FHWA/IN/JTRP-2006/26

Final Report

**EFFECTIVE AND ROBUST COORDINATION
OF TRAFFIC SIGNALS ON ARTERIAL
STREETS**

***VOLUME 1
RESEARCH REPORT***

**Wei Li
Andrew P. Tarko**

January 2007

Final Report

FHWA/IN/JTRP-2006/26

**EFFECTIVE AND ROBUST COORDINATION OF TRAFFIC SIGNALS ON
ARTERIAL STREETS**

Volume 1
Research Report

By

Wei Li
Graduate Research Assistant

and

Andrew P. Tarko
Associate Professor

School of Civil Engineering
Purdue University

Joint Transportation Research Program
Project No. C-36-17WWW
File No. 8-4-75
SPR-2947

Conducted in Cooperation with the
Indiana Department of Transportation and the
U.S. Department of Transportation
Federal Highway Administration

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 Indiana Department of Transportation or the Federal Highway Administration at the time of publication. The report does not constitute a standard, specification, or regulation.

Purdue University
West Lafayette, Indiana
January 2007

1. Report No. FHWA/IN/JTRP-2006/26		2. Government Accession No.		3. Recipient's Catalog No.	
4. Title and Subtitle Effective and Robust Coordination of Traffic Signals on Arterial Streets				5. Report Date January 2007	
				6. Performing Organization Code	
7. Author(s) Wei Li and Andrew P. Tarko				8. Performing Organization Report No. FHWA/IN/JTRP-2006/26	
9. Performing Organization Name and Address Joint Transportation Research Program 1284 Civil Engineering Building Purdue University West Lafayette, IN 47907-1284				10. Work Unit No.	
				11. Contract or Grant No. SPR-2947	
12. Sponsoring Agency Name and Address Indiana Department of Transportation State Office Building 100 North Senate Avenue Indianapolis, IN 46204				13. Type of Report and Period Covered Final Report	
				14. Sponsoring Agency Code	
15. Supplementary Notes Prepared in cooperation with the Indiana Department of Transportation and Federal Highway Administration.					
16. Abstract Many transportation agencies use Synchro and SimTraffic software package to optimize signal coordination. In general, this software package effectively reduces the efforts of traffic systems engineers and improves the quality of the designed coordination. Nevertheless, the initial solutions given by the software packages usually require adjustment, at both the design and implementation stages. The major objective of this research, therefore, is to reduce the time and effort needed in solution tuning and thus to improve the effectiveness and the efficiency of using Synchro/SimTraffic in arterial signal coordination design. The research outcome is expected to help traffic systems engineers reach reasonable signal settings in a shorter time. The first task of this research was to survey INDOT traffic system engineers and document INDOT's criteria for a good coordination plan. The outcome of this task served as the yardstick for other proposed design improvement procedures. The second part of the research focused on the Synchro/SimTraffic-based procedures of optimizing signal settings for urban streets and the practices and methods of using Synchro were scrutinized thoroughly. The outcome of this part of the research is a collection of supplemental guidelines for software-based selection of signal settings on urban streets. Finally, the robustness of the arterial signal coordination procedure was investigated. In current practice, signals are optimized to traffic volumes that represent a single time interval. In spite of the randomness of traffic, these plans, however, are executed for a long period of time until obvious insufficiencies of the signal timings are noticed and re-timing is necessary. The robustness of several alternative approaches was considered in the same traffic condition settings. The outcomes of these investigations are assembled into the second volume of this report, <u>Guidelines of Design</u> . The guidelines are in accordance with the defined criteria of good signal control along urban streets. The guidelines are not a manual of arterial signal design since every arterial system has its individual problems and requires specific treatment that cannot be reached by a uniform set of procedures. Instead, experiences collected from current traffic systems engineers and lessons learned through research are compiled into the guidelines to help reduce field tuning of the signal settings. The resulting guidelines are practical and they require no extra data beyond the current data collection practice. The guideline can serve as a reference for experienced traffic engineers and as an educational tool for new traffic engineers. The robustness of the signal design procedures was also investigated. Models for extracting the traffic variation pattern from 12 hours of traffic counts were developed based on real data collected at Purdue University. These models were used to generate reasonable traffic inputs to the micro-simulation tool. The robustness of the current signal design procedures was evaluated using these simulated traffic inputs. Several practical alternatives to the current arterial signal optimization were also evaluated using the same inputs. The performances of these methods were compared, and it was concluded that no obvious way to consistently improve the robustness of current signal design procedure is available.					
17. Key Words traffic signals, signal coordination, robust signal control, traffic variability			18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161		
19. Security Classif. (of this report) Unclassified		20. Security Classif. (of this page) Unclassified		21. No. of Pages 91	
				22. Price	

TABLE OF CONTENTS

LIST OF FIGURES	3
LIST OF TABLES	4
1 INTRODUCTION	5
2 DESIGN OF SIGNAL CONTROL SYSTEMS—OVERVIEW	7
2.1 Traffic Representation Model.....	8
2.2 Optimization Objectives.....	11
2.3 Optimum Searching Algorithms.....	12
2.4 Robustness of Solution Performance	13
3 METHODOLOGY	15
3.1 Current Approach to Arterial Coordination Design.....	15
3.2 Research Procedure	15
4 STUDY OF INDIANA PRACTICE.....	18
4.1 Questionnaire	19
4.2 Roundtable	21
4.3 Summary of results.....	21
4.3.1 INDOT criteria of good coordination plan.....	21
4.3.2 Design Issues and Practices Reported.....	22
5 ANALYSIS OF SELECTED COMPONENTS OF SYNCHRO/SIMTRAFFIC MODEL	28
5.1 Speed Model of SimTraffic	28
5.1.1 Cruise Speed.....	29
5.1.2 Acceleration Model	35
5.2 Saturation Flow Rate	38
6 MODELING TRAFFIC VARIATIONS	43
6.1 Motivation	43
6.2 Data description.....	45
6.3 Day-to-Day Variation Model.....	50
6.3.1 Gaussian Model.....	50
6.3.2 Poisson Model.....	51
6.3.3 Negative Binomial Model	52
6.4 Within Day Traffic Profile Model	55
6.4.1 Moving Average	55
6.4.2 Frequency Method	57
6.4.3 Polynomial Smoothing.....	59
6.4.4 Exponential Polynomial Smoothing.....	61
6.4.5 Weighted Polynomial Smoothing.....	62
6.4.6 Constrained Polynomial Smoothing.....	63
6.5 Integrated Traffic Model	68
7 ROBUSTNESS EVALUATION OF ALTERNATIVE PROCEDURES	71
7.1 Alternative procedures	71
7.2 Evaluated Systems.....	74
7.2.1 AM of US-52 at Cumberland Avenue	75
7.2.2 Midday of SR-28 at SR-39.....	76
7.2.3 PM of US-36 at SR-39	76
7.3 Results	77
7.3.1 AM of US-52 at Cumberland Avenue	77
7.3.2 Midday of SR-28 at SR-39.....	79
7.3.3 PM of US-36 at SR-39	81
7.3.4 Summary of Results	84
8 CONCLUSIONS	86
LIST OF REFERENCES	88

LIST OF FIGURES

Figure 5-1	Sample system to verify cruise speed model	30
Figure 5-2	Setting speed factors	31
Figure 5-3	Setting link speed.....	32
Figure 5-4	Vehicle parameters setting	33
Figure 5-5	Report option settings	34
Figure 5-6	Speed-time profiles	36
Figure 5-7	Verify Acceleration Model of SimTraffic	37
Figure 5-8	Adjust headway factors of external links.....	40
Figure 5-9	Adjust headway factors for internal movements	40
Figure 5-10	System for experiment of Saturation Flow Rate.....	41
Figure 5-11	Simulated saturation flow rate against Headway Factor.....	42
Figure 6-1	Location of data collection.....	45
Figure 6-2	15-min traffic counts (Apr 14—May 4, 2000).....	47
Figure 6-3	15-minute traffic counts from Apr 17—Apr 23, 2000.....	48
Figure 6-4	15-minute traffic counts from Apr 24—May 1, 2000.....	49
Figure 6-5	Sample variance vs. sample mean	51
Figure 6-6	Variance-Mean observations of southbound and northbound traffic	52
Figure 6-7	Moving Average Smoothing Method.....	56
Figure 6-8	Observed traffic of Apr 17, 2000	57
Figure 6-9	Frequency distribution of the counts of Apr 17, 2000	58
Figure 6-10	Extracted trend from the counts.....	59
Figure 6-11	Polynomial smoothing of the counts of Apr 17, 2000	60
Figure 6-12	Exponential polynomial smoothing	62
Figure 6-13	Weighted Polynomial Smoothing	63
Figure 6-14	Layout of a simple arterial system.....	64
Figure 6-15	an arterial system on US-52.....	66
Figure 6-16	Constrained Polynomial Smoothing	67
Figure 6-17	Actual 8 days' counts vs. Generated 8 days' counts	69
Figure 6-18	Generated traffic of US52 arterial system	70
Figure 7-1	Configuration of data access options	72
Figure 7-2	SimTraffic Setting of Read Multiple Intervals of Volume.....	73
Figure 7-3	Settings of Multiple Intervals in SimTraffic	74
Figure 7-4	Layout of US-52 at Cumberland Avenue.....	75
Figure 7-5	Layout of SR-28 at SR-39.....	76
Figure 7-6	Layout of US-36 at SR-39	76
Figure 7-7	Comparison of Average Total Stops.....	84
Figure 7-8	Comparison of Average Total Stops.....	85

LIST OF TABLES

Table 5-1	Experiment results of speed model	35
Table 5-2	Recommended Base Saturation Flow Rates for Indiana (Perez-Cartagena and Tarko 2004)	39
Table 5-3	Recommended Capacity Parameters for Indiana (Perez-Cartagena and Tarko 2004)	39
Table 5-4	Recommended Saturated Time Headways for Indiana (Perez-Cartagena and Tarko 2004)	39
Table 6-1	Over-dispersion factor estimation results	53
Table 7-1	Performance of Baseline method of US-52 system	77
Table 7-2	Performance of Time Average method of US-52 system.....	78
Table 7-3	Performance of 110% method of US-52 system.....	78
Table 7-4	Summary of the results of US-52 system	79
Table 7-5	Performance of Baseline method of SR-28 system	79
Table 7-6	Performance of Time Average method of SR-28 system.....	80
Table 7-7	Performance of 110% method of SR-28 system.....	80
Table 7-8	Summary of SR-28 system	81
Table 7-9	Performance of method of US-36 system	81
Table 7-10	Performance of method of US-36 system.....	82
Table 7-11	Performance of method of US-36 system.....	82
Table 7-12	Summary results of US-36 system.....	83
Table 7-13	Summary of Average Total Stops.....	84
Table 7-14	Summary of Average Total Delay	85

1 INTRODUCTION

The quality of the signal coordination is an important contributing factor in urban traffic conditions. Coordinated semi-actuated signals are commonly used on the INDOT-administered arterial streets, and hundreds of thousands of Indiana motorists are affected by the timing of arterial signals every day.

INDOT uses Synchro and SimTraffic software packages(Husch and Albeck 1993-2004) to optimize signal coordination. Synchro is an analytical tool for optimizing signals while SimTraffic is a microscopic simulation tool to test the optimized signal settings in simulated conditions. The software package is used to test a considerable number of signal settings within a relatively short period of time, including the type and sequence of the signal phases, the phase splits (force-off points), the signal offsets, and the background cycle.

In general, these software packages effectively reduce the efforts of traffic systems engineers and improve the quality of the designed coordination. Nevertheless, the initial solutions given by the software packages usually require adjustment, at both the design and implementation stages. Two cases frequently are encountered and reported by INDOT signal systems engineers: (1) the initial solution is not fully in line with the INDOT criteria for a good coordination plan; and (2) substantial additional field-tuning is needed since the actual operational performance of the signalized arterial system deviates a great deal from the performance predicted with the software. Substantial human and monetary resources are consumed in the field-tuning process. Furthermore, field-tuning is both temporally and spatially local (i.e., a traffic systems engineer can only tune up the signal timing settings according to the observed traffic conditions at one intersection during a limited period of time).

The major objective of this research, therefore, is to reduce the time and effort needed in solution tuning and thus to improve the effectiveness and the efficiency of using Synchro/SimTraffic in arterial signal coordination design. The research outcome is expected to help traffic systems engineers reach reasonable signal settings in a shorter time.

The first task of this research was to survey and document INDOT's criteria for a good coordination plan. Clear documentation of these criteria was vital since the difference between the Synchro optimizer's objective function and INDOT's practical benchmark is a major cause of unsatisfactory solutions. This documentation also served as the yardstick for other proposed design improvement procedures.

The second part of the research is focused on the Synchro/SimTraffic-based procedures of optimizing signal settings for urban streets. To adjust the solution of Synchro, advanced knowledge and techniques beyond the routine usage of the software are often necessary. The outcome of this part of the research is a collection of supplemental guidelines for software-based selection of signal settings on urban streets. Important components of the Synchro/SimTraffic model were scrutinized. The current issues of using Synchro/SimTraffic were investigated. Practices and alternative methods of addressing these issues were scrutinized thoroughly.

Finally, the robustness of the arterial signal coordination procedure was investigated. Robustness is herein defined as performance under randomly varied traffic conditions. In designing the arterial signal timings, the traffic data collected in a very limited period of time, usually 12 hours, are used as the input to the optimizing software. In current practice, signals are optimized to traffic volumes that represent a single time interval. In spite of the randomness of traffic, these plans, however, are executed for a long period of time until obvious insufficiencies of the signal timings are noticed and re-timing is necessary. The signal settings (background cycle, force-off points, and offsets) are fixed within such periods while traffic may still vary considerably. This inconsistency between fixed signal settings and varying traffic creates a challenge in coordinating signals. To evaluate the robustness of a timing plan, ideally, one should implement it and record the performance for an extended period of time. This method is obviously unrealistic and budget-prohibitive. Robustness therefore was evaluated in simulation. First, a model for extracting traffic variation patterns from limited observations was developed. Second, this model was used to expand limited traffic data to multiple days of traffic. Third, the performance of the timing plans was recorded in the micro-simulation tool under the generated traffic levels. The robustness of several alternative approaches was considered in the same traffic condition settings.

Chapter 2 reviews the state-of-the-art of designing signal control systems. Chapter 3 presents current approaches to arterial signal design and the research procedure of this study. Chapter 4 shows in detail the investigation of the current Indiana practice of arterial signal design. Chapter 5 demonstrates the research of the characteristics of the Synchro/SimTraffic models. In Chapter 6, the models of day-to-day and within-day traffic variations are developed on the basis of real data. In Chapter 7, these models are utilized to generate multiple days of traffic volumes. In addition, the robustness of several alternative approaches is presented. Finally, Chapter 8 concludes this report by providing general comments and recommendations.

2 DESIGN OF SIGNAL CONTROL

SYSTEMS—OVERVIEW

As introduced in Chapter 1, the major objectives of this research are to improve the effectiveness of Synchro-based arterial signal timing design and to investigate the robustness of different design procedures. In this chapter, an overview of the state-of-the-art of arterial signal timing design is provided.

In this research, the dominant signal control scheme of interest is arterial level semi-actuated control. Traffic signal optimization has been researched for almost 50 years since the seminal work of Webster (Webster 1958). In that research, Webster proposed to allocate the duration of green phases of an isolated intersection so that each movement has the same level of saturation. Since no viable vehicle detection technology was available at that time, the method results in a pre-timed signal with no flexibility allowed. Arterial level semi-actuated control is the marriage of the outcomes of two distinct directions of technological development (Gazis 2002). On one hand, the introduction of coordination enabled the simultaneous optimization of adjacent traffic signals. On the other hand, the vehicle sensor technology made possible traffic responsive control.

In terms of scale, traffic control system can be categorized as: intersection control, arterial control, and network control. The scale of optimization has evolved from isolated intersections to interdependent networks. The signals are working in a coordinated way to allow the vehicles to travel through with minimum delay. Intersection level control tries to optimize the traffic signal operations at the intersection level, considering each intersection as isolated from the others. This scheme is used when a signal is a great distance from other signals. Arterial control tries to coordinate the operations of the signals along an arterial. Smooth progression along the arterial in both directions is the common objective (Husch and Albeck 1993-2004). Network control tries to optimize the signals in a two-dimensional layout when the demands of intersecting movements are comparable. The overall network performance is considered as the objective function.

On the other hand, in terms of traffic responsiveness, the traffic control system can be categorized as: pre-timed, fully actuated, and coordinated-actuated (semi-actuated). Pre-timed signals do not respond to the fluctuation of traffic. All of the timings of the pre-timed signal are fixed regardless of the traffic conditions

Vehicle detection technology has enabled traffic-responsive control mechanisms. Instead of changing green phases rigidly according to the designed timing plans, traffic-responsive controllers adjust the signal timings in response to the real-time traffic variations. Fully-actuated signals use the information provided by the vehicle detectors and allocates the green phases dynamically, depending on the traffic condition. Theoretically speaking, the timings of fully-actuated signals can change without limit based on the demand of the traffic conditions. Coordinated-actuated schemes, sometimes called semi-actuated control, are a mix of the previous two methods. Some of their parameters are predetermined, such as cycle and force-off points; and some other parameters are determined by the actual traffic demand (e.g., green extensions). These signals have bounded flexibility compared with the full flexibility of the fully-actuated signals.

The relevant control scheme of this research is arterial level semi-actuated control. In this scheme, the signals are coordinated along the arterial. They have several pre-determined signal plans which often include parameters such as cycle length, offsets, and splits, and they also respond to the minor variation of traffic. The semi-actuated control logic seeks a reasonable compromise between adjusting signals to varying traffic at each intersection and maintaining signal coordination along the arterial street. In this logic, the traffic phases are actuated at intersections based on vehicle detection by varying between pre-set minimum and maximum lengths. This variation is bound and subordinated to the need of coordination in two ways (Gazis 2002).

- 1 Signal phases must sum up to a so-called “background cycle,” which has the same length at all coordinated intersections. If one phase takes more time, then the other phases have less time remaining in the background cycle.
- 2 A reference signal phase for traffic along the arterial street ends at a specific point in the cycle, called the “offset,” which is designed so that vehicles along the arterial street pass consecutive intersections with minimum delay.

In the following sections, the features of the tool used in optimizing arterial level semi-actuated controlled signal timing are reviewed.

2.1 Traffic Representation Model

A traffic representation model is the basis of every signal optimization tool. It depicts the characteristics of the traffic flow and its reaction to the signal timing settings. Generally speaking, two major models are found in current signal optimization software tools: analytical models and microscopic simulation models. Synchro/SimTraffic provides a seamless environment of using both models. The typical practice is to get a reasonably good initial solution in Synchro and evaluate it in the micro-simulation tool SimTraffic. Based on the performance observed in SimTraffic, adjustments to the initial solution are

made until satisfactory.

Synchro's optimizer uses an analytical model (Husch and Albeck 1993-2004) which is close to that of HCM 2000 (TRB 2000). These models are mostly macroscopic (Prigogine and Herman 1971). The behavior of the traffic is specified by a set of macroscopic equations. For example, in Webster's formulation (Webster 1958), the traffic was assumed to have Poisson arrivals rates and uniform discharging headways. Based on these analytical models, the formula to calculate important measures of effectiveness are deduced. For example, in the HCM 2000 model (TRB 2000), the delay for an arterial intersection is calculated as

$$D = D_1 * PF + D_2 + D_3$$

where,

$$\text{uniform delay } D_1 = 0.5 * C * \frac{(1 - g/C)^2}{[1 - (g/C) * \min(X, 1)]},$$

PF is Progression Factor,

$$\text{Incremental delay } D_2 = 900T [(X - 1) + \sqrt{(X - 1)^2 + \frac{8kIX}{cT}}]$$

residual delay D_3 ,

C is cycle length in seconds,

T is duration of analysis in hours,

g is effective green time in seconds,

X is volume to capacity ratio,

c is capacity in vph,

k is incremental delay factor depending on controller settings,

and I is upstream filtering factor.

In Synchro, the Percentile Delay Method is used in delay calculation. It calculates delays for the 10th, 30th, 50th, 70th, and 90th percentile volumes and takes a volume weighted average. It is reported that for most cases (Husch and Albeck 1993-2004), the delay calculated by Synchro is within "a few seconds" to that of the HCM 2000 method.

The advantage of this traffic representation method is the tractability and ease of computation. With the parameters set, the modern computer can output the performance measures in no time. A huge amount of signal timing scenarios can be evaluated in literally a few seconds.

The disadvantage is that these models are often built on general assumptions that can deviate a great deal from reality. For example, the arrival process can be vastly different from the Poisson process. This model is often used in the optimizing software to quickly evaluate the performance of the system.

Microscopic simulation models depict the behavior of each vehicle, instead of the traffic as a whole, and the interaction patterns between vehicles. SimTraffic (Husch and Albeck 1993-2004) and CORSIM (USDOT 1996) are the most prominent examples of micro-simulation tools. Instead of calculating performance measures using formulas, the micro-simulation tool obtains numerical estimations of the performance measures by running Monte Carlo simulations. It is similar to the experiment method in physical science. The advantage of this model is its closeness to the ground truth and its independence from strong assumptions of traffic process. The disadvantage is the high computational costs. To obtain an estimation of a scenario often take an exorbitant amount of time compared to the analytical models. Furthermore, the time increases exponentially with the size of the system. This model is often used in the evaluation stage of the signal design where better estimation of performance measures is critical.

As introduced above, the software package Synchro/SimTraffic incorporates both models into the designing process. The analytical model is used in Synchro to quickly evaluate the performance measures of different scenarios. The micro-simulation model is used in SimTraffic to provide better estimates of the performance measures.

The Cellular Automata Model was recently introduced into the area of traffic modeling (Nagel and Schreckenberg 1992) (Nagel, Wolf et al. 1998). This group of models is based on the discretization of time and space. It is a relatively new model in this area and the relationship of the CA model and traditional models is investigated by (Daganzo 2004). Several recent studies have found that these models provide important insight into traffic modeling in a convincing way (Gartner and Wagner 2004). In practice, currently the CA model is often considered an alternative or complementary approach to the traditional models. There is relatively little implementation of it in current commercial optimization packages. It is not further investigated in this research since it is not applicable to the practices of Indiana in the near future.

2.2 Optimization Objectives

The first major task of this research is to identify INDOT's criteria for good coordination. It is the premise for all other parts of the study. More importantly, the discrepancy between the traffic engineer's criteria of good coordination and that of the optimization software is a major source of unsatisfactory solutions. The objectives of signal optimization have evolved with time. The earliest goal of traffic control was avoiding conflicts of vehicles (Gazis 2002). Later, the emphasis was shifted unanimously toward maximizing mobility. Although it was acknowledged qualitatively that drivers' comfort of travel was the appropriate objective of optimization (Gazis 2002), the quantitative interpretations of this concept were divergent. In arterial signal optimization, two major types of objectives are most common: maximizing bandwidth and minimizing the disutility of travel.

Maximizing bandwidth and progression is the historically oldest type of arterial signal optimization. This group includes those of (Morgan and Little 1964), (Messer, Whitson et al. 1973), and *Multiband* (Gartner, Assmann et al. 1990). Synchro's Time-Space-Diagram provides the functionality to inspect the bandwidth of timing plans. The advantage of this method is that it is graphically well defined and experienced traffic engineers are able to find reasonably good signal plans with the trial-and-error method. The disadvantages of this objective are twofold: (1) it is deterministic, therefore, it is difficult to visualize the bandwidth under variable traffic and semi-actuated control; and (2) many important details, such as the internal queue information, are not available in this method. Therefore, this concept is largely superseded. It should be noted that the intuition of bandwidth maximization is so convincing that many traffic systems engineers try to manually adjust the solution given by Synchro based on the bandwidth graph in the Time-Space Diagram.

Minimizing disutility of travel is the dominant type of optimization objectives. Time of delay and number of stops are the most widely used measures of disutility. Later, more and more measures are introduced to capture the concerns of traffic planners such as fuel consumption, and environmental costs. Most modern optimization software packages, such as TRANSYT (McTransCenter 1990-2006), SYNCHRO (Husch and Albeck 1993-2004), and SCOOT (Hunt, Robertson et al. 1991) use these measures or a combination of these dis-utilities. Stops and delays are selected partly due to their close relationship to the quality of service and partly because they are easily quantifiable. The disadvantages are: (1) the difficulty of visualization and manual calculation; traffic engineers have to rely on the software packages to obtain these measures; and (2) the relation between drivers' perceived comfort of travel and these disutility measures are not well established.

Other complementary performance measures have been introduced recently, such as the transit priority objectives (Skabardonis 2000), emergency vehicle pre-emption (Nelson and Bullock 2000), and the maximization of travelers' economic welfare (Mannering

1988). These measures are often considered supplementary.

2.3 Optimum Searching Algorithms

The Optimum Searching Algorithm is the way to find the solution when the traffic models and the objective function is well defined. It is a vital part of every signal design package. In general, the optimization algorithms can be categorized into off-line and on-line methods.

Off-line methods are the dominant algorithm in practice. They optimize the signal timings based on history traffic data. The examples include MAXBAND (Little, Kelson et al. 1981), TRANSYT (McTransCenter 1990-2006), PASSER (Chang and Messer 1991), and SYNCHRO (Husch and Albeck 1993-2004). MAXBAND formulates the signal optimization problem as a mixed-integer programming problem and solves it with mathematical programming packages. TRANSYT is comprised of a model used to calculate delays and stops, and an optimization module using "hill-climbing" techniques to minimize a combination of network level delays and stops. SYNCHRO has a similar structure. It calculates delays and stops following the method of HCM 2000 closely and enumerates through almost all possible solution combinations. The recommended solution of SYNCHRO is the one with the minimum value of "Performance Index", which is also a linear combination of delays and stops.

The advantages of these off-line methods are the relatively low cost of computation. Since the traffic demands are given in these methods, the performance measures can be calculated in a short period of time. A reasonably good solution can be obtained quickly. However, the performance of these solutions under the randomly fluctuating traffic volume levels may deteriorate significantly. As a result, regular retiming of traffic signals is inevitable due to the growth and decline of traffic demands (Sunkari 2004).

On-line methods optimize the signal timings based on real time traffic data. They were enabled by the usage of computerized controllers and advanced vehicle detectors. The examples include SCOOT (Hunt, Robertson et al.) and OPAC (Gartner 1983). The SCOOT system can be considered the enhanced TRANSYT with vehicle actuation. It utilizes measurements of traffic volume and occupancy data. The central computer employs a similar optimization procedure of TRANSYT. The real-time traffic data, rather than the historical traffic data, are input into the model. SCOOT adjusts signal timing parameters in small steps but frequent intervals. All the parameters of the signal plans float gradually and constantly according to the traffic variations. Theoretically speaking, the system doesn't need re-timing since the signal timing parameters evolve autonomously with the variation of traffic demand.

OPAC represents another type of on-line optimization method (Gartner 1983). It knows no concept of cycle, offsets, or splits, which are key parameters for regular timing plans. It repeatedly predicts the traffic conditions into the future and determines the optimal phase changing time. The core of OPAC is the dynamic programming algorithm. The objective function is the total travel time spent in the system by all vehicles in the foreseen time horizon. The disadvantage of these methods, however, is that the exponential time complexity of the algorithm often makes it impossible to find the global optimal solution in real time.

In the U.S., these on-line adaptive control systems have not gained popularity partly because the performance evaluations of several experimental systems were not satisfactory. Traffic engineers are not convinced of their practical superiority despite the theoretical strengths of these methods. Several deficiencies were identified by a recent research (Shelby 2004) and it was concluded that at least a 250-fold increase in computational burden will be incurred to overcome these deficiencies.

Genetic Algorithms are search algorithms that mimic the mechanics of natural evolution (Goldberg 1989). It has been proposed to solve the extremely difficult problems in traffic signal optimization (Foy, Benekohal et al. 1993; Park, Messer et al. 1999; Park, Messer et al. 2000). The GA optimized plans showed prospects when applied in isolated intersections but have not been able to reach consistently better solutions than Synchro on arterials (Kamarajugadda and Park 2003).

This research is aimed at aiding the practice of INDOT traffic systems engineers. Therefore, the method of utilizing Synchro's offline algorithm to obtain reasonable solutions is investigated here at length.

2.4 Robustness of Solution Performance

A natural result of the offline optimization method of Synchro is that the signal plan is optimized to a static set of traffic conditions while it is later applied to constantly varying traffic conditions. The robustness of the solution (i.e. the performance under fluctuating conditions) is thus an unavoidable measure of travelers' comfort.

Synchro partly addresses this issue by using the Percentile Delay Method (Husch and Albeck 1993-2004). It assumes that the arrival process is Poisson and calculates the volume level according to the distribution. The delay output is a weighted combination of delays calculated at the 10th, 30th, 50th, 70th, and 90th percentile volumes.

Despite its obvious importance, the robustness has received only insufficient attention. Only recently, a study (Kamarajugadda and Park 2003) suggested since delay is a random

variable, instead of using the mean delay as the performance measure, more distribution factors (e.g. variance of delay) should also be estimated. An analytical method of estimating the delay variance was derived based on the assumption of the distribution of relevant variables (e.g. traffic volume). The estimation was then incorporated in the later optimization procedure. Genetic Algorithm was used in solution searching. Nonetheless, no consistently better solution was found for arterial systems. Another research (Yin, Madanat et al. 2005) investigated the robust optimization methods of road network planning under uncertain demand. They found that the performance variance may be reduced at the cost of a lower average level of service.

None of the above studies explicitly models the traffic variability from 12 hours of available data. The performance evaluation was done by numerical calculation or in CORSIM, which is far from the practice of INDOT engineers. Therefore, a model of extracting variability information from available data and evaluation procedure in SimTraffic is needed.

3 METHODOLOGY

The objective of this research, as described in Chapter1, is to aid traffic systems engineers in reaching robust solution with the software package Synchro/SimTraffic in a short period of time. In this chapter, the current practice of arterial signal coordination design is first described. Then, the research procedure to accomplish the research objective will be described.

3.1 Current Approach to Arterial Coordination Design

In Indiana, coordinated signals are designed by traffic systems engineers in the following steps.

1. A travel time study is conducted under existing signal timing conditions.
2. Data inputs for Synchro are assembled including the geometry and volume. Typically, traffic volumes are measured for 12 hours or extracted from recent measurements.
3. The traffic volumes and existing, or designed, arterial geometry are coded into Synchro.
4. A tentative signal coordination plan is obtained with the optimizer of Synchro.
5. The signal plan performance is evaluated in both Synchro and SimTraffic. If potential improvements are identified, return to step 4 and tune up the plan.
6. The fine-tuned signal settings are implemented on the arterial.
7. Observations of signal conditions and adjustments to the settings are made to eliminate conspicuous traffic problems, such as queue spillbacks or poor offsets.
8. A travel time study under the new signal coordination plan is conducted to verify and document the improvements.

In practice, it is noticed that a great amount of time is consumed in steps 4, 5, and 7. The discrepancy between the engineer's view of coordination and the objective function of Synchro is one of the major causes of repetitive iterations between steps 4 and 5. The field-tuning efforts in step 7 are largely due to the lack of calibration of SimTraffic.

3.2 Research Procedure

To address the problems stated above, the following topics were investigated.

1 Study of the current procedures of setting signal coordination in Indiana

The criteria for good signal control in Indiana were identified and documented through meetings, surveys, and roundtable discussions with INDOT traffic systems engineers. These well documented criteria became the yardstick for all further research.

The issues and practice of using Synchro/SimTraffic in optimizing traffic signals were collected, classified, and documented, and the survey and the follow-up roundtable discussions with the traffic systems engineers of INDOT were conducted. The practices to address the issues were later investigated by analysis and simulation according to the criteria documented. The investigated methods are expected to reduce the time spent in iterative tuning between steps 4 and 5 stated in Section 3.1.

2 Analyze the Synchro/SimTraffic method

The traffic and control representation in Synchro and SimTraffic, the optimization criteria, and the optimization algorithm were scrutinized. Specific characteristics of the software package, particularly the calibration of important local traffic parameters, were investigated, especially for those which are not expounded in detail in the software help manual.

Methods for making adjustments to reflect certain concerns were investigated, explored, and documented. These methods are consistent with the INDOT control objectives for urban arterials. The outcome of this part of the research is expected to reduce the time spent in field tuning due to the inappropriate default values in SimTraffic.

3 Study traffic variability within a coordination period

Traffic variability patterns were analyzed in detail. The Negative Binomial Model was selected as the day-to-day variation model and the Weighted Exponential Polynomial Smoothing method was selected as the model for extracting a daily traffic profile. A scheme to generate multiple days of traffic using only 12 hours of traffic counts was devised on the basis of these models.

4 Evaluate Robustness of Alternative Signal Design Procedures

Three alternative procedures, including the default procedure used by Synchro, were evaluated under the same set of traffic conditions generated by the model developed previously. Traffic volumes for multiple days and hours were used in the evaluation. The three methods were applied in three real systems. The performance measures were recorded and compared.

5 Develop the guidelines

Guidelines were developed to incorporate all the research outcomes and are applicable to the practice of INDOT traffic systems engineers. These guidelines are considered not definite but supplemental. They can serve as reference for the experienced engineers and educational tool for the inexperienced ones.

4 STUDY OF INDIANA PRACTICE

Signal coordination is a major control instrument of INDOT. As explained before, the discrepancy between INDOT practice and Synchro's optimization procedure is a major source of unsatisfactory initial Synchro solutions. Therefore, understanding the practices of the traffic systems engineers was vital for this research. First, in practice, traffic engineers have far more concerns than merely the time of delay and the number of stops. Formal documentation of practical criteria for good coordination became the premise for further study. Second, traffic engineers have devised myriad techniques to deal with the practical issues not covered by the software default procedures. Proper classification and documentation of the issues and practices is one of the major objectives of this research. Third, some unsolved issues were investigated in this research to aid traffic systems engineers in future design.

Several methods were utilized to study Indiana's practices, including reviewing design documentation of real systems, informal discussion with the engineers, and regular correspondence and inquiries. In particular, a survey study was conducted among INDOT traffic systems engineers to collect the issues in arterial coordination signal design, mainly pertaining to the usage of Synchro and SimTraffic, and seek their experience with handling these issues. The survey was conducted in the following steps:

1. The Indiana arterial signal coordination design procedures were consulted in a pilot meeting with INDOT traffic systems engineers.
2. A questionnaire was devised and distributed to all the INDOT traffic systems engineers based on the knowledge learned in the preliminary stage.
3. All the questionnaire feedback was categorized and presented in a roundtable discussion with the INDOT traffic engineers.
4. Comments and issues raised in the roundtable were documented.
5. Finally, their practices and experiences were investigated and combined into this report.

Details of the survey and the summary of the results are presented in the sections that follow.

4.1 Questionnaire

In the first stage of the survey, INDOT engineers pointed out that the initial solutions produced by Synchro and SimTraffic under the default settings are continually not satisfactory. A great deal of solution tuning is needed to circumvent the deficiency of the software package or to reflect the special considerations for each traffic signal system. Furthermore, currently there is no formal guide to the tuning process. Each traffic engineer had an individual set of concerns and special methods for tuning. Thus, a questionnaire was devised based on the signal designing procedure learned in the first stage and was distributed to INDOT traffic systems engineers. The questionnaire included the following questions:

1. In Synchro/SimTraffic, some input data are often unavailable, such as conflicting pedestrian and bike volumes, number of parking maneuvers, total lost time, vehicle parameters, and driver parameters. How do you deal with these missing data? Do you use the default values?
2. SimTraffic's capabilities to simulate some special signal controller features (e.g., "volume density control") are limited. How do you address these problems?
3. Synchro calculates control indicators such as natural cycles and coordinatability factors. Do you use them? If yes, for what purposes do you use them?
4. Synchro reports various measures of effectiveness (MOE) of an arterial system such as total and average stops; LOS, volume/capacity ratio; and performance index. Which of them do you consider and for what purpose?
5. Synchro allows the user to customize the optimization settings, such as the range of the cycle length, the offset optimization intensity, and the step size of the iterations. Do you use these features and if yes, which ones?
6. Synchro outputs time-space diagrams for the arterial system. Do you check the bandwidth to improve the Synchro solution? Does this tool help reach a desirable coordination solution?
7. Synchro may generate a questionable solution if two intersections are closely spaced (for example, a diamond interchange). Did you experience such cases? Did you have to modify the solution?
8. An early return to green is a known issue of arterial signal control. Do you have a way of mitigating this problem? Please describe.
9. Sometimes, Synchro generates short background cycles. If the minimum greens are long, then there is no room left in a cycle for adjusting the splits to varying

volumes. Do you experience such cases? Do you have a way of avoiding these situations?

10. Please point out other situations where the Synchro solution seems to be deficient. Have you worked out any ways to avoid or fix these problems? Please describe.
11. SimTraffic allows the user to designate the durations of the seeding period and recording period. How do you determine these parameters?
12. SimTraffic is capable of recording multiple runs of simulation. Do you use this feature? If so, how many runs do you perform?
13. SimTraffic can create reports of a number of MOEs. Which of them do you examine?
14. SimTraffic has special features thought to be useful in analyzing the solution. They include flagging individual vehicles, watching the signal controller status, and color-coded static graphics of the results. Which of these features do you use?
15. Do you watch SimTraffic animation to inspect the quality of the coordination? To which operational aspects do you pay attention? What operational deficiency do you typically find when viewing SimTraffic animations? How do you address them?
16. Once the coordination plan is implemented, additional operational problems may occur. To what do you pay attention when watching traffic on the arterial? What signal adjustments do you consider?
17. Please describe a typical procedure of monitoring traffic and adjusting signals after coordination design implementation.
18. How often and what type of operational problems do you find after implementation.
19. Multiple signal plans are used for different periods of the day. What traffic volumes are used in Synchro to design coordination for rush hour periods and for non-rush hour periods? How is the time of transition between coordination plans determined?

4.2 Roundtable

The feedback collected through the questionnaire was assembled and presented in the *Systems Engineer Peer Group Meeting* at the INDOT Seymour District on June 23, 2005. Some issues (e.g. the unrealistically slow speed of vehicles in SimTraffic) generated heated discussions. Moreover, some additional issues and practices were brought up and discussed.

4.3 Summary of results

4.3.1 INDOT criteria of good coordination plan

Typically, signal optimization software minimizes the total delays and the number of stops at intersections along the subject arterial street (Husch and Albeck 1993-2004) (McTransCenter 1990-2006). INDOT engineers follow a more perception-based approach to coordination design that exceeds the routine optimization objective of Synchro and other commercial optimization packages. An arterial signal plan is acceptable when the following conditions of good coordination are satisfied:

1. Traffic progression along the arterial is reasonably smooth. Most of the drivers moving along the arterial street do not stop at two consecutive intersections. Drivers do not stop more than once at the same intersection (no congestion).
2. No perceptible queuing interaction persists during the design period. There is no queue spillback affecting queue discharge at the upstream intersection. Through lanes on the arterial street are not blocked by adjacent queues of turning vehicles.
3. If the studied street is modernized, the average arterial travel time is shorter or at least comparable to that before modernization.

A current signal timing plan is warranted for updating if a new solution can be found that eliminates some of the problems of the existing solution and it meets the three conditions of good coordination. For new installations, only the first two conditions apply.

The design process usually progresses through a sequence of iterations. The above conditions are used repeatedly during the design process to evaluate the intermediate signal solutions and to decide if the design is complete or should be continued.

Under adverse traffic and/or geometry conditions, good coordination as determined by the three criteria may be difficult or impossible to reach. In such a case, modernization is

warranted and is considered successful if it alleviates the current operational deficiencies.

4.3.2 Design Issues and Practices Reported

In general, the development of a signal coordination plan follows the following procedures. First, the volume and geometry data are collected or obtained from past records and are then used as the inputs of the optimizer of the Synchro software. Second, the traffic systems engineer tunes up the solution given by Synchro based on engineering experience and judgments. Third, SimTraffic is used to simulate the running of the systems. By watching the animation and examining the MOE outputs of SimTraffic, the traffic engineers identify the problems and make additional modifications to the signal plan. Finally, the plan is implemented and tuned in the field. The issues and practices reported by INDOT traffic systems engineers are summarized below.

1. Slow speed issue

The vehicles are running at slower speed in SimTraffic than in reality. This may cause serious inconsistencies between the real traffic situation and the “design situation” and thus may induce much field tuning after the signal plans are implemented. Legally speaking, however, traffic systems engineers must design under the assumption that drivers conform to the posted speed limits. One method to approximate the real speeds on the road without violating the legal assumption is to adjust the driver parameters, such as the percentage of aggressive drivers, in SimTraffic. Research may be conducted in the future to identify other approximation methods; for example, the sensitivities of the default parameters on the vehicle speeds will be investigated.

(1) Inconsistency of Synchro/SimTraffic MOEs and reality

Although Synchro output the “optimal” possible solution, systems engineers can often improve the MOEs by tuning the solution. This may be caused by the discrepancy between Synchro’s criteria of “optimality” and that of the engineer. One method to reduce such problems is to use the “manual” option instead of the “automatic” option when requesting Synchro to search for the solution, and then select the optimal solution manually based on the MOEs outputs of the alternatives. This lack of control of the objective function is a major deficiency of Synchro.

In addition, sometimes an adjustment improves the MOEs in SimTraffic but worsens the traffic situation in reality. For example, sometimes the solution given by Synchro produces much worse performance than the original settings. Sometimes gridlock emerges, which can be a serious problem when emergency vehicles are blocked. This may require a great deal of field-tuning efforts.

3. Short cycle length

Synchro often gives a cycle length that is too short and Transyt often gives a

cycle length that is too long. A common practice is to set the range of allowed cycle length to exclude unrealistic results. A typical minimum cycle is 70 seconds.

More specifically, to determine the minimum cycle length of an arterial system, an engineer will manually calculate the minimum cycle length of the most congested intersection and then use this cycle length as the lower bound when searching for the optimal cycle length in Synchro. It is found, however, that Synchro often output this manually-determined cycle length as the “optimal” cycle length. In such situations, the systems engineers are actually designing the cycle length for the arterial system themselves.

4. Special optimization settings

The typical parameters of the Synchro optimizer are set as: five seconds of cycle increment, extensive optimization intensity, and 70 seconds of minimum cycle. Other special settings like “half cycle length” and “uncoordinated” signal are not usually allowed in optimization. In practice, it is extremely rare to allow an uncoordinated signal in a coordinated arterial system. “Half cycle length” is seldom used except for a two-phase signal among several eight-phase signals or for an exit ramp. When it becomes necessary, engineers usually run several scenarios to check the impacts of these two options. For example, the engineer will run the following scenarios: (1) never allow uncoordinated and never allow half cycle length; (2) allow sometimes uncoordinated and never allow half cycle length; and (3) never allow uncoordinated and allow half cycle length. By doing so, the engineer knows whether these two special settings will help to improve the solution.

5. Missing Input Parameters

Some inputs of Synchro/SimTraffic are often unavailable, such as conflicting pedestrian and bike volumes, number of parking maneuvers, total lost time, vehicle parameters, and driver parameters. It is common to use the default values of the software package. Occasionally, when traffic systems engineers find that the default values deviate significantly from reality and cause severe modeling errors, they may use specific estimated values based on the available information and personal engineering judgments.

6. Pedestrian issues

Generally there are no pedestrians in the arterial systems. It would not be cost-effective to incorporate the considerations of pedestrians for all intersections all the time since this will elongate some phases significantly and deteriorate the performance of the arterial systems. (Wide intersection? Median?)

6 Use of Special Synchro Control Indicators

Synchro provides some special control indicators like “natural cycle length” and “coordinatability factors.” These indicators are not often utilized by the INDOT traffic systems engineers. They have found in practice that the natural cycle lengths are often unrealistically low so they can only use the indicator to get a feel of the

minimum cycle length. Similarly, there are some inconsistencies in using the coordinatability factor and thus this control indicator is not examined usually. This factor may be used to decide: (1) not to coordinate some intersections at certain times of the day or (2) to use half and double cycles.

8. Multiplicity of Synchro MOEs

Synchro reports various measures of effectiveness (MOE) of an arterial system, such as the total and average stops, the LOS, the volume/capacity ratio, and the performance index (PI). The current practices of traffic systems engineers focus on the number of stops and the PI to guarantee the smoothness of traffic progression. Since truck traffic is a major component of the total traffic in Indiana, a guiding philosophy to reduce their trouble is that “once the traffic is moving, let’s keep it moving smoothly even if it slightly increases total delay.” The V/C ratio is often used as an indicator of capacity sufficiency. Generally speaking, SimTraffic’s MOEs are considered more realistic and are favored over Synchro’s MOEs in operational analysis. However, the practice is not standardized. Some systems engineers focus on delay, stops, V/C, and queues in decreasing order. The PI is considered vague by them and is not commonly used.

9. Use of SimTraffic MOEs

Various practices are reported in using the MOEs of SimTraffic. Some engineers prefer viewing a MOEs summary by arterial and will include all MOEs. Others focus on measures that are comparable to the observable field data, such as stops, delays, and speeds. Still others mainly use stops and delays (total and per vehicle). Sometimes the engineers use network performance averages for total stops and delays for quick comparisons and use the queue/blocking report when queuing is a concern.

10. Time Space Diagram

The Time Space Diagram (TSD) is considered one of the most used tools of Synchro by the traffic systems engineers. Flow Diagrams are considered more useful than Bandwidth Diagrams. It is a major tool to identify poor offsets and early-return-to-green problems under different levels of volume (90th, 70th, or 50th percentiles) in the design phases. In the implementation phase, TSD is sometimes used in test runs along the arterial to assure the offsets are properly implemented.

11. Diamond Interchange

Synchro sometimes provides poor solutions for diamond interchanges or closely spaced intersections. The reason may be that it emphasizes major movements and thus produces poor solutions for side traffic or off-the-ramp traffic. The dominant practice is to adjust the solutions manually or to use Passer III in such locations.

12. Early Return to Green Methods

Early-return-to-green is a known issue of arterial signal control. If an upstream intersection returns to green early due to weak side traffic, then arterial traffic that passes at the beginning of the green will likely stop at the downstream intersection. Traffic systems engineers believe there is no perfect solution to this issue under the current framework of arterial signal control. If this problem is severe, however, the engineers will try to hold the traffic longer at the upstream intersections, probably by recalling the side street since they will be stopped at the downstream intersections anyway. Another proposed solution is tightening up the uncoordinated phases, which might help since this adjustment reduces the fluctuations of the starts of green. As a last resort, the engineers will try to adjust the offsets in the Time Space Diagrams to avoid this problem as much as possible.

13. Preference to arterial traffic is not considered by Synchro In designing signal plans for arterials, the preference of smooth progression of arterial traffic over that of the side street is not well reflected in Synchro, which is probably because Synchro is designed for traffic networks where all movements are equally important.

14. Synchro Split Optimization

Traffic systems engineers report problems with the split optimization of Synchro. Sometimes certain phases (e.g. the left turn phase or the side street through phase) of the Synchro solution are too short. The current practice is to set a larger cycle length when this problem is caused by capacity shortage. It is also observed that when the intersection is well below capacity, the non-coordinated phases often get a too long green. Therefore, tightening up the non-coordinated phases with the excessive capacity may help bring more flexibility to the coordinated phases.

15. Simulation Settings in SimTraffic

Several scenarios of simulation settings are reported. Some engineers use 15 minutes of seeding Time to guarantee the time is sufficient to reach equilibrium and use 15 or more minutes of running time and rerun SimTraffic five times to reach a stable estimation. Other engineers use 10 minutes of seeding Time but 60 minutes of running time in one run to catch more variation of traffic. Still others use a seeding time of 10 minutes and a recording time of 60 minutes for the final evaluation while in the first steps, three minutes and 10 minutes are used to get a quick assessment of the system and the impacts of the adjustments. Sometimes the PHF factor and the Anti PHF factor are used for different simulation intervals to approximate the variable traffic.

16. Special Features of SimTraffic

SimTraffic provides some special features thought to be useful in analyzing the solution. They include flagging individual vehicles, watching the signal controller status, and color-coded static graphics of the results. Among them, flagging individual vehicles is often used to inspect the progression. The other features,

including a signal controller status window and color-coded static graphics are less frequently used. Another feature, Volume-Density Control, is seldom used since it is considered to have little effect on the coordinated phases.

17. Use of SimTraffic Animation

INDOT engineers use animation to inspect various issues of coordination, including platoon progression, capacity of the intersections, offset adjustments, insufficient left turn problems, side-street queuing problems, consecutive stops, and unstable speeds.

18. Time-of-Day Plans

At different time-of-day (TOD) settings, the signal control plans should be different. TOD plans are usually determined by the engineer's judgment of the particular traffic pattern for the location. It is desirable to perform Synchro analysis for each TOD. A possible method to determine the transition time is to plot 12 hours of hourly volume, or 15-minute volumes, versus the time of day. The peak hourly volumes for each TOD are chosen as the design volumes.

19. Field Tuning

After the coordination plan is implemented, additional operational problems may still occur. Extra efforts of field-tuning are crucial. The problems to observe in the field include: (1) poor platoon progression, (2) left turn and side street blockage, (3) possible early return to green, (4) excessively low travel speed, (5) long queues, and (6) back to back stops.

Typically, adjustments are made to the original plan. Offsets are most often adjusted, and splits are changed sometimes. The typical field-tuning includes the following steps: (1) implement the plans, (2) identify problematic areas by a test run, (3) watch and revise settings, (4) make sure adjacent intersections are not negatively affected, and (5) test drive again after a week or two. PCTravel from JAMAR is used to record travel times. Some engineers claim that most of the adjustments can be done in Synchro and SimTraffic.

20. Sensitivity Analysis of Default Parameters

Synchro and SimTraffic use many default values. Traffic systems engineers would like to have a better understanding of the effects of these parameters on the simulation. On the basis of this knowledge and the field observations, the engineers will be able to somewhat bridge the gap of micro-simulation and reality by making local adjustments to the default parameters.

21. Lack of Signal Controller Models

Synchro/SimTraffic lacks the capacity to model real signal controllers. It would be helpful to be able to model specific behaviors of signal controllers like Econolite, Peek, or Eagle. Furthermore, the knowledge of the traffic signal controller is crucial

for a traffic systems engineer. Knowing the features of controllers and how are they implemented and representation of them in Synchro/SimTraffic will be very helpful.

22. Model of Heavy Vehicles

Heavy vehicle movements are not correctly modeled. For example, trucks usually turn at slower speeds and may take up multiple lanes to complete the turn.

Some capacity factors not included. Long queues can develop after implementation due to slow startup times, mid-block traffic, or some controller features that are not considered by SYNCHRO/SimTraffic

Synchro/SimTraffic models traffic under assumed conditions that include reliable and accurate detection. Total detector latency of 500 ms can cause a controller over-perception of traffic volume by 360 vphpl.

23. Desired improvements of Synchro and SimTraffic

Traffic engineers report the need for improvements of the software. Some features that will be very useful: (1) a time space diagram output of SimTraffic, (2) a frequency of side-street max-out, (3) better representation of the controllers' behaviors, and (4) customization of the objective function.

24. In SimTraffic, no vehicle will stay in the intersection and block it, which is not true in reality.

5 ANALYSIS OF SELECTED COMPONENTS OF Synchro/SimTraffic MODEL

In optimizing traffic signal systems, a great amount of time is consumed in field-tuning after implementation, which not only considerably damages the efficiency of signal modernization, but also potentially deteriorates the level of service of the system since the field-tuning is inherently local. The adjustments made according to the limited observation of the traffic conditions at a particular spot may in the long run result in worse overall system performance.

A major cause of the field-tuning is the discrepancy between SimTraffic operations and actual system operations observed by the engineers. One of the important sources of this difference is the inaccuracy of important traffic parameters, such as speeds and saturation flow rate.

Therefore, the model of Synchro/SimTraffic was carefully investigated in this research. In general, the help manual (Husch and Albeck 1993-2004) of the software provides clear exposition. The manual, however, is not exhaustive. Some issues encountered in the research require knowledge beyond the level of detail provided by the manual. In such situation, several methods are used to obtain as much information as possible, particularly experimenting in the micro-simulation environment.

In the following sections in this chapter, the investigation of the speed model and the saturation flow rate model is presented.

5.1 Speed Model of SimTraffic

Correctly simulated speed is vital for accurate modeling of the cruise times and arrival times of vehicles at intersections along an arterial street. It has been found that under the default SimTraffic settings, the simulated cruise speeds are often lower than entered in Synchro. This discrepancy may cause inadequate signal timings that have to be adjusted later during the field testing. In the manual, the speed model is explained under the topic “The SimTraffic Model/Car Following and Speed Selection.” Micro-simulation experiments were performed to isolate the relevant settings which affect the speed

selection and accelerating behavior of the vehicles.

5.1.1 Cruise Speed

The free flow speed is defined as the desired speed of the driver when no impediment exists. In the manual, it is referred to as “cruise speed.” We will follow the terminology of Synchro to avoid confusion among users already familiar with the Synchro terminology.

In SimTraffic, each simulated vehicle has been assigned a driver type and a vehicle type. The driver type and the vehicle type are assigned randomly at the moment of entering a vehicle to the system. The share of each type of vehicle and driver in simulated traffic reflects the percentage breakdown by category.

There are 10 types of drivers in SimTraffic. Each type represents 10% of the drivers. The driver type determines the characteristic associated with the driver preference. Generally speaking, type 1 represents the most conservative drivers and type 10 represents the most aggressive drivers. Although the percentage breakdown is fixed for driver types, the parameters of each type are configurable. For example, if the user believes that 10% of drivers are conservative, 70% are average, and the remaining 20% are aggressive, then the user should enter the parameters characterizing a conservative driver for type 1 driver, the parameters characterizing an average driver for types 2 through 8, and the parameters characterizing an aggressive driver for types 9 and 10.

A **speed factor** is one of the parameters associated with the driver types. It is particularly important for calibrating the **cruise speed**. The parameter can be changed through the menu command “**Options/Driver Parameters**”. Figure 5-2 shows how to set the speed factors in SimTraffic.

The cruise speed is calculated as:

$$\text{Cruise Speed} = \text{Speed Factor} \times \text{Link Speed}$$

If the traffic systems engineer believes that there are discrepancies between the actual and simulated average speeds between intersections, then the speed factors should be adjusted to better approximate the reality.

To verify this model, a sample system is configured. The layout of this system is illustrated in Figure 5-1. Both intersections are unsignalized to exclude the effects of signal-related deceleration. The WB and EB movements are controlled by stop signs at

the left intersection and the NB and SB are controlled by stop signs. Therefore, the EB vehicles will first stop at the left intersection. The distance between the two intersections are sufficiently long so that the vehicles can accelerate to its desired speed between the two intersections. The EB approach speed of the right intersection is thus approximated as the actual cruise speed simulated in SimTraffic. The volumes are set as 60 vph so that the car interaction effects are minimized.

Figure 5-1 Sample system to verify cruise speed model

As introduced above, the speed factors will affect the cruise speed of the vehicles. For convenience, the speed factors of all 10 types of drivers are set the same as in Figure 5-2. Another relevant setting is the link speed. This can be configured in Synchro by double clicking the link. The link speed setting window is illustrated in Figure 5-3. Finally, the example used only one type of vehicle as in Figure 5-4. All types of heavy vehicles are excluded. The maximum acceleration was mostly set at its maximum of 15 ft/sec² and modified once or twice to see if this parameter affected the cruise speed of the vehicles. To obtain the report of the speed at the right intersection, the report options are set as in Figure 5-5.

SimTraffic Parameters

Fuel-Buses	HC Emissions	CO Emissions	NOx Emissions		
Vehicles	Drivers	Intervals	Data Options		
Fuel-Cars	Fuel-Trucks				
Driver Types	1	2	3	4	5
Yellow Decel (ft/s ²)	12	12	12	12	1
Speed Factor (%)	1.40	1.40	1.40	1.40	1.4
Courtesy Decel (ft/s ²)	2	9	8	7	
Yellow React (s)	0.7	0.9	1.0	1.0	1.
Green React (s)	0.8	0.7	0.6	0.6	0.
Headway @ 0 mph (s)	0.65	0.63	0.60	0.58	0.5
Headway @ 20 mph (s)	1.80	1.70	1.60	1.50	1.4
Headway @ 50 mph (s)	2.20	2.00	1.90	1.80	1.7
Gap Acceptance Factor	1.15	1.12	1.10	1.05	1.0
Positioning Distance (ft)	1500.0	1300.0	1100.0	900.0	700.
Positioning Advantage (veh)	15.0	15.0	15.0	15.0	15.
Optional Advantage (veh)	2.3	2.3	2.3	1.0	1.
Mand. Factor, Controlled (%)	10	10	10	20	2
Mand. Factor, Free (%)	30	30	30	50	5

OK Cancel Default Driver Parameters

Maximum deceleration rate for yellow light (ft/s²).

Figure 5-2 Setting speed factors

Link Settings

Street Name:

	WB	EB
Link Speed (mph):	<input type="text" value="45"/>	<input type="text" value="45"/>
Link Distance (ft)	<input type="text" value="7000"/>	<input type="text" value="7000"/>
Travel Time (s)	<input type="text" value="106.1"/>	<input type="text" value="106.1"/>

Link OD Volumes

Lane Change Dists:

Scope

☒ Links named "Main"

☐ Entire Length of Street

☐ This Link Only

☒ Directional Speed and Distances Equal

Red values are overrides. Press F12 to revert.

Figure 5-3 Setting link speed

SimTraffic Parameters

Fuel-Buses

HC Emissions

CO Emissions

NOx Emissions

Vehicles

Drivers

Intervals

Data Options

Fuel-Cars

Fuel-Trucks

Vehicles Types	1	2	3	4	5
Vehicle Name	Car1	Car2	Truck SU	SemiTrk1	SemiTrk
Vehicle Occurence (%)	1.00	0.00	0.00	0.00	0.0
Maximum Speed (mph)	80	75	75	68	6
Maximum Acceleration (ft/s^2)	10	9	5	3	
Vehicle Length (ft)	16.0	14.0	35.0	53.0	53.
Vehicle Width (ft)	6.0	6.0	8.0	8.0	8.
Vehicle Fleet	Car	Car	Trk	Trk	Tr
Average Number of people per	1.3	1.3	1.2	1.2	1.

OK

Cancel

Default

Vehicle Parameters

Maximum deceleration rate for yellow light (ft/s^2).

Figure 5-4 Vehicle parameters setting

Figure 5-5 Report option settings

The results of the experiments are listed in the tables below. The link speed for the westbound traffic is set as 50 mph, and 45 mph for the eastbound traffic. It is concluded that the cruise speed selection model is accurate. However, it is important to recognize that the impediment of traffic, the distance available for acceleration, and the driver type are all relevant factors.

	Speed Factor	Max. Acceleration	Max. Speed	Speed (mph)	
	%	(ft/s ²)	(mph)	West Bound (50mph)	East Bound (45 mph)
case 1	1.5	15	80	75	64
case 2	1.1	15	80	55	49
case 3	1.2	15	80	60	53
case 4	1.3	15	80	65	57
case 5	1	15	80	50	44
case 6	1.4	15	80	70	60
case 7	1.4	10	80	70	59

Table 5-1 Experiment results of speed model

5.1.2 Acceleration Model

According to the manual of SimTraffic, there are two important vehicle characteristics affecting the vehicle speed: the maximum speed and the acceleration model. The maximum speed is the speed reachable by the vehicle. It is associated with the vehicle type. Figure 5-4 shows how to set the vehicle parameters, including the maximum speed. On the other hand, a driver accelerates only to the cruise speed which is associated with the driver type. Figure 5-2 shows the settings of the driver type.

The acceleration model is described in the manual as follows: “Vehicles can accelerate at the maximum acceleration at speed 0, and have zero acceleration at the vehicle's maximum speed. The maximum acceleration rate declines **linearly** as speed increases.” Let the maximum acceleration be $a_{\max}$, and the maximum speed be $v_{\max}$, then the above description can be translated into the differential equation:

$$\frac{dV}{dt} = a_{\max} \left(1 - \frac{V}{V_{\max}}\right)$$

where V is the speed of the vehicle and t is the time. Suppose that the vehicle accelerate from 0 speed. Then, $V(0) = 0$. The solution of the equation is

$$V(t) = v_{\max} [1 - \exp(-\frac{a_{\max}}{v_{\max}} t)]$$

The speed-time profile generated by this equation is illustrated in Figure 5-6. For example, when $a_{\max} = 7 \text{ ft/s}^2$ and $v_{\max} = 30 \text{ mph}$, then if a vehicle accelerates from 0 speed and no impediment exists, it will use around 8 seconds to accelerates to 30 mph and continue to run at this speed.

Figure 5-6 Speed-time profiles

To verify this solution, a simple pre-time system is set up as in Figure 5-7. Flag a vehicle stopped by red signal by clicking on it. The vehicle status window will show in the upper right corner. When the signal turns green, play the animation frame-by-frame with the playback control toolbar to record the speed-time profile. Notice that one frame represents 0.5 seconds and the speed and acceleration are not updated precisely in the status window. Therefore, it is recommended to forward the animation for 14 to 18 frames and record the speed. Another caveat is that $a_{\max}$ is related to vehicle type as

indicated in Figure 5-4.

Numerous experiments are performed and it is verified that the SimTraffic animation closely follows $V(t) = v_{\max} [1 - \exp(-\frac{a_{\max}}{v_{\max}} t)]$. The following variants of this formula are also useful.

$$t = -\frac{v_{\max}}{a_{\max}} \ln\left(1 - \frac{v}{v_{\max}}\right)$$

$$a_{\max} = -\frac{v_{\max}}{t} \ln\left(1 - \frac{v}{v_{\max}}\right)$$

Attention should be paid to the units when using these formulas.

Figure 5-7 Verify Acceleration Model of SimTraffic

5.2 Saturation Flow Rate

Saturation flow rate is an important factor in replicating the traffic conditions. Due to the difficulties involved with precisely measuring the saturation flow rate in a roadway system, the default values are often used. According to a recent study of the saturation flow rate of Indiana (Perez-Cartagena and Tarko 2004), however, indicates that the range of saturation flow rate in different areas can be considerably wide. Table 5-2, Table 5-3, and Table 5-4 list the recommended traffic parameters for Indiana. The area is categorized according to the population size of the jurisdiction.

Number of Lanes in lane group	Population Size Near Intersection		
	< 20,000	20,000 – 100,000	Indianapolis
1	1540	1800	1960
2	1580	1840	2010
3	1600	1860	2020

Table 5-2 Recommended Base Saturation Flow Rates for Indiana (Perez-Cartagena and Tarko 2004)

Parameter	Value
Green Time Extension	2.8 s
Heavy Vehicle Equivalency Factor	2.1
Start-up Lost Time	1.9 s
Protected Left-turn Adjustment Factor	0.96

Table 5-3 Recommended Capacity Parameters for Indiana (Perez-Cartagena and Tarko 2004)

Number of Lanes in lane group	Population Size Near Intersection		
	< 20,000	20,000 – 100,000	Indianapolis
1	2.34	2.00	1.83
2	2.27	1.95	1.79
3	2.25	1.94	1.78

Table 5-4 Recommended Saturated Time Headways for Indiana (Perez-Cartagena and Tarko 2004)

Some of the above parameters can be easily adjusted in Synchro or SimTraffic to reflect the local condition. Nevertheless, according to the help manual of SimTraffic, there is no method of controlling the saturation flow rate precisely. The only recourse is to change the, so-called “headway factors” for all links in Synchro. For example, in the system of Figure 5-8, to increase the saturation flow rate simulated in SimTraffic, first, the headway factors of the external link “1-3”, “3-4”, “3-5”, “6-7”, “6-8”, and “6-2” must be replaced by a value smaller than 1.

Figure 5-8 Adjust headway factors of external links

Second, the headway factors of the internal movements must be overridden accordingly in the lane window of the intersections, as is illustrated below.

LANE WINDOW	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Headway Factor	0.80	0.80	0.80	0.80	0.80	0.80	0.80	0.80	0.80	0.80	0.80	0.80

Figure 5-9 Adjust headway factors for internal movements

The simulated saturation rate also depends on the link speed. A simple system is set up in order to obtain the actual saturation flow rate profile in SimTraffic. The EB traffic is controlled by a stop sign. The traffic demand is set far above the capacity (e.g., 10,000 vphpl). Then, the vehicle exiting rate is approximately the saturation flow rate. For example, in an experiment, in 10 minutes of simulation, 342 vehicles exit the system. The corresponding saturation flow rate is thus approximately $342 \div 6 = 2052 \text{ vphpl}$.

Figure 5-10 System for experiment of Saturation Flow Rate

The following Figure 5-11 summarizes the experimental results of the effects of headway factor adjustment on the simulated saturated flow rate with different link speeds. This figure can be used as a guide to help approximate the desired saturation flow rates in SimTraffic. For example, the headway factor of 0.77 seems to be suitable for obtaining the saturation flow rate of 2,000 vphpl along an arterial street with a 45 mph speed. These values are marked on the graph in Figure 5-11.

Figure 5-11 Simulated saturation flow rate against Headway Factor

6 MODELING TRAFFIC VARIATIONS

6.1 Motivation

To avoid frequent re-timing and unexpected poor performance of the traffic signal system, the performance of the system under fluctuating traffic conditions is investigated. Vehicular traffic is inherently stochastic and cannot be fully characterized by a deterministic model. The current approach to signal coordination partly addresses the issues of within-day and day-to-day random variations by applying different pre-designed signal settings during different periods of a day and for different days of a week. A typical arterial system will have signal timing plans for AM peak, midday, and PM peak, which will probably differ as well for weekdays and weekends. In optimizing the signal timings, the current practice is to select a set of volumes observed in a field study as the presumed baseline peak volumes. The signal coordination settings (background cycle, force-off points, and offsets) are optimized with the software package and kept fixed within each period. Local real-time actuation control is deployed to accommodate minor variations of traffic.

The robustness of this approach, however, has not been adequately investigated or validated. In other words, the “optimized” plans’ performances are unknown under randomly fluctuating traffic volume conditions. A robust coordination plan should be suitable for various traffic volumes expected during the entire coordination period and preferably on different days. Several fundamental concerns are yet to be answered:

1. Is it appropriate to optimize the signal timings only for a set of selected 15-minute volumes and apply them to the multi-hour periods? For example, according to the traffic counts of 12 continuous hours, the signal timings were optimized to the volumes of 7:15 a.m. since it was the peak volume observed in the morning. Disregarding other volumes of the morning, the plan so optimized was applied to the entire AM peak period from 6:00 a.m. to 9:00 a.m. Obviously, such a timing plan may or may not be optimal when the mobility of the entire three hours instead of the peak 15 minute is concerned. How much mobility is compromised for the “disregarded” time intervals?
2. Is it appropriate to optimize signal timings based on one day’s counts and use this plan for several years? For example, the signal plan was optimized to the traffic observed in April 2000 and it was then applied in the period 2001 to 2005. Does

day-to-day traffic variation affect the operational performance significantly?

3. Is it appropriate to use traffic data collected on different days? Due to the constraint of manpower, the traffic data of the intersections along an arterial system are typically collected on different days, sometimes even different years. What are the possible effects of optimizing signal settings based on such data?
4. Is there a way to select, or compute, a more representative set of volumes which are the full available information? The current approach only utilizes limited information, namely. The peak volumes. Nevertheless, typically 12 continuous hours of traffic are recorded by the traffic systems engineers in the field study stage. All off-peak volumes are ignored as soon as the peak intervals are determined. How can an engineer incorporate the seemingly “redundant” but obviously valuable data?

In summary, there is a need for a practical procedure to evaluate the robustness of signal timing plans that takes into account traffic variability within coordination periods and from day-to-day for the same time of day. Ideally, an applicable procedure of improving signal timing robustness is desired. This procedures should not require multiple day measurements; instead, the information of off-peak intervals should be utilized and the inconsistency in data collection should be addressed.

Remarkably, the postulate of robust coordination is valid even for the future coordination settings adjustable in real time. Since these adjustments are made around base settings in a predetermined range, robust base settings reduce the probability of drastic adjustment and thus increase the efficacy of signal coordination and potentially traffic safety.

To accomplish this goal, these components are indispensable: (1) a stochastic model of traffic variation patterns for both within-day and day-to-day fluctuations, and (2) and a robustness evaluation procedure.

The following sections demonstrate the investigations of these topics. Based on the models, a procedure to simulate the traffic of multiple days is devised and the robustness of different methods is evaluated in the micro-simulation software under the generated traffic volumes.

6.2 Data description

Before exploring the procedures of robust signal design, the traffic variation patterns must be modeled first. In this research, the traffic variation pattern was investigated on the basis of true data collected in 2000. The traffic data were collected with pneumatic detectors on Northwestern Ave., Purdue University at West Lafayette, Indiana. (Figure 6-1) Continuous traffic data were available from April 15 to May 3, 2000. Since the two stations are closely spaced, most of the counts were identical. Therefore, only the data collected from station 2001 were analyzed.

Figure 6-1 Location of data collection

To emulate the data available in coordination signal design, the traffic counts are aggregated for each 15-minute interval. Figure 6-2 illustrates both the southbound and northbound traffic counts from April 15 to May 4, 2000. Since May 2 to May 5 were the weekdays of the final exam week of Purdue University's academic calendar, the traffic patterns were different from other regular weekdays and thus were excluded from further analysis.

Several further observations were obvious from Figure 6-2.

- Southbound and northbound traffic followed different patterns and thus could be considered separately;
- Weekday traffic and weekend traffic were considerably different.
- Friday's traffic patterns were different from other weekdays. This is obvious from Figure 6-3 and Figure 6-4 where the traffic counts of weeks 1 and 2 are illustrated.

Therefore, to investigate the traffic variation pattern of regular weekdays, only the data from Monday to Thursday from both weeks were further analyzed.

Figure 6-215-min traffic counts (Apr 14—May 4, 2000)

Figure 6-315-minute traffic counts from Apr 17—Apr 23, 2000

Figure 6-415-minute traffic counts from Apr 24—May 1, 2000

6.3 Day-to-Day Variation Model

From day to day, the traffic volume of the same period fluctuates randomly. Modeling day-to-day variation is indispensable for mimicking real traffic. Several alternatives were investigated in this research.

6.3.1 Gaussian Model

The starting model was the normal model. Due to the Central Limit Theorem, many distributions can be approximated by normal model to certain degrees. Previous research (Fox and Clark 1998) has found that the normality assumption is acceptable for modeling traffic flows. However, several inadequacies of the Gaussian Model preclude it from being applied in this study.

- It is possible to obtain negative traffic with the Gaussian model, which is totally unreasonable. Usually, when the mean of the Gaussian distribution is well above 0 and the standard deviation is not exceptionally large, the probability of negative values is negligible. However, in this study, low volume cases (e.g., for minor movements or for morning periods) are not scarce.
- For the Gaussian model, the variance and the mean traffic are independent, which is not in accordance with the available data. As is shown in Figure 6-6, the variance and the mean of the traffic were strongly positive-correlated. Furthermore, in practice, usually only one day of traffic counts are available for each period. There is no way to estimate two independent distribution parameters (i.e., the mean and the variance) when only one observation is available.
- Non-integer values will be generated with the Gaussian model since it is a continuous distribution model. Although this can be easily addressed by rounding off, it may introduce unnecessary inaccuracy.

Figure 6-5 Sample variance vs. sample mean

6.3.2 Poisson Model

A common model for count data is the Poisson model, in which the traffic counts of the same time interval follows the Poisson distribution. SimTraffic, for example, uses the binomial distribution vehicle generation mechanism to approximate the Poisson distribution (Husch and Albeck 1993-2004). The probability mass function of the distribution is:

$$\Pr(x = n) = e^{-\lambda} \frac{\lambda^n}{n!} \quad n = 0, 1, 2, \dots$$

An important property of this distribution is that its expectation and variance are equal, i.e.,

$$E(X) = Var(X) = \lambda$$

This is a convenience for the estimation process. Once the expected count is estimated, the associated PMF is determined and it follows that the variance is determined. Numerous empirical researches, however, suggests that the Poisson model is too restrictive. It is often found that the variance is significantly larger than the expectation. This phenomenon is called over-dispersion in count data modeling (Washington, Karlaftis et al. 2003).

Figure 6-6 shows the (sample variance /sample mean) values of different time intervals of the day for both northbound and southbound traffic. If the Poisson model is valid, then the points should scatter randomly around the $var/mean = 1$ line. However, it is clear the figure suggests that this claim is hardly acceptable. The majority of the time intervals have much larger variances than the mean values.

Figure 6-6 Variance-Mean observations of southbound and northbound traffic

6.3.3 Negative Binomial Model

A common alternative count data model is the Negative Binomial Model, whose PMF function is:

$$\Pr(x = n) = \frac{\Gamma(r + n)}{n! \Gamma(r)} p^r (1 - p)^n$$

where $\Gamma(\cdot)$ is a Gamma function. The expectation and variance of the negative binomial random variable is:

$$E(X) = r \frac{1-p}{p}$$

$$Var(X) = r \frac{1-p}{p^2}$$

This model allows the variance to be larger than the mean. Their relationship is:

$$Var(X) = E(x)(1 + \alpha E(x))$$

where $\alpha = 1/r$ is the over-dispersion factor (Washington, Karlaftis et al. 2003).

The Poisson Model is a limiting case of the Negative Binomial Model with the over-dispersion factor of 0. To model the day-to-day traffic variation pattern, the over-dispersion should be checked first. Assuming that the eight traffic counts of the same interval observed in these days were the samples from the same distribution, the population mean can be estimated with the sample mean and the population variance can be estimated with the sample variance.

$$E(X) = \bar{x} = \frac{1}{8} \sum_{i=1}^8 x_i$$

$$Var(X) = s^2 = \frac{1}{7} \sum_{i=1}^8 (x_i - \bar{x})^2$$

where x_i are the traffic counts.

More rigorous statistical tests were also performed with the aid of the software LIMDEP. Fitting the data with the negative binomial model, it was found that the over-dispersion factors were significant for both the SB and NB movements. As is shown in Table 6-1, the associative p-values of the hypothesis $\alpha = 0$ are only 0 and 0.05, respectively. Thus, enough evidence was found to reject the hypothesis that no over-dispersion exists.

Movement	Over-dispersion factor	t-statistic	P Z >t
Southbound	.417E-02	6.05	0.00
Northbound	.126E-02	1.948	0.05

Table 6-1 Over-dispersion factor estimation results

Therefore, it was determined that the variance of traffic is generally significantly larger than the expectation. The Poisson model, which is assumed by SimTraffic, tends to under-estimate the variability of traffic. The Negative Binomial Model should be employed to model the day-to-day variation.

A convenient feature of Negative Binomial Distribution is that it can be considered as a compound Poisson distribution with the mean value varying as a Gamma distribution. For example, if the over-dispersion factor is k and the expected level of traffic is u , then, Negative Binomial Random variable X can be generated in two steps.

- Generate l according to $Gamma(1/k, uk)$
- Generate X according to $Poisson(l)$

6.4 Within Day Traffic Profile Model

Several characteristics of the daily profile of the expected traffic are assumed for estimations.

- The profiles are smooth. The expected mean traffic volumes are assumed to be much more stable. The observed rough curves are considered as randomly disturbed realization of the smooth expected traffic profile.
- The expected traffic of any time should be non-negative.
- The variation models of all time intervals are related.

In addition, the peak traffic estimations are considered most important since the worst-case performance is of particular importance to the motorists' perceptions of the system's capability. The current method is not sufficient for the purpose of robustness investigation. First, it does not estimate the off-peak traffic volumes. Nonetheless, not only the expected peak traffic level, but also the expected off-peak traffic levels are required to examine the timing plan's performance for the entire day. Second, this method fails to utilize the information provided by the off-peak volumes. A within day traffic profile model is needed to address the problem and utilize this wasted information.

The following sections present the investigated alternatives of modeling the within day traffic profile.

6.4.1 Moving Average

Moving Average is one of the major methods used by INDOT engineers. The smoothed traffic, represented with $y(t)$, can be calculated with

$$y(t) = \frac{1}{4}[x(t) + x(t + 1) + x(t + 2) + x(t + 3)]$$

where $x(t)$ are the original observed traffic counts. The current approach of selecting the peak hour volume is:

1. for each intersection, sum up the traffic counts of all movements for each 15-minute interval;
2. calculate the four-point moving average traffic for each intersection;
3. identify the intervals where the moving average volume reaches the maximum in the morning, afternoon, and mid-day periods;
4. Use the peak 15-minute volumes at these peak intervals as the estimation of the

peak time traffic and calculate proper peak hour factors (PHF).

It is obvious that non-negativity is guaranteed by this method since all the observed traffic counts are non-negative. Figure 6-7 shows an example of this method. Several disadvantages of this method are readily illustrated by the figure.

- The moving average curve always underestimates the peak volumes and overestimates the valley volumes.
- In estimating an expected traffic level, only four observations are involved.
- Although the moving average curve is stabilized, the curve is not smooth enough.

Basically, this method uses the maximum four-point moving average traffic volume in each time window as the estimation of peak hour traffic. Only the volumes of the hour around the peak interval are utilized once the peak intervals are determined. The off-peak volumes observations are abandoned and no estimation of off-peak traffic is obtained. The method is conservative since it selects peak 15-minute volumes from the peak hour although the traffic of different movements may peak in different intervals.

Figure 6-7 Moving Average Smoothing Method

6.4.2 Frequency Method

This method is commonly used in digital signal processing research, especially for periodic data. The basic philosophy is that the high frequency elements are considered as the “noise” and the low frequency components are considered the “trend.” First, the time domain data is transformed into the frequency domain. Then, to extract the smoothed daily profile of the traffic, the high frequency noisy components are filtered out and then the filtered frequency signals are transformed back to the time domain signal (Oppenheim and Schaffer 1975). To illustrate this method, the northbound traffic of Apr 17, 2000 is studied as below.

As is shown in Figure 6-8, the observed traffic curve is not smooth. There is a peak in the morning around interval 32 (8:00 a.m.) and another peak in the afternoon around interval 70 (17:30 p.m.).

Figure 6-8 Observed traffic of Apr 17, 2000

Figure 6-9 shows the transformed curve in frequency domain. The majority of the signals are concentrated in the lowest and highest frequency band. Following the previous philosophy, the high frequency components are considered random disturbance and thus

are filtered out. The lowest 15 frequency elements are preserved and transformed back to the time domain. Figure 6-10 shows the original counts and the extracted trend with this method.

Although the trend satisfies the smoothness and the non-negativity requirements of the expected curve, the peak and valley traffic volumes are not followed closely by the trend line. Therefore, this method is considered insufficient for this research.

Figure 6-9 Frequency distribution of the counts of Apr 17, 2000

Figure 6-10 Extracted trend from the counts

6.4.3 Polynomial Smoothing

The polynomial profile function is a natural choice when knowledge about the underlying profile is limited due to its nice properties. In this model, the expected traffic profiles are assumed to be polynomial functions.

$$\hat{y} = a_0 + a_1t + a_2t^2 + \dots + a_nt^n$$

where $\hat{y}$ is the fitted value of traffic, a_i 's are the coefficient of the function, and t is the sequence of time intervals from 1 to 96. The order of the polynomial is n . Obviously, the larger n is, the more flexible the curve will be. However, more computation and precision will be required.

The key advantages of this method are:

1. The polynomial curve is smooth. It is well known that the polynomial curve is continuous and differentiable, therefore, smoothness is guaranteed.

2. The efficient computational algorithm (i.e., least-square estimator) can be used to fit the curve. The fitted profile line automatically minimizes the sum of the squared errors and maximizes the R-square.
3. Numerous special treatment methods can be used for certain modeling issues, such as fitting with constraints, non-negative transformation, and assigning weights to certain intervals.

To illustrate the method, again the counts of April 17, 2000 are fitted with a polynomial line of the 8th order. From Figure 6-11, the disadvantages of this method can be identified.

- Non-negativity is not guaranteed. For the intervals between 10 and 20, the fitted traffic counts are negative.
- The fitted curve is well below the actual observed counts at the peak intervals 34 and 72.

Figure 6-11 Polynomial smoothing of the counts of Apr 17, 2000

6.4.4 Exponential Polynomial Smoothing

A common technique to enforce the non-negativity requirement is to use the exponential polynomial method (Lewis and Shedler 1979) . In this model, the expected traffic profiles are assumed to be exponential polynomial functions.

$$\hat{y}_t = e^{(a_0 + a_1 t + a_2 t^2 + \dots + a_n t^n)}$$

where $\hat{y}_t$ is the fitted value of traffic, a_i 's are the coefficient of the function, and t is the sequence of time intervals from 1 to 96. The order of the polynomial is n . Due to the non-negativity of the exponential function, the non-negativity of the estimated traffic level is thus guaranteed.

To fit the curve, a common practice is to first do a logarithm transformation to the counts and then fit the transformed data using the usual polynomial fitting method. The fitted log-values are then transformed back with the exponential function. Once again, to illustrate the effects of the method, the northbound traffic counts of April 17, 2000 are fitted with an exponential polynomial line of the 8th order. Not surprisingly, all the fitted values are non-negative.

However, the peak time volumes are still not closely captured.

Figure 6-12 Exponential polynomial smoothing

6.4.5 Weighted Polynomial Smoothing

Since the volume levels of the peak intervals are of particular importance, an ad hoc method must be applied. A common method is weighted least square. In this method, the errors of some particular counts are weighted more than other counts. In the usual least square fitting, the objective function is the sum of square.

$$S = \sum_i (y_i - \hat{y}_i)^2$$

where y_i are the actual counts and $\hat{y}_i$ are the fitted counts. In the weighted least squared method, a weight is given to each fitting error. The objective function becomes

$$S = \sum_i w_i (y_i - \hat{y}_i)^2$$

where w_i are the weights (Washington, Karlaftis et al. 2003).

To illustrate the method, the peak intervals 34, 50, and 70 are given weights of 15 and all other intervals have a weight of 1 for the northbound traffic counts of April 17, 2000. Figure 6-13 shows the result of such a weighting procedure. Comparing with Figure 6-11, it is observed that the peak intervals are captured better while the non-peak intervals have slightly worse fitting results. To guarantee non-negativity, the exponential transformation introduced in Section 6.4.4 can be applied similarly.

Figure 6-13 Weighted Polynomial Smoothing

6.4.6 Constrained Polynomial Smoothing

One of the characteristics of the roadway traffic is the conservation of traffic. Since the traffic counts are aggregated into 15-minute counts and the travel time between two adjacent coordinated intersections is usually much less than 15 minutes, the sum of the upstream traffic should be approximately the same as the sum of the downstream traffic.

Figure 6-14 Layout of a simple arterial system

For example, Figure 6-14 is a simple two-intersection system. The following notations are used:

$I : \{1, 2\}$	<i>Intersection ID</i>
$J : \{1, 2, \dots, 12\}$	<i>Movement ID</i>
$t : \{1, 2, \dots, 48\}$	<i>Time interval</i>
$y_{i,j}(t) :$	<i>Traffic counts</i>

In this system,

$$E(\hat{\mathbf{a}}_{j=1,2,3} y_{1,j}(t)) = E(\hat{\mathbf{a}}_{j=2,9,10} y_{2,j}(t)) \quad t \in \{1, 2, \dots, 48\} \quad (\text{eastbound traffic})$$

$$E(\hat{\mathbf{a}}_{j=4,5,6} y_{2,j}(t)) = E(\hat{\mathbf{a}}_{j=5,7,12} y_{2,j}(t)) \quad t \in \{1, 2, \dots, 48\} \quad (\text{westbound traffic})$$

It should be noticed that this constraint is enforced on the expected values. Therefore, the Theil's method should be used (Theil and Goldberger 1961). This method is best explained in matrix form. The usual regression model is represented by:

$$y = Xb + e$$

where y is a $n \times 1$ matrix representing the dependent variables, b is a $k \times 1$ matrix representing the coefficient to be estimated, X is a $n \times k$ matrix of the observed values of the independent variables, and e is a $n \times 1$ matrix representing the error terms. For example, if 12 hours of vehicle counts are available and 4th order of polynomial function of the time intervals are used as the smoothing method, then the equations can be written as:

$$\begin{bmatrix} y_1 \\ y_2 \\ \vdots \\ y_M \\ y_{47} \\ y_{48} \end{bmatrix} = \begin{bmatrix} 1 & 1 & 1 & 1 & 1 \\ 2 & 2^2 & 2^3 & 2^4 & \\ 3 & 3^2 & 3^3 & 3^4 & \\ M & M & M & M & M \\ 48 & 48^2 & 48^3 & 48^4 & \end{bmatrix} \begin{bmatrix} b_0 \\ b_1 \\ b_2 \\ b_3 \\ b_4 \end{bmatrix} + \begin{bmatrix} e_1 \\ e_2 \\ e_3 \\ M \\ e_{48} \end{bmatrix}$$

The ordinary least square estimator of this system (Washington, Karlaftis et al. 2003) is:

$$b_{OLS} = (X'X)^{-1}X'y$$

In the simplest case, if the constraint is $E(y_u) = E(y_d)$, then the matrix form can be re-written as:

$$\begin{bmatrix} y_u \\ y_d \end{bmatrix} = \begin{bmatrix} T & 0 \\ 0 & T \end{bmatrix} \begin{bmatrix} b_u \\ b_d \end{bmatrix} + \begin{bmatrix} e_u \\ e_d \end{bmatrix}$$

where $T = \begin{bmatrix} 1 & 1 & 1 & 1 & 1 \\ 2 & 2^2 & 2^3 & 2^4 & \\ 3 & 3^2 & 3^3 & 3^4 & \\ M & M & M & M & M \\ 48 & 48^2 & 48^3 & 48^4 & \end{bmatrix}$. Applying the OLS estimator on this transformed

system of equations, the estimates will naturally minimize the difference between $\hat{y}_u$ and $\hat{y}_d$.

An extension of this method is to enforce the constraint $E(y_u) = k \times E(y_d)$ where k is a traffic balance factor. Ideally this factor should be 1, however, allowing k to be a value around 1 will possibly bring a better fitting result to the equations.

To examine the effects of this method, a real arterial system is tested. Figure 6-15 shows the layout of the system. The northbound traffic conservation is enforced using the above method and the optimal balance factor k is selected by iteration for the AM and PM time periods.

Figure 6-15 an arterial system on US-52

Figure 6-16 illustrates the fitted curve of both independent fitting and constrained fitting. It is found that for major movements, the fitted values of both methods are approximately the same. For minor movements, a significant difference exists.

A key disadvantage of this method is that the constrained fitted curve sometimes goes far below zero to keep the balance of the traffic. Although the exponential polynomial method can be used to guarantee non-negativity, the Theil's method of enforcing stochastic constraint becomes infeasible. Therefore, non-negativity and traffic conservation cannot be enforced at the same time. Since non-negativity is fundamental and the traffic conservation can be handled automatically by Synchro using mid-block flows, it is decided that the traffic conservation will not be enforced in further modeling.

Figure 6-16 Constrained Polynomial Smoothing

6.5 Integrated Traffic Model

As discussed in Section 6.3 and Section 6.4, the Negative Binomial Model will be used to model the day-to-day variation, and the Weighted Exponential Polynomial Model will be used in modeling the daily traffic profile. Each movement will be modeled independently.

The algorithm of generating simulated traffic from 12 hours of counts is summarized below:

- Use the logarithm transformation to preprocess the counts y_i . The Transformed data are denoted as $z_i = \log(y_i)$.
- Split the 12 hours into three time periods representing AM, Midday, and PM.
- Identify the peak interval of each time period and add extra weights for the peak intervals. Since the peak time may last longer than 15 minutes, it is subjected to the engineer's judgment to give extra weight to the intervals around the peak 15 minutes.
- Fit the data z_i with the polynomial function of time intervals and the weights specified above. Obtain the fitted value $\hat{\mu}_i$
- Use the residual squares

$$e_i^2 = (z_i - \hat{\mu}_i)^2$$

as the estimation of the variance of the exponentially transformed data.

- Since the transformed counts are assumed to be normally distributed, the original counts can be approximated by the lognormal distribution. The estimator (Casella and Berger 2002) of the mean is

$$\hat{\mu}_i = \exp(\hat{\mu}_i + e_i^2 / 2)$$

and the estimator of the variance is

$$\hat{\sigma}_i^2 = [\exp(e_i^2) - 1] \exp(2\hat{\mu}_i + e_i^2).$$

- Calculate the over-dispersion factor $\hat{\lambda}_i = (\hat{\sigma}_i^2 - \hat{\mu}_i) / \hat{\mu}_i$. If $\hat{\lambda}_i < 0$, then let $\hat{\lambda}_i = 0$.
- Generate the mean values $l_{i,j}$ of the traffic according to $\text{Gamma}(1 / \hat{\lambda}_i, \hat{\mu}_i / \hat{\lambda}_i)$
- Since SimTraffic will generate traffic according to $\text{Poisson}(l_{i,j})$, the actual simulated traffic will be distributed as negative binomial distribution.

Figure 6-17 Actual 8 days' counts vs. Generated 8 days' counts

Figure 6-17 demonstrates an example of the above procedure. The upper graph shows day 1's actual counts in a solid line, the fitted curve of weighted exponential polynomial smoothing in a dashed line, and all eight days' counts in dots. Correspondingly, the lower graph shows eight days of simulated traffic in dots. It can be seen that the generated traffic volumes are quite similar to the actual counts. Figure 6-18 shows the result of applying the algorithm to the actual arterial system on US-52. The algorithm is satisfactory and will be used in the later stage of robustness evaluation.

With this algorithm in hand, 12 hours of traffic counts in one day can be expanded to multiple days of traffic volumes following the similar statistical distribution.

Figure 6-18 Generated traffic of US52 arterial system

7 ROBUSTNESS EVALUATION OF ALTERNATIVE PROCEDURES

As discussed in Section 6.1, the robustness of different signal optimization procedures will be evaluated in this chapter. To accomplish this goal, the following procedure was employed:

- Identify the representative volume set with the method.
- Input the set of volumes to the Synchro optimizer.
- Obtain the solution with the default settings of Synchro.
- Run the simulation of the obtained timing plan in SimTraffic. Use the generated volumes of multiple days

Usually the simulation is run under the optimized set of volume. The performance index obtained will be over-optimistic since the system is actually run under different levels of traffic demand. The above procedure provides a comprehensive performance measure since multiple intervals and multiple days of volumes are all incorporated in the simulation.

7.1 Alternative procedures

Three alternatives for selecting a representative volume set are examined. The alternatives are easily applicable without extra data and intensive computation.

The first method, called the baseline method, is the default method of Synchro. With 12 hours of traffic counts at hand, the traffic engineer can input the counts into Synchro using “transfer/data access/read volume/” and select the representative volume and the corresponding PHF automatically. Figure 7-1 shows the setting of such operations.

UDF Database Access

Read Volume | Write Volume | Timing | Phasing | Lane | Layout

Active File(s)

E:\Research\Project\variation\traffic model | **Select**

File Style: Single File comma delim (Volume.CSV)

Read

Limit Records By

Start Time From: 1330 To: 1800

Date From: 1-1-2000 To: 1-1-2000

Days Of Week:

Read Options

Method for Averaging Volume Counts:

Maximum Peak Hour Set PHF ☒

Scope

☐ Single Intersection SR 28 & Columbia

☐ Zone

☒ Entire Network

Close

Figure 7-1 Configuration of data access options

The second method, called the 110% method, is more conservative than the first one. All the volumes of the 12 hours are increased by 10% and then the default method of Synchro is applied. In other words, the timing plan is optimized not for the actual volumes but for the artificially-increased volumes.

The third method, called the time average method, seeks better splits among movements. Consider a simplest case of an intersection with only two movements. Movement 1's peak volume is 1,000 vph and the movement's peak volume is 500 vph. According to the default method, 1,000 and 500 vph will be selected as the representative volumes. However, there are two disadvantages of the default method: (1) the peak may not occur at the same time; and (2) the relative weight of traffic demand may deviate much from 2/1.

Using the time average method, first, the volumes for the two movements are summed up and the peak interval is identified based on the sum of traffic. For example, suppose that

the peak of the sum of the traffic is 1,450. Then, the average level of traffic of each movement during the relevant time period is calculated. Suppose movement 1's time average traffic level is 500 and movement 2's time average is 300. The peak volume of 1,450 vph is split according to the average traffic level. That is, instead using the ratio 1,000/500 to split 1,500 vph, the ratio 500/300 is used to split 1,450 vph.

$$y_1 = \frac{500}{500 + 300} \cdot 1450 \approx 906 \text{ vph}$$

$$y_2 = \frac{300}{500 + 300} \cdot 1450 \approx 544 \text{ vph}$$

When doing simulation with SimTraffic, instead of using the usual volume, multiple intervals of volumes should be used. First, check the Options/Data - Access/Data Options/Read Volume from the "UTDF Data File" box, and select the proper file of the volume and the day of simulation as in Figure 7-2. Second, use the "Options/Interval and Volumes" to insert extra intervals and select proper "Data Start Time" for each interval as in Figure 7-3.

Figure 7-2 SimTraffic Setting of Read Multiple Intervals of Volume

Intervals	0	1	2	3	4
Interval Name	Seeding	Recording			
Start time (hhmm)	05:57 A	06:00 A	06:15 A	06:30 A	06:45 A
Duration (min)	3	15	15	15	15
Record Statistics	No	Yes	Yes	Yes	Yes
Growth Factor Adjust	Yes	Yes	Yes	Yes	Yes
PHF Adjust	No	No	No	No	No
AntiPHF Adjust	No	No	No	No	No
Percentile Adjust	No	No	No	No	No
Percentile Adjust (%ile)	—	—	—	—	—
Timing Plan ID	—	—	—	—	—
Data Start Time (hhmm)	0600	0600	0615	0630	0645

Random Number Seed:

Enter time for volume data in data file.

Figure 7-3 Settings of Multiple Intervals in SimTraffic

7.2 Evaluated Systems

Three real systems are tested in this study. They are all systems in the Crawfordsville District of INDOT. Each system is evaluated in a different time periods.

7.2.1 AM of US-52 at Cumberland Avenue

Figure 7-4 Layout of US-52 at Cumberland Avenue

7.2.2 Midday of SR-28 at SR-39

Figure 7-5 Layout of SR-28 at SR-39

7.2.3 PM of US-36 at SR-39

Figure 7-6 Layout of US-36 at SR-39

7.3 Results

The performance measure used for the comparison are the total stops, total delay, arterial delay, and arterial speed for the entire simulated time periods and stops and delays for the peak interval in the simulated time periods. These measures were selected to approximate the concerns of the INDOT traffic systems engineers as summarized in Section 4.3.1. Five days of traffic are generated and evaluated under the plan generated for each alternative. The results are presented below.

7.3.1 AM of US-52 at Cumberland Avenue

Baseline								
		Day1	Day2	Day3	Day4	Day5	Average	Stand dev
	Total Stops	4978.00	4801.00	5209.00	5001.00	5097.00	5017.20	151.41
	Veh Exited	8027.00	7905.00	8231.00	8056.00	7940.00	8031.80	127.27
	Stop/Veh	0.62	0.61	0.63	0.62	0.64	0.62	0.01
	Total Delay (hr)	58.10	54.50	58.80	57.80	57.60	57.36	1.66
	Delay/Veh (sec)	26.06	24.82	25.72	25.83	26.12	25.71	0.52
SB	Arterial delay/veh (sec)	24.60	22.20	23.80	23.10	23.80	23.50	0.90
SB	Arterial Speed (mph)	42.00	42.00	42.00	42.00	41.00	41.80	0.45
WB	Arterial delay/veh (sec)	20.50	19.50	20.60	21.80	22.40	20.96	1.15
WB	Arterial Speed (mph)	39.00	40.00	40.00	39.00	38.00	39.20	0.84
Peak Interval	Veh Exited	930.00	917.00	907.00	873.00	868.00	899.00	27.32
	Total Stops	600.00	578.00	649.00	595.00	558.00	596.00	33.89
	Stop/Veh	0.65	0.63	0.72	0.68	0.64	0.66	0.03
	Total Delay (hr)	8.30	7.00	8.10	6.90	7.10	7.48	0.66
	Delay/Veh (sec)	32.13	27.48	32.15	28.45	29.45	29.93	2.13

Table 7-1 Performance of Baseline method of US-52 system

Time Average								
		Day1	Day2	Day3	Day4	Day5	Average	Stand dev
	Total Stops	4922.00	4800.00	5239.00	4984.00	4881.00	4965.20	167.01
	Veh Exited	7958.00	7875.00	8195.00	8064.00	7935.00	8005.40	126.08
	Stop/Veh	0.62	0.61	0.64	0.62	0.62	0.62	0.01
	Total Delay (hr)	59.20	57.90	63.60	59.80	58.70	59.84	2.21
	Delay/Veh (sec)	26.78	26.47	27.94	26.70	26.63	26.90	0.59
SB	Arterial delay/veh (sec)	22.70	21.50	24.10	21.50	21.30	22.22	1.19
SB	Arterial Speed (mph)	43.00	43.00	42.00	43.00	43.00	42.80	0.45
WB	Arterial delay/veh (sec)	21.70	19.90	22.00	21.30	22.70	21.52	1.04
WB	Arterial Speed (mph)	38.00	39.00	39.00	39.00	38.00	38.60	0.55
Peak Interval	Veh Exited	934.00	914.00	891.00	871.00	875.00	897.00	26.71
	Total Stops	631.00	556.00	657.00	610.00	545.00	599.80	48.14
	Stop/Veh	0.68	0.61	0.74	0.70	0.62	0.67	0.05
	Total Delay (hr)	8.30	7.40	9.00	7.80	7.40	7.98	0.68
	Delay/Veh (sec)	31.99	29.15	36.36	32.24	30.45	32.04	2.72

Table 7-2 Performance of Time Average method of US-52 system

110% Volume								
		Day1	Day2	Day3	Day4	Day5	Average	Stand dev
	Total Stops	5085.00	4971.00	5188.00	4929.00	4689.00	4972.40	187.99
	Veh Exited	8043.00	7887.00	8291.00	8049.00	7699.00	7993.80	219.15
	Stop/Veh	0.63	0.63	0.63	0.61	0.61	0.62	0.01
	Total Delay (hr)	56.00	56.30	58.70	54.90	52.70	55.72	2.18
	Delay/Veh (sec)	25.07	25.70	25.49	24.55	24.64	25.09	0.50
SB	Arterial delay/veh (sec)	22.70	22.00	23.10	21.90	20.60	22.06	0.96
SB	Arterial Speed (mph)	43.00	42.00	42.00	43.00	43.00	42.60	0.55
WB	Arterial delay/veh (sec)	19.60	20.80	20.20	20.20	19.10	19.98	0.65
WB	Arterial Speed (mph)	40.00	39.00	40.00	39.00	40.00	39.60	0.55
Peak Interval	Veh Exited	853.00	864.00	880.00	903.00	849.00	869.80	22.11
	Total Stops	563.00	533.00	595.00	536.00	495.00	544.40	37.24
	Stop/Veh	0.66	0.62	0.68	0.59	0.58	0.63	0.04
	Total Delay (hr)	6.60	7.00	7.10	6.40	5.90	6.60	0.48
	Delay/Veh (sec)	27.85	29.17	29.05	25.51	25.02	27.32	1.95

Table 7-3 Performance of 110% method of US-52 system

Summary		Baseline	Time Average	110%
	Total Stops	5017.2	4965.2	4972.4
	Veh Exited	8031.8	8005.4	7993.8
	Stop/Veh	0.6	0.6	0.6
	Total Delay (hr)	57.4	59.8	55.7
	Delay/Veh (sec)	25.7	26.9	25.1
SB	Arterial delay/veh (sec)	23.5	22.2	22.1
SB	Arterial Speed (mph)	41.8	42.8	42.6
WB	Arterial delay/veh (sec)	21.0	21.5	20.0
WB	Arterial Speed (mph)	39.2	38.6	39.6
Peak Interval	Veh Exited	899.0	897.0	869.8
	Total Stops	596.0	599.8	544.4
	Stop/Veh	0.7	0.7	0.6
	Total Delay (hr)	7.5	8.0	6.6
	Delay/Veh (sec)	29.9	32.0	27.3

Table 7-4 Summary of the results of US-52 system

7.3.2 Midday of SR-28 at SR-39

Baseline								
EB EB WB WB		Day1	Day2	Day3	Day4	Day5	Average	Stand dev
	Total Stops	21732.00	22264.00	23555.00	21316.00	19947.00	21762.80	1319.18
	Veh Exited	12363.00	12843.00	12696.00	12455.00	12576.00	12586.60	190.48
	Stop/Veh	1.76	1.73	1.86	1.71	1.59	1.73	0.10
	Total Delay (hr)	794.50	702.50	751.20	592.00	366.80	641.40	171.08
	Delay/Veh (sec)	231.35	196.92	213.01	171.11	105.00	183.48	49.13
	Arterial delay/veh (sec)	69.40	142.90	235.30	211.20	75.20	146.80	76.01
	Arterial Speed (mph)	14.00	12.00	10.00	12.00	14.00	12.40	1.67
	Arterial delay/veh (sec)	48.50	40.70	46.90	46.10	42.00	44.84	3.33
	Arterial Speed (mph)	16.00	17.00	16.00	16.00	17.00	16.40	0.55
Peak Interval	Veh Exited	845.00	856.00	754.00	868.00	840.00	832.60	45.24
	Total Stops	1726.00	1889.00	1606.00	1624.00	1607.00	1690.40	121.65
	Stop/Veh	2.04	2.21	2.13	1.87	1.91	2.03	0.14
	Total Delay (hr)	74.70	67.50	77.00	77.50	40.20	67.38	15.71
	Delay/Veh (sec)	318.25	283.88	367.64	321.43	172.29	292.70	73.61

Table 7-5 Performance of Baseline method of SR-28 system

Time Average								
		Day1	Day2	Day3	Day4	Day5	Average	Stand dev
EB EB WB WB	Total Stops	23205.00	22293.00	24148.00	23054.00	21350.00	22810.00	1049.37
	Veh Exited	12577.00	12632.00	12823.00	12501.00	12599.00	12626.40	120.00
	Stop/Veh	1.85	1.76	1.88	1.84	1.69	1.81	0.08
	Total Delay (hr)	466.10	952.20	663.40	972.90	546.10	720.14	232.26
	Delay/Veh (sec)	133.41	271.37	186.25	280.17	156.04	205.45	66.95
	Arterial delay/veh (sec)	181.40	188.10	185.20	403.90	111.20	213.96	110.90
	Arterial Speed (mph)	11.00	11.00	10.00	10.00	12.00	10.80	0.84
	Arterial delay/veh (sec)	50.10	46.60	52.60	51.30	48.50	49.82	2.35
	Arterial Speed (mph)	16.00	16.00	15.00	15.00	16.00	15.60	0.55
Peak Interval	Veh Exited	823.00	828.00	818.00	857.00	800.00	825.20	20.68
	Total Stops	1980.00	1785.00	1783.00	1596.00	1587.00	1746.20	162.35
	Stop/Veh	2.41	2.16	2.18	1.86	1.98	2.12	0.21
	Total Delay (hr)	49.10	81.30	69.40	115.80	50.90	73.30	27.26
	Delay/Veh (sec)	214.78	353.48	305.43	486.44	229.05	317.83	109.95

Table 7-6 Performance of Time Average method of SR-28 system

110 Percent								
		Day1	Day2	Day3	Day4	Day5	Average	Stand dev
EB EB WB WB	Total Stops	24074.00	23991.00	23126.00	21784.00	22089.00	23012.80	1055.84
	Veh Exited	12609.00	12890.00	12723.00	12518.00	12567.00	12661.40	148.55
	Stop/Veh	1.91	1.86	1.82	1.74	1.76	1.82	0.07
	Total Delay (hr)	768.10	970.50	581.10	969.00	494.80	756.70	218.14
	Delay/Veh (sec)	219.30	271.05	164.42	278.67	141.74	215.04	61.52
	Arterial delay/veh (sec)	226.40	684.40	324.40	733.40	262.60	446.24	242.94
	Arterial Speed (mph)	10.00	9.00	10.00	10.00	11.00	10.00	0.71
	Arterial delay/veh (sec)	61.30	58.10	64.10	62.10	68.60	62.84	3.88
	Arterial Speed (mph)	14.00	15.00	14.00	14.00	13.00	14.00	0.71
Peak Interval	Veh Exited	816.00	796.00	800.00	824.00	792.00	805.60	13.74
	Total Stops	1864.00	1873.00	1803.00	1731.00	1996.00	1853.40	97.92
	Stop/Veh	2.28	2.35	2.25	2.10	2.52	2.30	0.15
	Total Delay (hr)	86.30	133.40	77.00	144.10	56.80	99.52	37.56
	Delay/Veh (sec)	380.74	603.32	346.50	629.56	258.18	443.66	164.20

Table 7-7 Performance of 110% method of SR-28 system

Summary		Baseline	Time Average	110 Percent
	Total Stops	21762.80	21350.00	23012.80
	Veh Exited	12586.60	12599.00	12661.40
	Stop/Veh	1.73	1.69	1.82
	Total Delay (hr)	641.40	546.10	756.70
	Delay/Veh (sec)	183.48	156.04	215.04
	EB Arterial delay/veh (sec)	146.80	111.20	446.24
	EB Arterial Speed (mph)	12.40	12.00	10.00
	WB Arterial delay/veh (sec)	44.84	48.50	62.84
	WB Arterial Speed (mph)	16.40	16.00	14.00
Peak Interval	Veh Exited	832.60	800.00	805.60
	Total Stops	1690.40	1587.00	1853.40
	Stop/Veh	2.03	1.98	2.30
	Total Delay (hr)	67.38	50.90	99.52
	Delay/Veh (sec)	292.70	229.05	443.66

Table 7-8 Summary of SR-28 system

7.3.3 PM of US-36 at SR-39

Baseline								
		Day1	Day2	Day3	Day4	Day5	Average	Stand dev
	Total Stops	6922.0	6736.0	6846.0	6649.0	7221.0	6874.8	219.8
	Veh Exited	8777.0	8514.0	8685.0	8495.0	8880.0	8670.2	166.4
	Stop/Veh	0.8	0.8	0.8	0.8	0.8	0.8	0.0
	Total Delay (hr)	66.3	64.3	66.6	63.0	71.1	66.3	3.1
	Delay/Veh (sec)	27.2	27.2	27.6	26.7	28.8	27.5	0.8
	EB Arterial delay/veh (sec)	26.8	27.2	27.7	27.3	30.4	27.9	1.4
	EB Arterial Speed (mph)	28.0	28.0	28.0	28.0	27.0	27.8	0.4
	WB Arterial delay/veh (sec)	29.5	28.1	28.9	27.6	30.2	28.9	1.0
	WB Arterial Speed (mph)	26.0	26.0	26.0	26.0	26.0	26.0	0.0
Peak Interval	Veh Exited	521.0	455.0	475.0	463.0	567.0	496.2	47.1
	Total Stops	398.0	361.0	366.0	364.0	471.0	392.0	46.6
	Stop/Veh	0.8	0.8	0.8	0.8	0.8	0.8	0.0
	Total Delay (hr)	4.4	3.6	3.6	3.3	5.1	4.0	0.7
	Delay/Veh (sec)	30.4	28.5	27.3	25.7	32.4	28.8	2.6

Table 7-9 Performance of method of US-36 system

Time Average								
EB EB WB WB		Day1	Day2	Day3	Day4	Day5	Average	Stand dev
	Total Stops	6911.0	6695.0	6976.0	6563.0	7040.0	6837.0	200.8
	Veh Exited	8771.0	8501.0	8789.0	8434.0	8822.0	8663.4	181.3
	Stop/Veh	0.8	0.8	0.8	0.8	0.8	0.8	0.0
	Total Delay (hr)	66.2	64.6	68.8	61.5	68.6	65.9	3.0
	Delay/Veh (sec)	27.2	27.4	28.2	26.3	28.0	27.4	0.8
	Arterial delay/veh (sec)	27.3	27.0	27.5	26.4	29.7	27.6	1.3
	Arterial Speed (mph)	28.0	28.0	28.0	28.0	27.0	27.8	0.4
	Arterial delay/veh (sec)	29.1	27.7	30.3	28.0	29.6	28.9	1.1
	Arterial Speed (mph)	27.0	27.0	26.0	26.0	26.0	26.4	0.5
Peak Interval	Veh Exited	523.0	454.0	470.0	449.0	551.0	489.4	45.2
	Total Stops	404.0	364.0	375.0	363.0	482.0	397.6	50.0
	Stop/Veh	0.8	0.8	0.8	0.8	0.9	0.8	0.0
	Total Delay (hr)	4.2	3.6	3.8	3.3	4.9	4.0	0.6
	Delay/Veh (sec)	28.9	28.5	29.1	26.5	32.0	29.0	2.0

Table 7-10 Performance of method of US-36 system

110 Percent								
EB EB WB WB		Day1	Day2	Day3	Day4	Day5	Average	Stand dev
	Total Stops	6985.0	6739.0	7126.0	6672.0	7256.0	6955.6	248.7
	Veh Exited	8762.0	8511.0	8815.0	8546.0	8854.0	8697.6	158.3
	Stop/Veh	0.8	0.8	0.8	0.8	0.8	0.8	0.0
	Total Delay (hr)	66.9	65.2	70.1	63.1	70.1	67.1	3.1
	Delay/Veh (sec)	27.5	27.6	28.6	26.6	28.5	27.8	0.8
	Arterial delay/veh (sec)	27.6	27.3	28.3	26.7	30.0	28.0	1.3
	Arterial Speed (mph)	28.0	28.0	28.0	28.0	27.0	27.8	0.4
	Arterial delay/veh (sec)	29.5	29.0	30.9	28.3	30.0	29.5	1.0
	Arterial Speed (mph)	26.0	26.0	26.0	26.0	26.0	26.0	0.0
Peak Interval	Veh Exited	523.0	453.0	462.0	459.0	549.0	489.2	43.8
	Total Stops	394.0	361.0	386.0	345.0	473.0	391.8	49.4
	Stop/Veh	0.8	0.8	0.8	0.8	0.9	0.8	0.0
	Total Delay (hr)	4.2	3.6	3.6	3.2	4.8	3.9	0.6
	Delay/Veh (sec)	28.9	28.6	28.1	25.1	31.5	28.4	2.3

Table 7-11 Performance of method of US-36 system

Summary		Baseline	plus 10%	Time Average
EB EB WB WB	Total Stops	6874.8	6955.6	6837.0
	Veh Exited	8670.2	8697.6	8663.4
	Stop/Veh	0.8	0.8	0.8
	Total Delay (hr)	66.3	67.1	65.9
	Delay/Veh (sec)	27.5	27.8	27.4
	Arterial delay/veh (sec)	27.9	28.0	27.6
	Arterial Speed (mph)	27.8	27.8	27.8
	Arterial delay/veh (sec)	28.9	29.5	28.9
	Arterial Speed (mph)	26.0	26.0	26.4
Peak Interval	Veh Exited	496.2	489.2	489.4
	Total Stops	392.0	391.8	397.6
	Stop/Veh	0.8	0.8	0.8
	Total Delay (hr)	4.0	3.9	4.0
	Delay/Veh (sec)	28.8	28.4	29.0

Table 7-12 Summary results of US-36 system

7.3.4 Summary of Results

From Table 7-4, Table 7-8, and Table 7-12, it is noticed that none of the two alternatives proposed significantly and consistently improve the simulated performance of the initial solution of Synchro. The total number of stops and the time of delay results are illustrated in Figure 7-7 and Figure 7-8. Therefore, the current practice of using Synchro is shown to have acceptable robustness under varied traffic levels.

It should be noted, however, that the confirmation of the robustness of the current method does not mean that the current method is optimal in terms of robustness. It only means that no easy method is available to significantly improve the robustness consistently. Careful tuning of the solutions is of great value and should not be overlooked.

Total Stops	Base	Time Average	110%
US52	5017.2	4965.2	4972.4
Frankfort city	21762.80	21350.00	23012.80
Danville	6874.8	6955.6	6837.0

Table 7-13 Summary of Average Total Stops

Figure 7-7 Comparison of Average Total Stops

Total Delay	Base	Time Average	110%
US52	57.4	59.8	55.7
Frankfort city	641.40	546.10	756.70
Danville	66.3	67.1	65.9

Table 7-14 Summary of Average Total Delay

Figure 7-8 Comparison of Average Total Stops

8 CONCLUSIONS

The major objective of this research was to reduce the time and effort needed in solution tuning and thus improve the effectiveness and the efficiency of using Synchro/SimTraffic in arterial signal coordination design.

First, the INDOT criteria of good coordination was surveyed and documented. INDOT engineers follow a more perception-based approach to coordination design that exceeds the routine optimization objective of Synchro and other commercial optimization packages. An arterial signal plan is acceptable to INDOT engineers when the following conditions of good coordination are satisfied:

- Traffic progression along the arterial is reasonably smooth. Most of the drivers moving along the arterial street do not stop at two consecutive intersections. Drivers do not stop more than once at the same intersection (no congestion).
- No perceptible queuing interaction persists during the design period. There is no queue spillback affecting queue discharge at the upstream intersection. Through lanes on the arterial street are not blocked by adjacent queues of turning vehicles.
- If the studied street is modernized, the average arterial travel time is shorter or at least comparable to that before modernization.

The design process usually progresses through a sequence of iterations. The above conditions are used repeatedly during the design process to evaluate the intermediate signal solutions and to decide if the design is complete or should be continued.

Second, the issues encountered in the designing process were collected and classified. The INDOT engineers' methods to address these issues were investigated. Other possible addressing methods were also explored. The outcomes of these investigations are assembled into the second volume of this report, Guidelines of Design.

The guidelines are in accordance with the INDOT-defined criteria of good signal control along urban streets. The guidelines are not a manual of arterial signal design since every arterial system has its individual problems and requires specific treatment which cannot be reached by a uniform set of procedures. Instead, experiences collected from current traffic systems engineers and lessons learned through research are compiled into the guidelines to help reduce field-tuning of the signal settings. The resulting guidelines are practical and they require no extra data beyond the current data collection practice. The guideline can serve as a reference for experienced traffic engineers and as an educational tool for new traffic engineers.

Third, the robustness of the signal design procedures was also investigated. Models for extracting the traffic variation pattern from 12 hours of traffic counts were developed based on real data collected at Purdue University. These models were used to generate reasonable traffic inputs to the micro-simulation tool. The robustness of the current signal design procedures was evaluated using these simulated traffic inputs. Several practical alternatives to the current arterial signal optimization were also evaluated using the same inputs. The performances of these methods were compared, and it was concluded that no obvious way to consistently improve the robustness of current signal design procedure is available.

LIST OF REFERENCES

- Casella, G. and R. Berger (2002). Statistical Inference, Duxbury.
- Chang, E. C. and C. J. Messer (1991). Arterial Signal Timing Optimization Using PASSER II-90 – Program User Manual.
- Daganzo, C. F. (2004). In Traffic Flow, Cellular Automata = Kinematic Waves., Institute of Transportation Studies.
- Fox, K. and S. Clark (1998). "Evaluating the Benefits of a Responsive UTC System Using Micro-Simulation." Institute for Transport Studies, University of Leeds, Leeds, UK. (Working Paper).
- Foy, M. D., R. F. Benekohal, et al. (1993). "Signal Timing Determination Using Genetic Algorithms." Transportation Research. Record(1365): 108-115.
- Gartner, N. H. (1983). "OPAC: A DEMAND-RESPONSIVE STRATEGY FOR TRAFFIC SIGNAL CONTROL." Transportation Research Record(906): 75-81.
- Gartner, N. H., S. F. Assmann, et al. (1990). "MULTIBAND--A VARIABLE-BANDWIDTH ARTERIAL PROGRESSION SCHEME." Transportation Research Record(1287).
- Gartner, N. H. and P. Wagner (2004). "ANALYSIS OF TRAFFIC FLOW CHARACTERISTICS ON SIGNALIZED ARTERIALS." Transportation Research Record (1883): 94-100.
- Gazis, D. C. (2002). Traffic Theory, Kluwer Academic Pub.
- Goldberg, D. E. (1989). Genetic Algorithms in search, Optimization and Machine learning. Massachusetts, Addison-Wesley Publishing Co.
- Hunt, P. B., D. I. Robertson, et al. (1991). SCOOT: A traffic responsive method of coordinating signals. Crowthorne, Transport and Road Research Laboratory.
- Husch, D. and J. Albeck (1993-2004). Synchro 6 User Guide. 1009B Solano Avenue, Albany, CA 94706, Trafficware Ltd.
- Kamarajugadda, A. and B. Park (2003). Stochastic Traffic Signal Timing Optimization, Center for ITS Implementation Research. University of Virginia.

- Lewis, P. and G. Shedler (1979). "Simulation of Nohhomogeneous Poisson Process with Degree-Two Exponential Polynomial Rate Function." Operations Research **27**(5): 1026-1040.
- Lieu, H. C. and S.-M. Lin (2004). " BENEFIT ASSESSMENT OF IMPLEMENTING WEATHER-SPECIFIC SIGNAL TIMING PLANS BY USING CORSIM." Transportation Research Record(1867): 202-209.
- Little, J., M. D. Kelson, et al. (1981). "MAXBAND: A PROGRAM FOR SETTING SIGNALS ON ARTERIES AND TRIANGULAR NETWORKS." Transportation Research Record(795): 40-46.
- Mannering, F. (1988). Compensating variation approach to optimal traffic signal timings. Spring Meeting of the Operations Research Society of America, Washington, DC.
- McTransCenter (1990-2006). Transyt7F Hardcopy Users Guide, University of Florida.
- Messer, C. J., R. H. Whitson, et al. (1973). "A Variable-Sequence Multiphase Progression Optimization Program." Highway Research Record (445): 24-33.
- Morgan, J. T. and J. D. C. Little (1964). "Synchronizing Traffic Signals for Maximal Bandwidth." Operations Research **12**(6, Special Transportation Science Issue): 896-912.
- Nagel, K. and M. Schreckenberg (1992). "A Cellular Automaton Model for Freeway Traffic." J. Physique I **2**(12): 2221-2229.
- Nagel, K., D. E. Wolf, et al. (1998). "Two-Lane Traffic Rules for Cellular Automata: A Systematic Approach." Physical Rev. E **58**(2): 1425-1437.
- Nelson, E. J. and D. Bullock (2000). "IMPACT OF EMERGENCY VEHICLE PREEMPTION ON SIGNALIZED CORRIDOR OPERATION: AN EVALUATION." Transportation Research Record(1727): 1-11.
- Oppenheim, A. and R. Schafer (1975). Digital Signal Processing, Prentice Hall.
- Park, B., C. J. Messer, et al. (1999). "Traffic Signal Optimization Program for Oversaturated Conditions: A Genetic Algorithm Approach." Transportation Research Record (1683): 133-142.
- Park, B., C. J. Messer, et al. (2000). "ENHANCED GENETIC ALGORITHM FOR SIGNAL-TIMING OPTIMIZATION OF OVERSATURATED INTERSECTIONS." Transportation Research Record(1727).
- Perez-Cartagena, R. I. and A. P. Tarko (2004). Predicting Traffic Conditions at Indiana Signalized Intersections. West Lafayette, Purdue University.
- Prigogine, I. and R. Herman (1971). Kinetic Theory of Vehicular Traffic. Amsterdam,

Elsevier.

- Shelby, S. G. (2004). "SINGLE-INTERSECTION EVALUATION OF REAL-TIME ADAPTIVE TRAFFIC SIGNAL CONTROL ALGORITHMS." Transportation Research Record(1867): 183-192.
- Skabardonis, A. (2000). "CONTROL STRATEGIES FOR TRANSIT PRIORITY." Transportation Research Record(1727): 20-26.
- Sunkari, S. (2004). "The benefits of retiming traffic signals." ITE Journal **74**(4): 26-29.
- Theil, H. and A. Goldberger (1961). " On Pure and Mixed Statistical Estimation in Economics." International Economic Review **2**(1): 65-78.
- TRB (2000). Highway Capacity Manual 2000. Washington D.C, Transportation Research Board, National Research Council.
- USDOT (1996). CORSIM User Manual Version 1.01, Federal Highway Administration.
- Washington, S., M. Karlaftis, et al. (2003). Statistical and Econometric Methods for Transportation Data Analysis, Chapman & Hall/CRC Press.
- Webster (1958). "Road Research Technical Paper 39: Traffic Signal Settings." U.K. Transport and Road Research Laboratory, Crowthorne, Berkshire, England.
- Yin, Y., S. M. Madanat, et al. (2005). Robust Improvement Schemes for Road Networks under Demand Uncertainty. Transportation Research Board 84th Annual Meeting, Washington, DC.