1-P_g^i(*))$$

4.2.2 Signal Timing Guideline

Initially, a conceptual guideline was developed to improve the signal timing strategy for non-coordinated arterials pertaining to the developed probabilistic model. To be specific:

- The expected percentage of stops is estimated by applying the developed probabilistic model based on a non-coordination plan.
- If the percentage of stops along a non-coordinated arterial exceeds 50%, it is highly recommended to switch from the non-coordination plan to a coordination plan.
- If the percentage of stops along a non-coordinated arterial is 20% or lower, it is highly recommended to run signals free.
- If the percentage of stops falls between 20% and 50%, engineering judgment should be applied to decide whether to apply a coordination plan or non-coordination plan. However, as the percentage of stops approaches 50%, it is recommendable to consider a coordination plan rather than a non-coordination plan.

4.3 Summary and Discussion

This section proposed a probabilistic model for predicting the expected number of stops along non-coordinated arterial streets and correspondingly, a signal strategy guideline discussing when signal coordination plans should operate and when they should not. The developed probabilistic model is a function of the number intersection and g/C ratio in which the v/c ratio is indirectly treated. According to the model, the higher the g/C ratio is the lower the expected number of stops would be. Basically, under low traffic volume conditions, it is expected that a fewer number of stops along an arterial street will be made which leads to a larger g/C ratio. When signals are operating in a fully actuated mode as the traffic volume decrease, the g/C ratio decreases as well, which is in contrast with the previous statement. Therefore, in order to produce a larger g/C ratio under low traffic volume conditions, min-recalls should be placed in coordinated directions. As a conclusion, the developed probabilistic model is workable as long as the min-recalls are placed in the main directions. The next chapter will discuss the validation of both the developed model and proposed guideline.

5. SIMULATION MODEL

In order to validate the developed probabilistic model introduced in the previous section a simulation model is applied. The intent of the simulation model is not only to assess the performance of the probabilistic model but also to make a comparison between coordination and non-coordination plans to evaluate the developed guideline efficiency and accuracy for signal timing strategies. Therefore, Sparks Blvd, located in the Reno-Sparks area in Nevada was chosen as a case study.

5.1 Sparks Blvd Characteristic

Sparks Blvd is one of the major arterials in the Reno-Sparks urban area with nearly 4.5 miles in length that encompasses 10 intersections. Figure 5.1 illustrates the study section of Sparks Blvd.

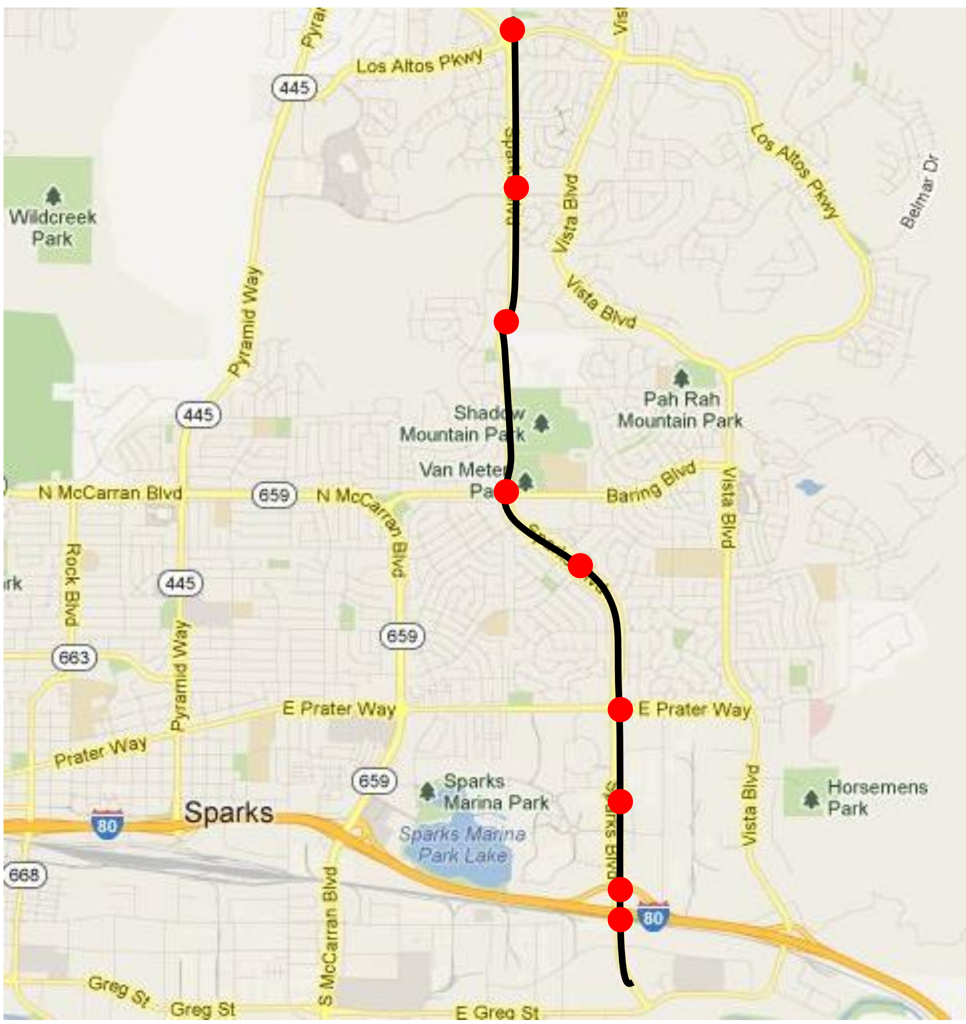

Figure 5.1 Sparks Blvd and Its Vicinity

Table 5.1 lists all the signalized intersections and relevant spacing information which was obtained from Google Maps.

Table 5.1 Intersection Spacing Distance along Sparks Blvd

No	Intersection (1)	Intersection (2)	Distance (feet)
1	Sparks/Los Altos	Sparks/Disc Dr.	3,794
2	Sparks/Disc Dr.	Sparks/Shadow Ln	7,822
3	Sparks/Shadow Ln	Sparks/Baring	2,613
4	Sparks/Baring	Sparks/O'Callaghan	2,502
5	Sparks/O'Callaghan	Sparks/Prater Way	3,783
6	Sparks/Prater Way	Sparks/E Lincoln	3,033
7	Sparks/E Lincoln	Sparks/I80 WB	1,357
8	Sparks/I80 WB	Sparks/I80 EB	729
SUM			22,633

AM and PM peak hour traffic volumes are about 1200 vph to 1700 vph, and the midday peak hour is around 700 vph. These values consider the total of northbound and southbound traffic volumes. The posted speed limit at Sparks Blvd is 40 mph. The prevailing drivers speed did not measure in the field and since there are always drivers who drive above the speed limit, in this study the prevailing speed is assumed to vary from 35 to 45 mph.

Due to the speed limit and inventory of each intersection along Sparks Blvd, having an exclusive left-turn lane, each intersection has a dual left-turn phase.

5.2 Demand Scenario

The developed probabilistic model is a function of the g/C ratio which is a function of volume-to-capacity (v/c) ratio. Therefore, in order to perform a thorough evaluation of the probabilistic model and the developed guideline, various demand levels ranging from low traffic conditions to high traffic conditions should be considered. Since only AM, midday, and PM peak hour traffic volumes are available other traffic conditions, covering the off-peak hours, were generated. The mid-day traffic volume was considered to be the starting point for generating various traffic demands, and then traffic volume was first increased by 20 percent, then it was reduced by 20, 40, 60, 70, 80, and 90 percent. Since these numbers do not directly transfer to a meaningful definition, the concept of the average v/c ratio was applied. In this regard, the average of the v/c ratio for the NB and SB through movements of all intersections is considered as a distinguishing point between demand scenarios. For getting the v/c ratio, the turning movement volumes are put into TRAFFIX then the v/c is obtained by using the optimization option according to the HCM 2000 methodology. Table 5.2 presents v/c ratios that are associated with each demand scenario being applied in this study.

Table 5.2 Volume-to-Capacity Ratio

Percent of Demand % (from mid-day)	$(\boldsymbol{v/c})^{NB}$	$(\boldsymbol{v/c})^{SB}$	$\boldsymbol{Ave}\ (\boldsymbol{v/c})$
+20	0.63	0.54	0.59
00	0.53	0.46	0.51
-20	0.41	0.36	0.39
-40	0.29	0.25	0.27
-60	0.19	0.16	0.18
-70	0.14	0.11	0.13
-80	0.09	0.07	0.08
-90	0.04	0.04	0.04

5.3 Timing Plan and Actuated g/C Ratio

Signal timing parameters such as minimum green, maximum green, yellow change interval, all-red time, vehicle extension, pedestrian WALK, and FDW in addition to other relevant information related to the inventory of the intersections were obtained from the Nevada Department of Transportation, NDOT, in the format of a Synchro file. Signals were run in a fully actuated mode and min-recalls were placed for through movements in the north and south bound directions considered as the main directions. Therefore, green will remain on the main street unless there is a side street call.

The main parameter in the probabilistic model is the actuated g/C ratio. For the purpose of this study the actuated g/C ratio is obtained from Synchro in both the HCM and Synchro method. There is a difference between the Synchro and HCM methodologies for calculating actuated g/C which comes from the fact that the HCM method requires all lost time from other phases to be included in the red time and cycle length. However, the percentile method discounts part of the lost time for other phases if they sometimes skip.

The average g/C ratio for each demand scenario regarding the HCM and Synchro methodologies is provided in Table 5.3.

Table 5.3 Actuated g/C Ratios Associates with Demand Scenarios

	NB Direction		SB Direction	
$Ave\ (v/c)$	$(g/c)^{HCM}$	$(g/c)^{Synchro}$	$(g/c)^{HCM}$	$(g/c)^{Synchro}$
0.59		0.45		0.40
0.51	0.43	0.45	0.39	0.40
0.39	0.46	0.50	0.42	0.46
0.27	0.51	0.55	0.47	0.50
0.18	0.58	0.66	0.56	0.63
0.13	0.63	0.72	0.62	0.70
0.08	0.69	0.82	0.67	0.80
0.04	0.74	0.85	0.71	0.84

Another timing plan for the coordinated condition was developed in order to make thorough comparisons between coordination and non-coordination plans. The coordinated plan mostly favored the southbound direction.

5.4 VISSIM Model and Simulation Outputs

5.4.1 VISSIM Model

After obtaining and preparing all necessary information including intersection inventory, signal timing parameters, traffic volumes, speed limit, and so on. Sparks Blvd was modeled in VISSIM 5.40. Figure 5.2 depicts Sparks Blvd's network in VISSIM. The simulation duration was set to be 1 hour with 600 seconds as warm up time. The warm-up time was set as 600 seconds since the travel time along Sparks Blvd, in a bad condition is nearly 500 seconds. The number of runs was set as 10 and an average output of 10 times was considered as the VISSIM output.

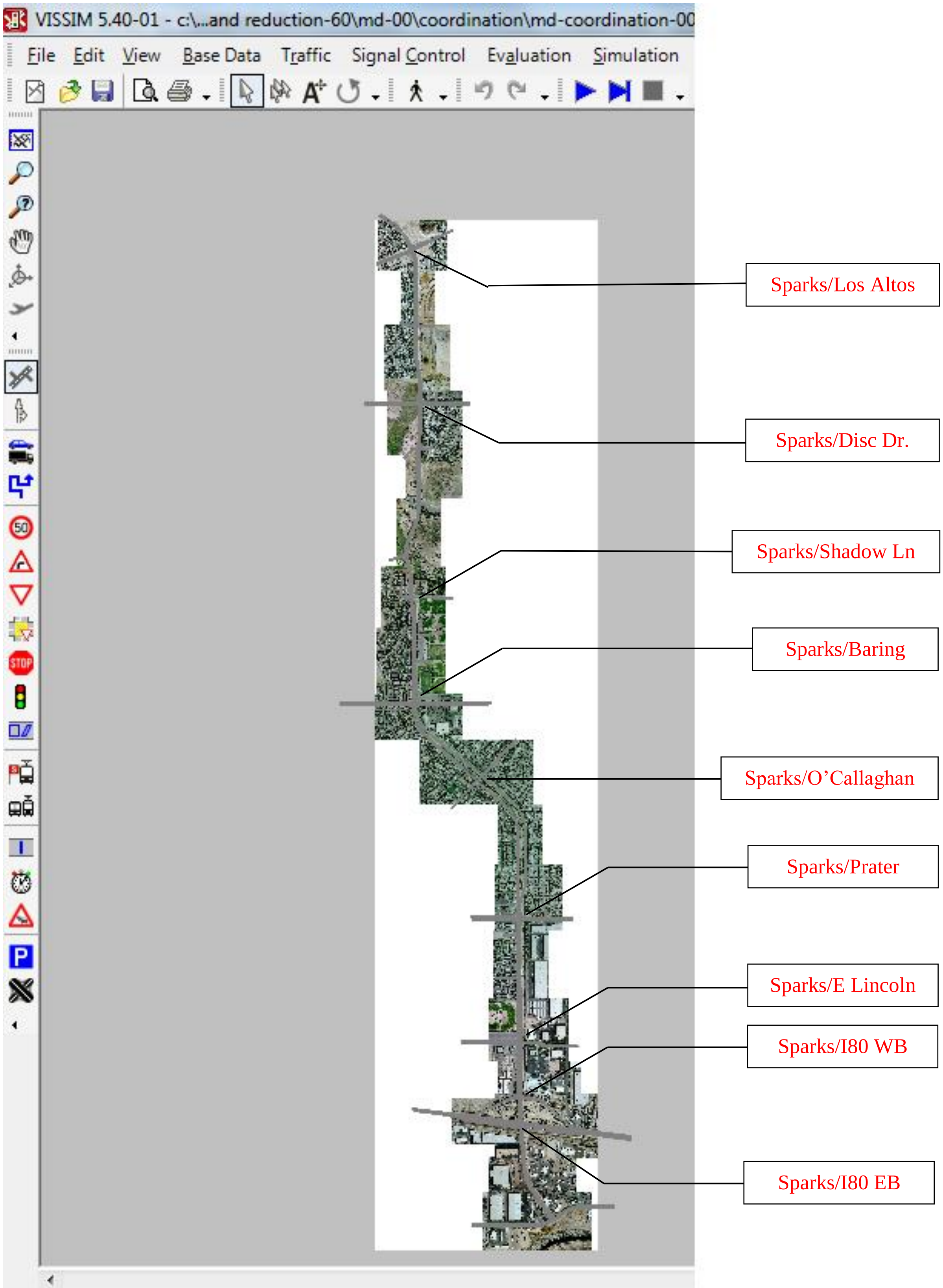

Figure 5.2 Sparks Blvd in VISSIM

5.4.2 Simulation Outputs

Travel time, number of stops, and side street delays were considered as the most significant MOEs. Therefore, the simulation model reported these MOEs for each demand scenario.

- **Travel Time**

Arterial travel times under coordinated and uncoordinated operations were compared in order to judge if coordination was beneficial. Figures 5.3 and 5.4 show the northbound and southbound travel times for both coordinated and uncoordinated patterns.

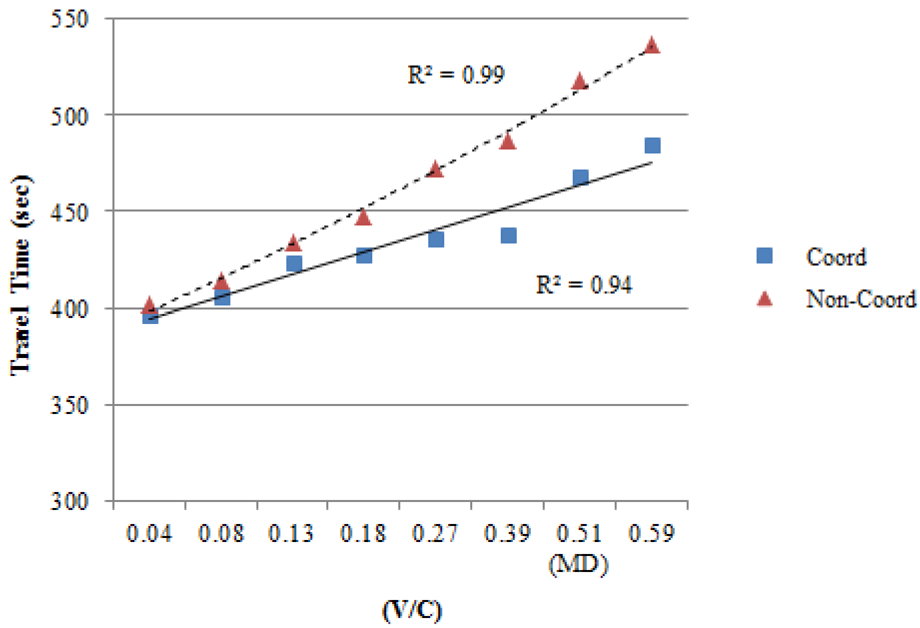

Figure 5.3 Northbound Travel Time: Coordination vs. Non-coordination Plan

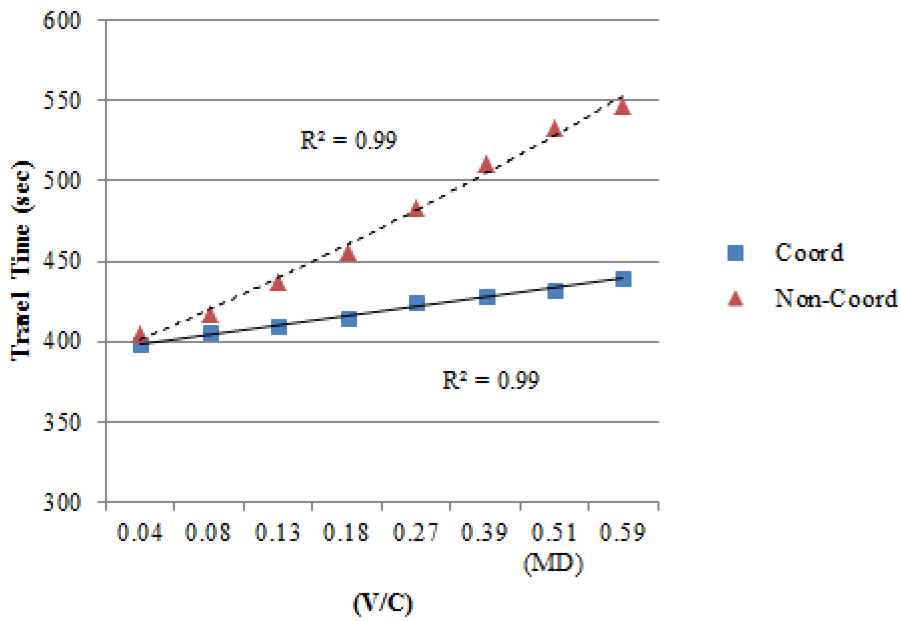

Figure 5.4 Southbound Travel Time: Coordination vs. Non-coordination Plan

It can be seen from these graphs that the travel times are converging as the v/c ratio decreases and the difference increase as volume increases.

- **Number of Stops**

One of the major benefits from signal coordination is a reduction in the percentage of stops. If no major difference exists in the coordinated and uncoordinated operation it could be inferred that coordination is no longer beneficial.

Figures 5.5 and 5.6 illustrate the effect of coordination and non-coordination on percentage of stops along Sparks Blvd.

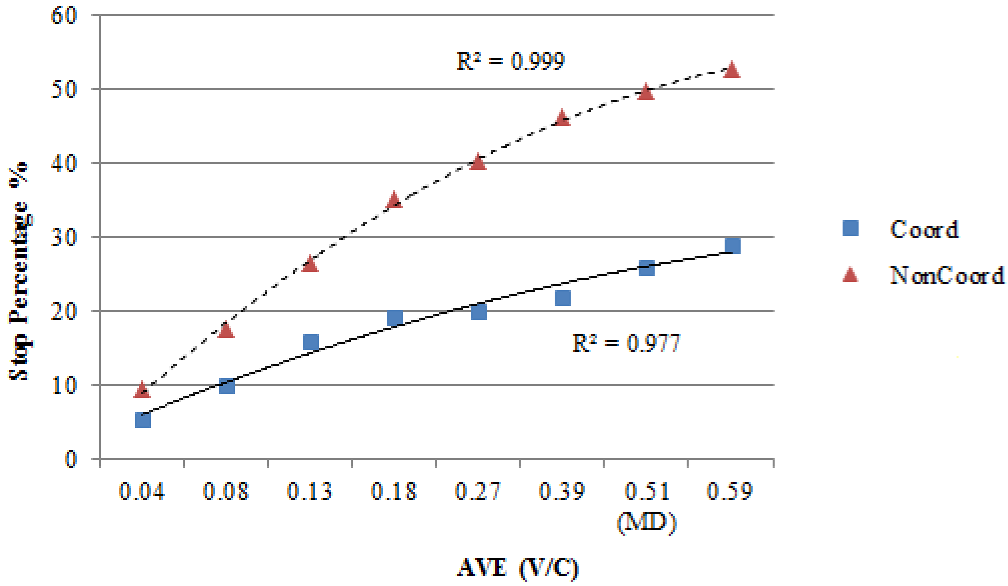

Figure 5.5 Percentage of Stops for NB direction: Coordination vs. Non-coordination Plan

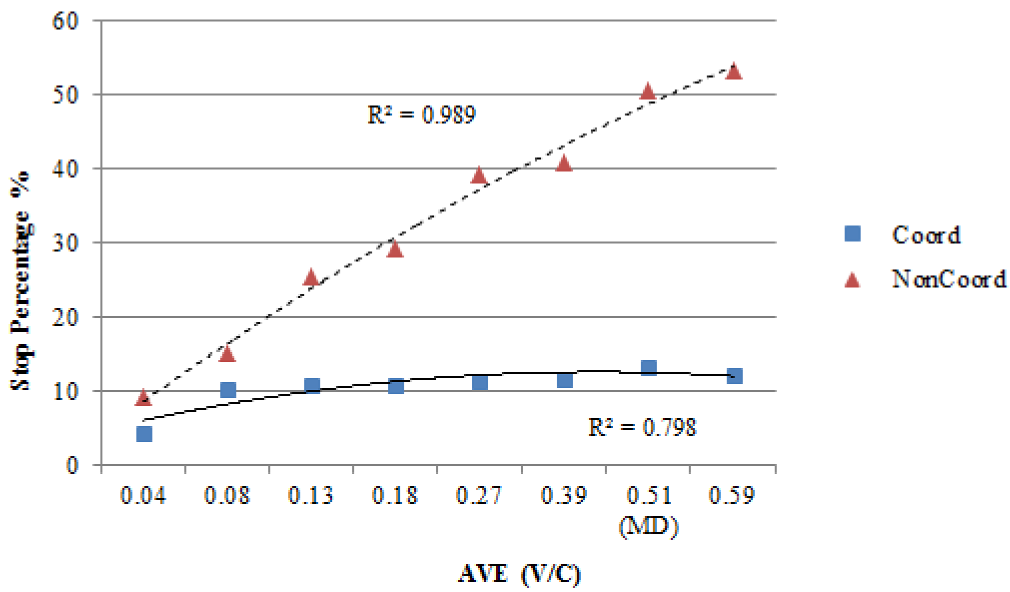

Figure 5.6 Percentage of Stops for SB direction: Coordination vs. Non-coordination Plan

From Figures 5.5 and 5.6 it can be concluded that the rate of increase in the percentage of stops for the coordinated plan is very low or even zero for a well-coordinated direction. However, this rate is very high for a non-coordination plan. Additionally, the percentage of stops in a non-coordination operation becomes constant as the volume increases.

- **Side Street Delay**

One of the most important MOEs for evaluating signalized intersection performance is delay time which determines the level of service (LOS). In this study, delay experienced by side street drivers during coordination was a major factor in considering excessive delay to the side street. The delay becomes more influential when side street vehicles wait for a long time and meanwhile they see no vehicles or a very few vehicles on main street which is a main source of public complaining.

Since there are nine intersections in the study area, an attempt was made to provide an index representing the side street delay time at all intersections. The process is summarized below:

$$d^k = \frac{\sum_i d_i v_i}{\sum_i v_i} \tag{21}$$

$$d(a) = \frac{\sum_k d^k . v^k}{\sum_k v^k} \tag{22}$$

$$D_C = \sum_{a=1}^{9} d_C(a) \tag{23}$$

$$D_{NC} = \sum_{a=1}^{9} d_{NC}(a) \tag{24}$$

Figure 5.7 demonstrates total side street delay for both the coordination and non-coordination plan.

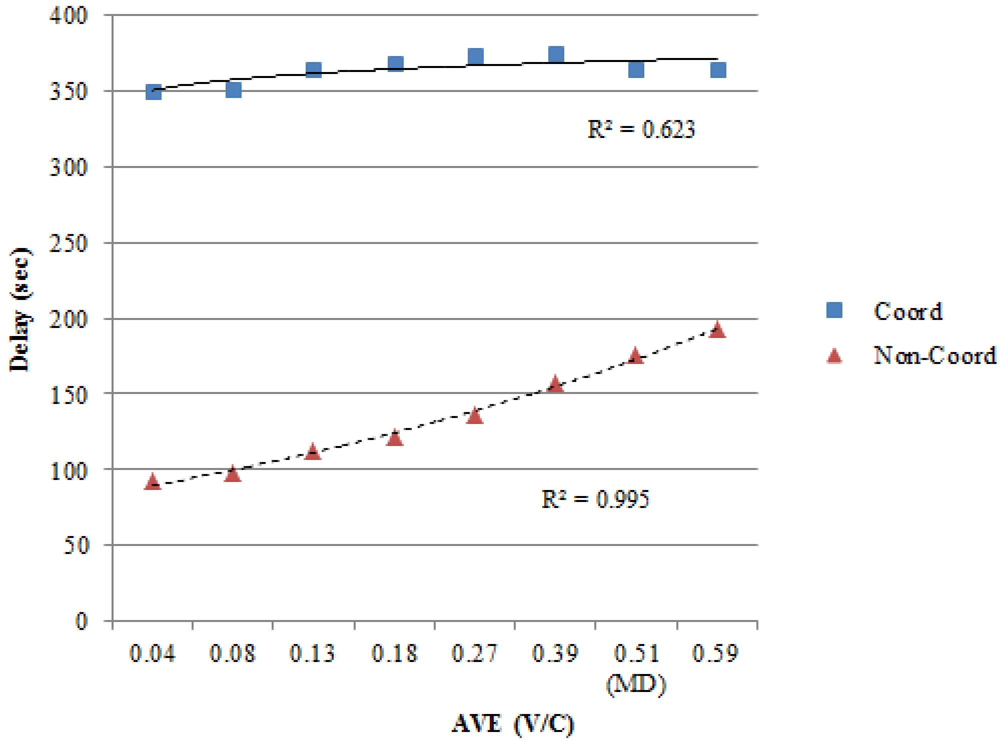

Figure 5.7 Total Side Street Delay: Coordination vs. Non-coordination Plan

5.5 Probabilistic Model and Guideline Validation

In order to assess the workability of the developed probabilistic model and the developed guideline, making comparisons between the simulation outputs and model outputs is necessary. Since, the developed model is supposed to predict the number of stops along non-coordinated arterials, a quantile-quantile plot (QQ-Plot) in terms of number of stops is provided to envision the fitness of the developed model with simulation outputs. Figures 5.8 and 5.9 depict the QQ-Plot regarding the HCM and Synchro methodologies for calculating g/c ratio.

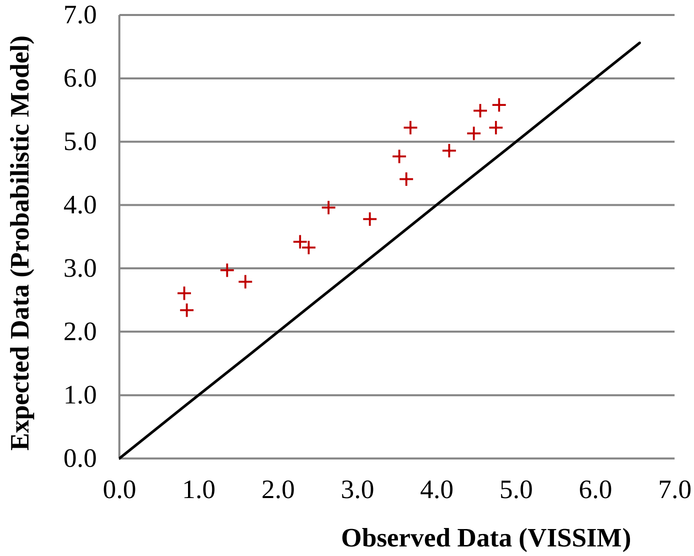

Figure 5.8 QQ-Plot for Number of Stops: HCM Method

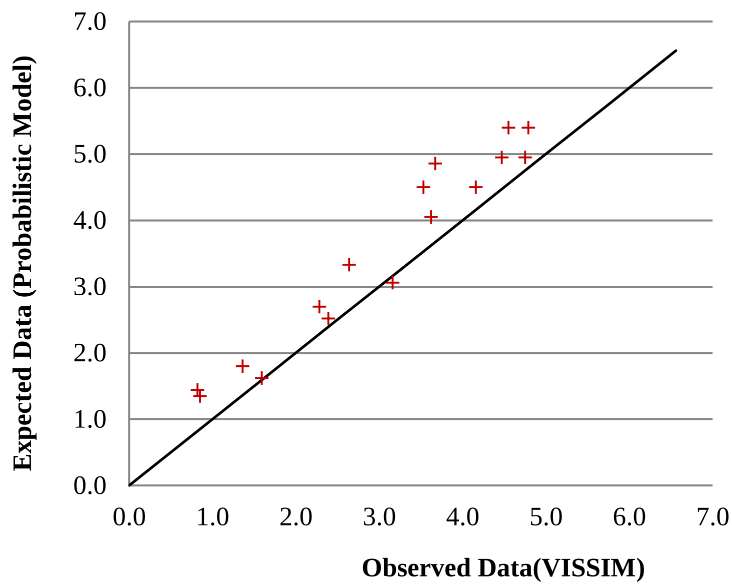

Figure 5.9 QQ-Plot for Number of Stops: Synchro Method

Both figures show that there is a good fitness between the developed model and VISSIM outputs. Nevertheless, the developed model using the Synchro methodology for computing g/C ratio has a better fitness. Additionally, both graphs show that the relationship between the developed model and VISSIM outputs is linear. Table 5.4 provides the correlation matrix between the probabilistic model and VISSIM outputs.

Table 5.4 Correlation Matrix

	VISSIM	Probability Model HCM method	Probability Model Synchro method
VISSIM	1.00	0.960	0.965
Probability Model HCM method	0.960	1.00	0.997
Probability Model Synchro method	0.965	0.997	1.00

The expected number of stops from the probabilistic model outnumbers the number of stops from the VISSIM model. Therefore, linear regression is applied to find the relationship between the probabilistic model and VISSIM output which is presented in general form, below:

$$(No_{Stops})^{VISSIM} = \alpha.(No_{Stops})^{Probabilistic\ Model} \tag{25}$$

Since having the consent constant is not logical here, it is assumed the constant is 0.00 then the regression model depending on the method of computing g/C ratio would be:

Table 5.5 Regression Results

	HCM Methodology			**Synchro Methodology**		
	Coef.	**Std. error**	***t*-Stat**	**Coef.**	**Std. error**	***t*-Stat**
Const.	0.00	--	--	0.00	--	--
α	0.768	0.035	21.790	0.868	0.021	41.687
R^2		0.969			0.991	

In regards to Table 5.5, it can be concluded that the probabilistic model with less than 5% error predicts the number of stops from 15% up to 25 % greater than VISSIM, depending on the method of computing g/c.

The analysis above shows that the performance of the probabilistic model is acceptable regardless of which method for computing the g/C ratio is used. However, Synchro methodology seems to be more promising than the HCM method.

- **Guideline Validation**

In order to validate the developed guideline, two factors were taken into consideration simultaneously: (1) level of improvement (reduction) in travel time due to the coordination plan and, (2) additional side street delays resulting from the coordination plan. Each of these factors was calculated by applying the following equations:

$$\Delta T = |T_{NC} - T_C| \tag{26}$$

$$\Delta D = |D_{NC} - D_C| \tag{27}$$

In above equations, T_{NC} and T_C are travel times along Sparks Blvd during the coordination plan and non-coordination plan, respectively. D_{NC} and D_C are considered the total side street delay times during the coordination plan and non-coordination plan, respectively.

$$T_C = T_C^{NB} + T_C^{SB} \tag{28}$$

$$T_{NC} = T_{NC}^{NB} + T_{NC}^{SB} \tag{29}$$

For each demand scenario both ΔT and ΔD have been computed and the results are depicted in Figure 5.10 to show their variations and interactions. In this figure it is assumed that every 1 sec improvement in travel time is tantamount to every 1.5 sec improvement in total side street delay.

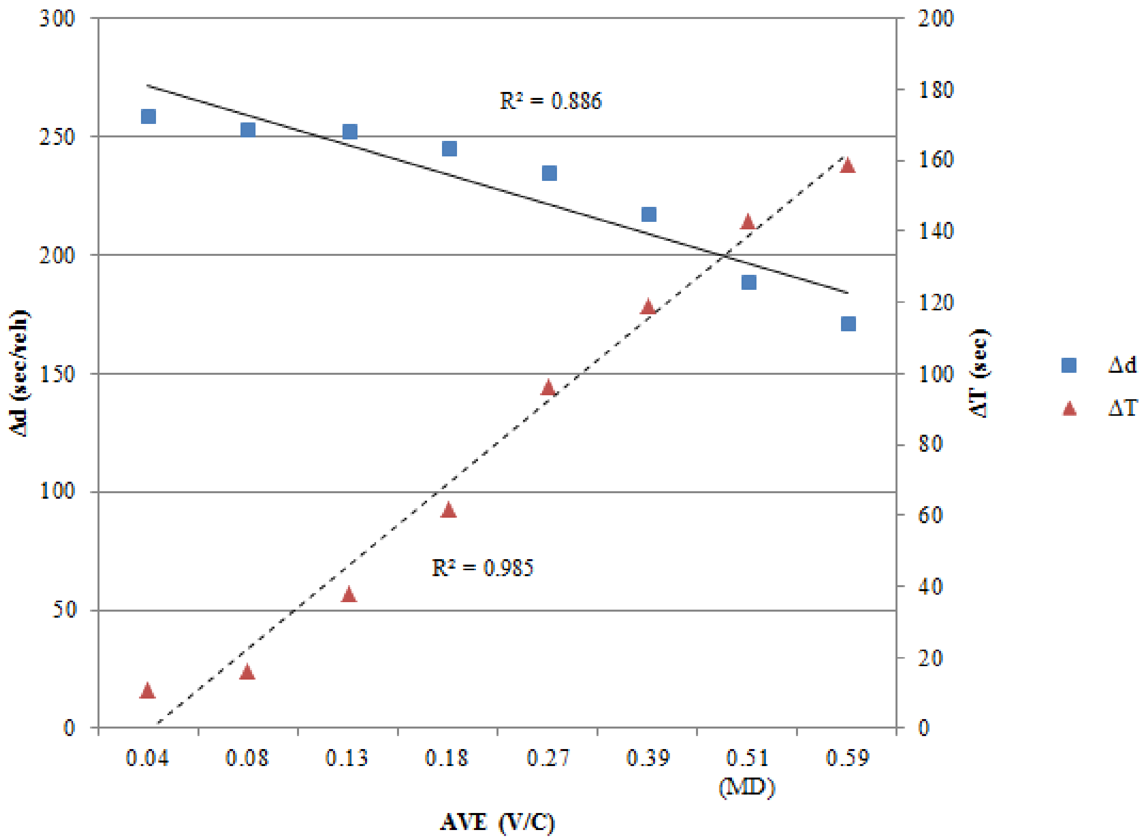

Figure 5.10 Travel Time Improvement vs. Excessive Side Street Delay; 1.0 to 1.5

Since an optimal value for both Δd and ΔT does not exist, the intersection of these two lines is considered a turning point. This idea simply comes from the fact that a poor coordination plan results in a larger Δd and smaller ΔT. On the contrary, a good coordination plan results in a smaller Δd and larger ΔT. Therefore, from Figure 5.10 it could be inferred that from a v/c ratio of 0.00 to 0.42 it is advantageous to run signals free and above a v/c ratio of 0.42 it is advantageous to coordinate signals. Theses v/c values are true for Sparks Blvd, but for other arterials these values may change.

For the validation of the developed guideline, the boundaries of the guideline are embedded in Figure 5.10 to seek at which points boundaries intersect the v/c ratios thus, Figure 5.11 is provided.

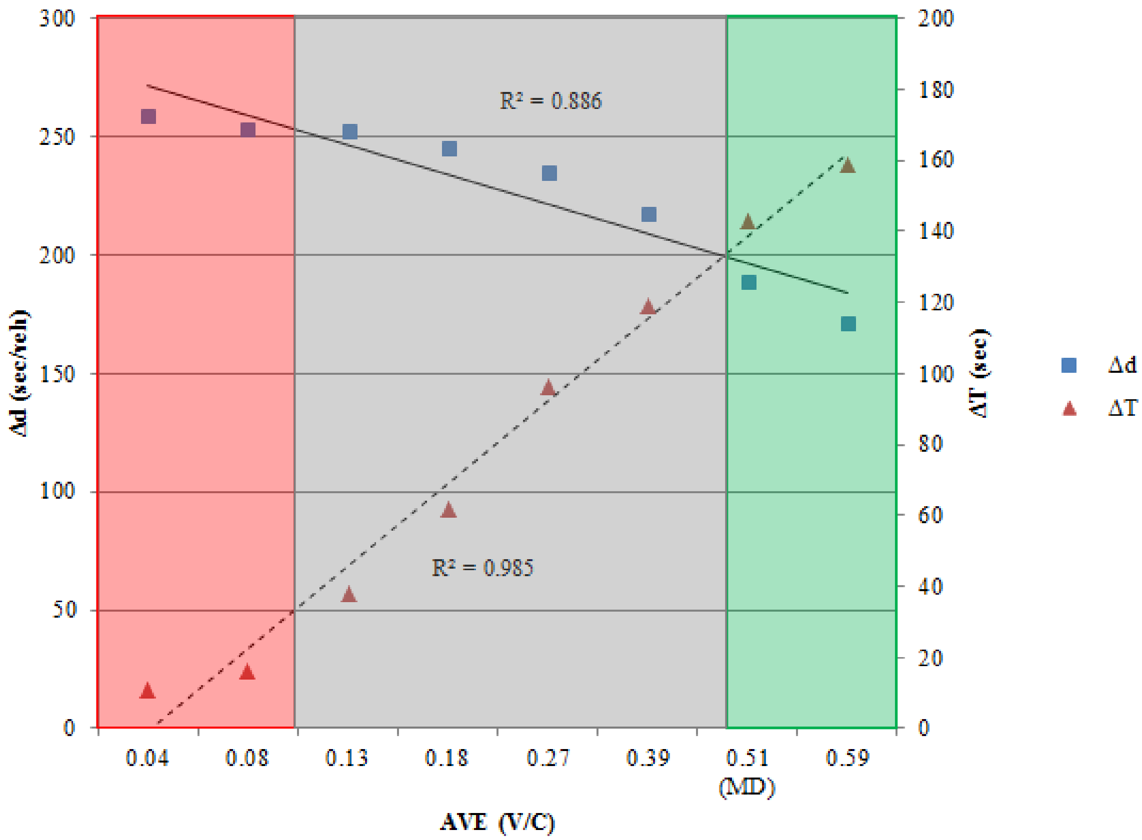

Figure 5.11 Developed Guideline vs. Travel time and side street delay 1.0 to 1.5

By reviewing the above figure, it could be concluded that the developed guideline works well, since 50% of stops happen on the right side of the balance point and 20% occur on the left side of the balance point.

It is obvious that by changing the scale of the vertical axis, the balance point shifts either to the right or to the left depending on the applied scale. Therefore, in another attempt and by the aim of thorough analysis, it is assumed that every 1 sec improvement in travel time is tantamount to 1 sec improvement in side street delay time. Figure 5.12 depicts how ΔT and Δd are interacting in the defined new scale.

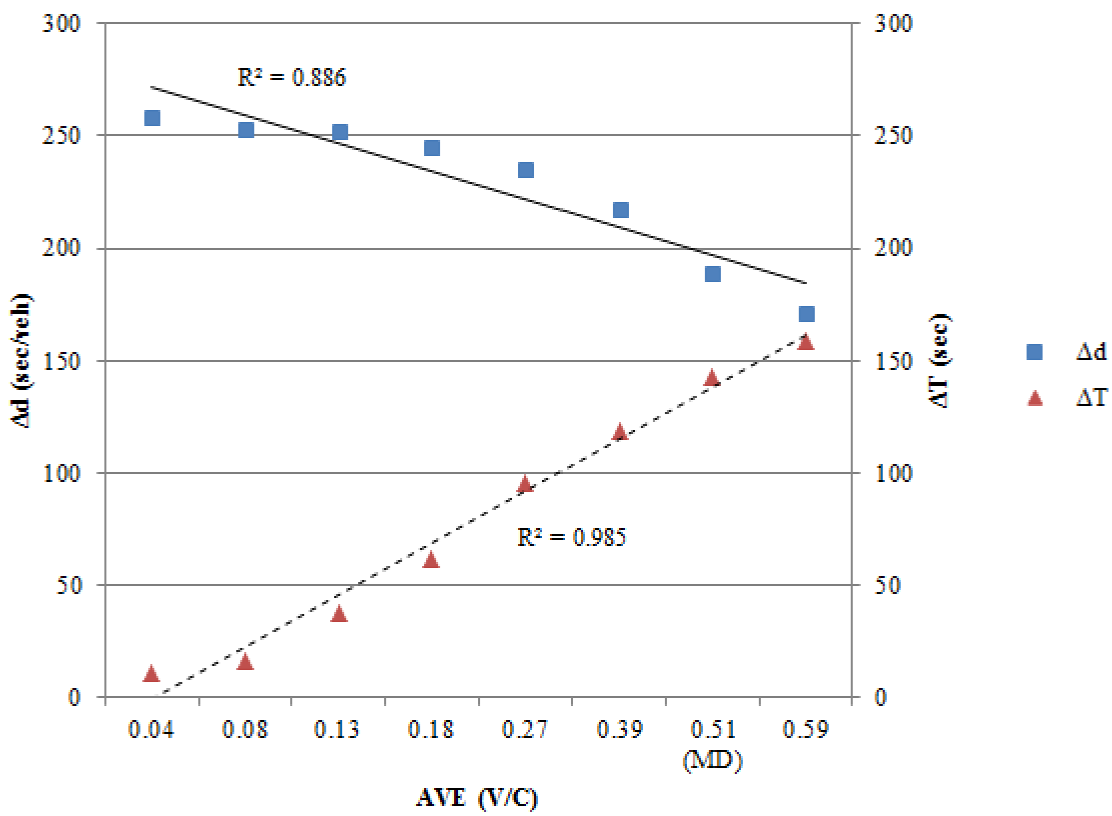

Figure 5.12 Travel Time Improvement vs. Excessive Side Street Delay; 1.0 to 1.0

Considering the new scale it is expected that the two lines (ΔT and Δd) are intersecting at the v/c ratio of 0.63. Figure 5.13 shows the continuation of the ΔT and Δd so as to determine where they are intersecting. In addition, the projection of developed guideline is embedded in Figure 5.13. In this figure the intersection of two lines (balance point) occurs on the right side of the 50% of number of stops point but in a close proximity of each other.

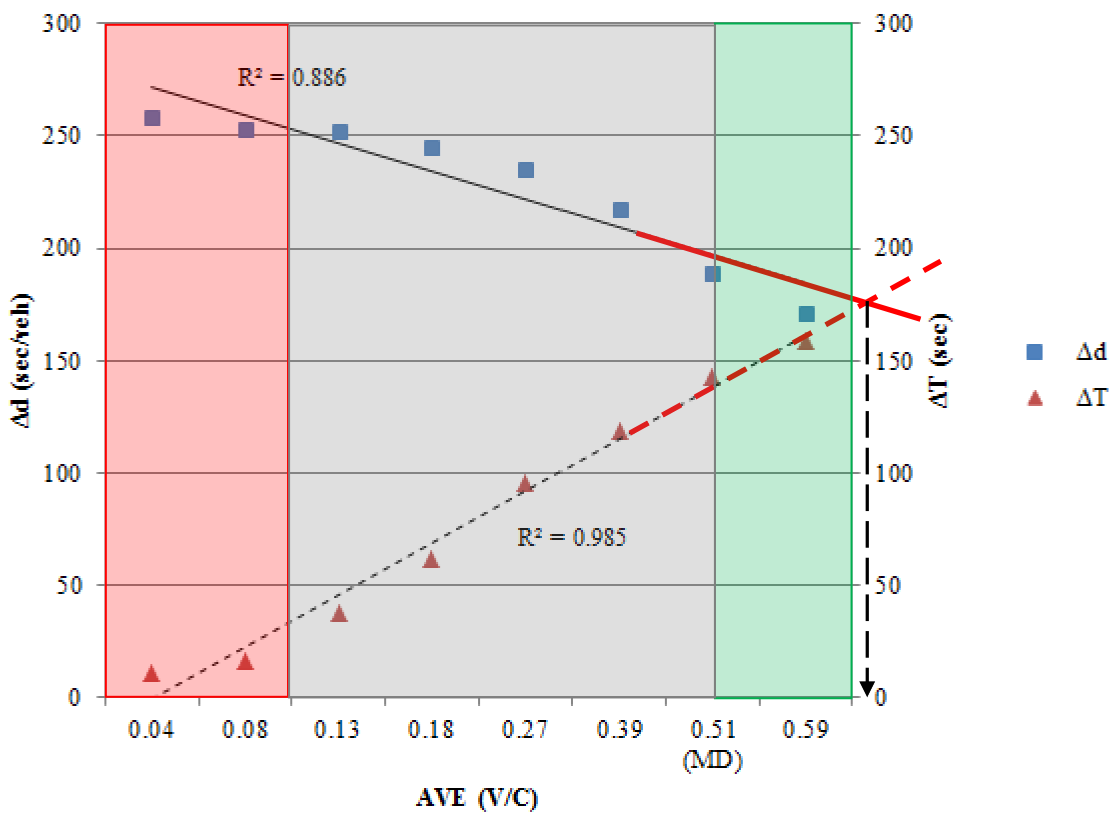

Figure 5.13 Developed Guideline vs. Travel Time and Side Street Delay; 1.0 to 1.0

It is concluded that the scale of ΔT and Δd has an impact on the balance point and the associated v/c ratio. Choosing an appropriate scale demands an engineering judgment or even further research and study.

5.6 Summary and Conclusions

This section focused on simulation outputs with the aim of validating the probabilistic model and the developed signal timing guideline. Sparks Blvd, one of the most important arterials in the Reno-Sparks area was chosen as a case study and was simulated in VISSIM. For the aim of this study, two timing plans, one for the coordination and one for the non-coordination operation, were generated by providing various demand scenarios.

The simulation results show that the developed probabilistic model is highly reliable since there was a strong correlation between the VISSIM outputs and model predictions in terms of number of stops. For the aim of validating the guideline, the level of travel time improvement and the amount of increase in side street delay resulting from coordination plan, were considered. The results confirmed that the proposed guideline is applicable and operational in regard to main street travel time improvement and excessive side street delay.

6. SUMMARY AND CONCLUSIONS

The focus of this study was to assess signal coordination strategies under varying traffic demands. A practical signal timing guideline was developed using number of stops as a threshold. The number of stops can be predicted using a probabilistic model developed in this research. The probabilistic model is a function of the number of intersections and the actuated green-to-cycle g/C ratio. The main advantage of using g/C is that it indirectly links to the volume-to-capacity (v/c) ratio which reflects the traffic demands. The model was validated using VISSIM based on real signalized arterial.

The guideline for determining the conditions when an arterial coordination should execute is as following:

- If the percentage of stops along a non-coordinated arterial exceeds 50, coordination is recommended for the arterial.
- If the percentage of stops along a non-coordinated arterial is 20 or lower, an actuated operation is recommended for the arterial.
- If the percentage of stops falls between 20 and 50, an engineering judgment should be applied to decide whether to coordination is necessary.

Finally, case study was conducted to demonstrate the application of the guideline. The same Sparks Blvd. arterial was used in the case study but with various traffic demand conditions. The case study confirmed the practicality of guideline.

REFERNCES

1. Koonce, P., L. Rodegerdts, K. Lee, S. Quayle, S. Beaird, C. Braud, J. Bonneson, P. Tarnoff, and T. Urbanik. *Traffic Signal Timing Manual*. Publication FHWA-HOP-08-024. FHWA, U.S. Department of Transportation, 2008.

2. *Manual on Uniform Traffic Control Devices for Street and Highways*. Publication FHWA. FHWA, U.S. Department of Transportation, 2009.

3. Henry RD. *Signal Timing on a Shoestring*. Publication FHWA-HOP-07-006. FHWA, U.S. Department of Transportation, 2005.

4. Bhattachrya, P. *Methodology to Optimize for Pedestrian Delay and Vehicular Delay in a Signal Network*. M.Sc. thesis, University of Missouri-Columbia, USA, 2004.

5. *Traffic Control Systems Handbook*. Publication FHWA-HOP-06-006. FHWA, U.S. Department of Transportation, 2005.

6. Chang, E. C-P., and C.J. Messer. *Warrants for Interconnection of Isolated Traffic Signals*. FHWA/TX-86/67+293-1F, U.S. Department of Transportation, 1986.

7. Bhattachrya, P. *Methodology to Optimize for Pedestrian Delay and Vehicular Delay in a Signal Network*. M.Sc. thesis, University of Missouri-Columbia, USA, 2004.

8. Koshi, M. Cycle Time Optimization in Traffic Signal Coordination. *Transpn. Res*. Vol 23A No. 1, pp. 29-34, 1989.

9. Little J.D.C. The Synchronization of Traffic Signals by Mixed-integer Linear Programming. *Opns. Res*. 14, 568-594, 1966.

10. J. A. Hillier and Whiting, Appendix to Glasgow's Experiment in Area Traffic Control. *Traffic. Engng. And Control* 7(9), 569-571, 1966.

11. Gartner N.H., Little J. D. C. and Gabbay H. (1975) Optimization of Traffic Signal Settings by Mixed-integer Linear Programming, Part I: The Network Coordination Problem, Part II: The Network Synchronization Problem. *Transportation Science*. 9, 321-363

12. Pacey, G.M. *Progress of a Bunch of Vehicles Released from Traffic Signal*. Report No. RN/2665. Transportation and Road Research Laboratory, Crowthrone, Berkshire, United Kingdom, 1956.

13. Hillier, J. A., and R. Rothery. The Synchronization of Traffic Signals for Minimum Delay. *Transportation Science*, Vol. 1, 1967, pp. 81-94.

14. Robertson, D.I. *Transyt: A Traffic Network Study Tool*. Report No. LR 253. Road Research Laboratory, Crowthorne, Berkshire, United Kingdom, 1969.

15. Manar, A., and K.G. Baas. Traffic Platoon Dispersion Modeling on Arterial Streets. *Transportation Research Record* 1566.

16. Yagoda, H.N, E.N. Principe, C.E. Vick, and B. Leonard. Subdivision of Signals Systems into Control Areas. *Traffic Engineering*, Vol. 43, No. 12, 1973, pp. 42-45.

17. Hook, D., and A. Albers. Comparison of Alternative Methodologies to Determine Breakpoints in Signal Progression. *Proc., of the ITE Annual Meeting*, Kissimmee, Fla., ITE, Washington, D.C., 1999.

18. *Synchro 6 User Guide*: Trafficware, Albany, CA 2004.

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