University of Nevada, Reno

Measuring the Quality of Arterial Traffic Signal Timing – A Trajectory-based Methodology

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ABSTRACT

Evaluating the benefits from traffic signal timing is of increasing interest to transportation policymakers, operators, and the public as integrating performance measurements with agencies' daily signal timing management has become a top priority. This dissertation presents a trajectory-based methodology for evaluating the quality of arterial signal timing, a critical part of signal operations that promises reduced travel time and fewer vehicle stops along arterials as well as improved travelers' perception of transportation services. The proposed methodology could significantly contribute to performance-oriented signal timing practices by addressing challenges regarding which performance measures should be selected, how performance measurements can be performed cost-effectively, and how to make performance measures accessible to people with limited knowledge of traffic engineering.

A review of the current state of practice and research was conducted first, indicating an urgent research need for developing an arterial-level methodology for signal timing performance assessments as the established techniques are mostly based on by-link or by-movement metrics. The literature review also revealed deficiencies of existing performance measures pertaining to traffic signal timing. Accordingly, travel-run speed and stop characteristics, which can be extracted from vehicle GPS trajectories, were selected to measure the quality of arterial signal timing in this research.

Two performance measures were then defined based on speed and stop characteristics: the attainability of ideal progression (AIP) and the attainability of user satisfaction (AUS). In order to determine AIP and AUS, a series of investigations and surveys were conducted to characterize the effects of non-signal-timing-related factors (e.g., arterial congestion level) on average travel speed as well as how stops may affect travelers' perceived quality of signal timing. AIP was calculated considering the effects of non-signal-timing-related factors, and AUS accounted for the changes in the perceived quality of signal timing due to various stop circumstances.

Based upon AIP and AUS, a grade-based performance measurement methodology was developed. The methodology included AIP scoring, AUS scoring, and two scoring adjustments. The two types of scoring adjustments further improved the performance measurement results considering factors such as cross-street delay, pedestrian delays, and arterial geometry.

Furthermore, the research outlined the process for implementing the proposed methodology, including the necessary data collection and the preliminary examination of the applicable conditions. Case studies based on real-world signal re-timing projects were presented to demonstrate the effectiveness of the proposed methodology in enhancing agencies' capabilities of cost-effectively monitoring the quality of arterial signal timing, actively addressing signal timing issues, and reporting the progress and outcomes in a concise and intuitive manner.

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Chapter 1 INTRODUCTION

1.1 Overview

For the purpose of improving urban arterial operations, a methodology for measuring the quality of arterial traffic signal timing and the pursuit of its implementation are presented in this dissertation by leveraging travel-run trajectories. This chapter includes discussions of the motivation and the objectives of the research. The potential contributions and impacts of this research are then described, followed by the organization of the dissertation.

1.2 Research Motivation

Traffic signal control, a way to assign the right-of-way to various traffic and pedestrian movements at intersections, has been in place for more than 100 years in the United States. Today, there are more than 330,000 traffic signals scattered across the nation [1], and numerous miles driven on the road are influenced by traffic signal operations [2]. The amount of traffic delay at signalized intersections has been steadily increasing over the past 20 years [3], which currently can contribute to an estimated 5 to 10 percent of overall travel delay, equivalent to 295 million vehicle-hours annually on major roadways alone [4]. Inferior traffic operations often correlate with inefficient performances of transportation systems, resulting in congestion, excess fuel consumption, and unwanted vehicular emissions. Besides, travelers can perceive the poor quality of the transportation system operations while being stopped at traffic signals. Due to the profound effects on transportation system efficiency as well as travelers' perceptions of transportation service quality, improving traffic signal operations has been regarded as one of the most rewarding endeavors of urban traffic management.

Traffic signal timing is crucial for signal operations, which involves determining a variety of parameters, such as the allocation of green times among vehicular, pedestrian, and transit traffic by movement or approach, the sequential order of signal control phases, and the time relationship of timing plan operations between adjacent intersections. High-quality signal timings allow all users of transportation systems to traverse the intersection(s) safely and efficiently. There has been an increased interest in improving traffic signal timing due to the remarkable payoffs in time and environmental benefits with benefit-cost ratios ranging between 20:1 and 55:1, which can translate into tens of millions of dollar savings annually in the U.S. [5]

Nevertheless, the outcomes remain far from ideal, despite considerable efforts spent on improving signal timing over recent decades. A nationwide survey evaluated the signal timing practices in the U.S. and only assigned a grade of "C," indicating that the signal operations across the country were barely performing at adequate levels [3]. The report pinpointed how this unsatisfactory grade was determined, as stated in the following sentence:

"Traffic signal timing performance is not regularly measured in connection to objectives, resulting in outdated timing patterns that do not reflect current traffic

and pedestrian needs, and coordinated signals may force travelers to stop at multiple adjacent intersections" [3].

As described above, performance measurements should be brought to the forefront of attention, which can link the day-to-day practices to the operational objectives. In the view of many transportation agencies who superficially consider signal timing as a process of parametric calculations, the performance of signal timing is determined after the software or model outputs have been implemented to field signal controllers; however, discrepancies can be observed in practice often as traffic patterns are versatile over time, which implies that the performance of signal timing is dynamic along with the change of traffic demands and flow profiles on movements. Consequently, regular performance measurements are necessary in order to continue operating traffic signals at their best levels.

Effective performance assessments require a decent amount of observation and study, where a high-standard set of expertise and resources is needed. As for performance evaluations of traffic signal coordination, the process still mostly relies on manual observations, which is often a costly and labor-intensive procedure conducted by a group of people standing at intersections to observe traffic flow or driving along arterials repetitiously. In addition, manual observations may not be capable of characterizing the overall performance but only provide partial information on the control effects. Practitioners may have to use clear thinking and careful reasoning to deal with many varieties of data gathered from the field for identifying the cause of signal timing problems. Due to a lack of sufficient resources and competent staff, many agencies usually schedule signal re-timing based on citizen complaints. Then the improvements are often arbitrary and overdue.

Furthermore, because of inadequate or inaccurate reporting from agencies, the transportation policymakers and elected officials may underestimate the need to re-time traffic signals when the fund is budgeted. The agency would sequentially encounter increased difficulties in measuring the quality of arterial signal timing in the future, as a vicious circle created.

Although most agencies across the nation realize the importance of signal timing performance measurements, they have not yet incorporated performance measurements into their daily practice because of technical, operational, and budgetary concerns. There has been an ongoing debate regarding what data collection techniques can be used and how local agencies can perform the measurements when funding and staffing are tight. A performance measurement methodology is needed, which can assist signal timing engineers and technicians in monitoring and improving arterial signal operations through accessible data sources. Most importantly, the performance measurement methodology can be implemented in a convenient and affordable manner.

1.3 Objectives and Scope

This research aims to propose a performance measurement methodology for arterial signal timing analysis. The primary objectives of this research include:

1) Establish a portfolio of standards (performance measures) based on vehicle travel-run trajectory data to assess the quality of signal timing concerning arterial-level operations;
2) Define a grade-based evaluation framework according to the proposed performance measures, describing the quality of arterial signal timing through the use of intuitive and understandable language in an effort to apprise decision-makers and the public; and
3) Provide a scheme to implement the proposed performance measurement methodology in real-world operations.

The performance measurements of signal timing can be conducted at different levels, such as at isolated intersections, along signalized arterials, or for grid networks. This dissertation mainly focuses on developing a methodology for measuring the quality of signal timing regarding arterial-level operational considerations (e.g., platoon progression and driving experience along the arterial). The dissertation does not include discussions pertaining to isolated intersections or grid networks; however, it should be noted that the quality of signal timing at isolated intersections (e.g., whether capacity is adequately allocated to each traffic movement or not) can affect the quality of arterial signal timing meanwhile achieving network-level operational objectives (e.g., reducing system delay) is based on optimizing the quality of signal timing along arterials.

The methodology presented in this dissertation can be used for unsaturated arterial segments. When oversaturation recurrently happens, unique signal timing approaches should be adopted where different performance measurements need to be performed. Such performance measurements are not included in this dissertation.

In addition, it is assumed that all signal timings have met the safety requirements prior to being assessed. Therefore, analyses of yellow and red intervals timing are beyond the scope of this dissertation.

1.4 Contribution and Significance

This research was inspired by the significant signal timing challenges described in a report published by the Federal Highway Administration (FHWA) [6]. At present, these challenges are still unsolved, which have hindered the signal timing practices. Table 1.1 presents these significant limitations and specifies how these limitations can influence the current arterial signal timing practices.

Table 1.1 Current Signal Timing Challenges

Limitations	Specifics during Timing Signals for Arterial Operations
No Standard Established for Good Operations	The widely adopted measures, such as reductions of travel time or the number of stops, can be used to demonstrate the improvements through before-after analyses; however, whether the "best" performance has been attained or not is still unknown. Active management is impossible without the definitions of good operations, and accordingly, citizen complaints become the de facto traction toward signal timing improvements [7].
Inadequate Performance Measurements Due to Limited Resources	Arterial-level signal timing performance measurements usually require a massive amount of work, including manual observations and floating-car studies. Many agencies hardly or arbitrarily evaluate the performance of arterial-level signal operations due to budgetary and human-resource limitations
A Lack of Practical Methodologies and Applications	Very few software packages are currently available for signal coordination development [8,9,10], especially among which almost no one is capable of measuring the quality of arterial signal timing.
Ineffectively Reporting the Progress and Outcomes	Practitioners face difficulties in articulating the progress of arterial signal timing practices with intuitive language, which may mislead the decision-makers and the public who substantially define the returns on investments and decide the future funding allocation.

The research presented in this dissertation can help agencies across the nation in addressing the abovementioned limitations by the following ways:

1) The research proposes a methodology for measuring the quality of arterial signal timing based on various considerations, including progression quality and traveler perception. The quality of signal timing links signal timing design to arterial operations and perceived quality of service, which can guide signal timing improvement projects to reduce arterial travel delays, stops, and potential citizen complaints. For developing the proposed methodology,

the effects of non-signal-timing factors (e.g., arterial congestion levels, arrival profiles, and geometry constraints) on arterial operations have been considered, which makes the resulting quality of signal timing more informative and accurate compared to the conventional signal timing performance measurements. For example, when congestion occurs, signal timing is usually considered problematic through the conventional performance measurements; however, the poor performance can be primarily due to the factors such as oversaturated traffic demands or insufficient infrastructure capacity. The proposed methodology addresses the signal timing aspect explicitly, yielding unbiased performance assessments to assist transportation system operators and decision-makers in properly choosing congestion relief strategies beyond signal timing, such as widening roadways to increase capacity and promoting public transit and car-free transportation modes to reduce demands;

2) The proposed methodology provides a grade-based evaluation framework, which can be easily understood by signal timing project stakeholders, the elected officials, and the public who have limited signal timing knowledge. This can facilitate the budgeting, the progress reporting, and the public involvement processes during the signal timing project. The performance grades can also promote the exchange of information among practitioners, including signal timing experts and trainees who can quickly share results and develop their expertise.

3) The proposed performance measurement can be conducted in a scalable and multipurpose fashion. It is feasible in the contexts of multiple signal control modes (e.g., pre-timed, actuated, or adaptive signal control), various objectives (e.g., improving progression along arterial through movements, improving transit signal priority, or improving mobility along a specific route of interest), and various scopes (e.g., ad-hoc performance studies or daily monitoring). The proposed performance measurement is based on vehicle travel-run trajectories, which are constituted by the Global Positioning System (GPS) data. Such trajectory data can be gathered through probe vehicle investigations, acquired from third parties (e.g., INRIX and the National Performance Management Research Data Set (NPMRDS)), and obtained from the emerging connected-vehicle applications [11]. Using trajectory data possesses excellent flexibility in budgeting and resourcing. The agency can still perform performance measurements under tight-budget conditions by assigning a few technicians to do travel runs along arterials for generating GPS data, while the agency can also obtain abundant trajectory data from third-party data providers or by deploying connected-vehicle technologies if the budget allows. In addition, based on the trajectory generated by transit buses or bicycles, the proposed methodology can be used to evaluate the quality of signal timing with respect to the transit and bicycle traffic moving along arterials.

With the proposed performance measurement methodology, agencies across the nation may be more capable and willing to integrate performance measurements with daily practices and decision-making processes, which ultimately improve traffic signal operations. Both data-driven and problem-driven signal timing procedures can be enhanced, as exhibited in Figure 1.1, where effective, active (or even proactive), and multi-stakeholder involved signal timing management can be achieved. Therefore, the previously mentioned "vicious circle" would be terminated and then turn benign toward a continuously improved quality of signal timing at economical costs.

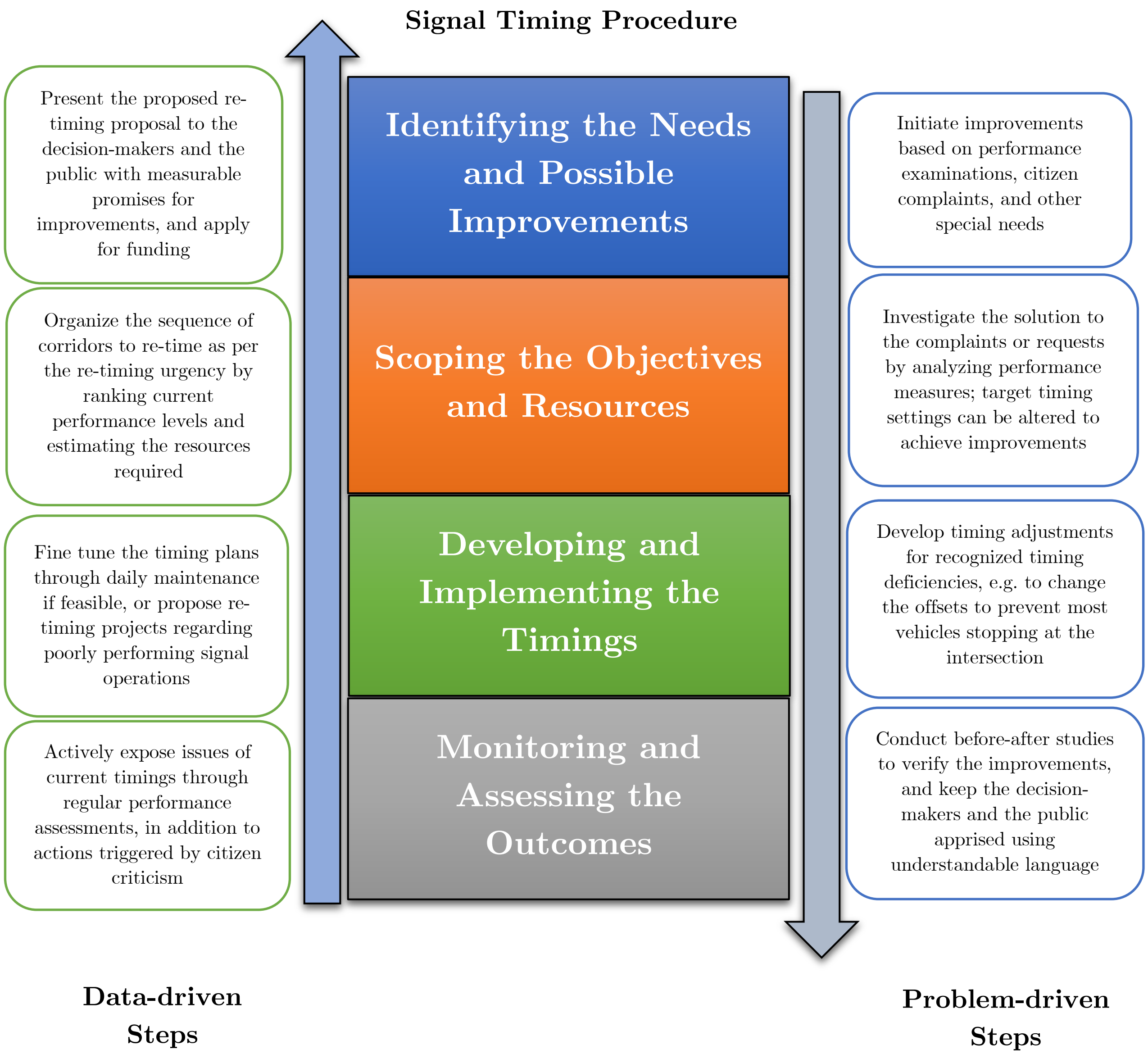


Figure 1.1. Integrating Performance Measurements with Signal Timing Practices

1.5 Organization of the Dissertation

This dissertation is organized into six chapters that document the contents pertaining to the development of two performance measures, the determinations of the quality of arterial signal timing, and the method implementation.

The research motivations, objectives, and potential contributions are presented in Chapter 1.

A literature review is documented in Chapter 2, which covers the current research and practices related to signal timing performance measurements. The existing signal timing performance measures and two major performance measurement techniques are analyzed.

Chapter 3 presents the development of the proposed methodology for measuring the quality of arterial signal timing. Two performance measures named Attainability of Ideal Progression (AIP) and Attainability of User Satisfaction (AUS) are newly developed. Based on AIP and AUS, a grade-based performance evaluation framework is established.

Chapter 4 outlines the method implementation, including data gathering, data processing, and the examination of applicability prior to the implementation.

Chapter 5 describes the case studies which demonstrate the effectiveness of the proposed method through real-world implementations.

Chapter 6 summarizes the research findings and potential future research to extend and improve the proposed method.

Chapter 2 LITERATURE REVIEW

2.1 Overview

This chapter provides a literature review of the current studies, techniques, and applications with regards to signal timing performance measurements.

For decades, the federal government has expressed a keen interest in performance measurements, and the local agencies have been required to strategically plan how they will deliver high-quality services as well as to measure their programs' performance in meeting these commitments specifically. A key feature of MAP-21 (Moving Ahead for Progress in the 21st Century Transportation Bill) is the establishment of performance- and outcome-based programs, placing much emphasis on promoting the concept of performance measurements.

A succinct definition of performance measurements can be expressed as [12]: assessing progress toward expected program achievements, including information on the efficiency with which resources are transformed into goods and services (outputs), the quality of those outputs (how well they are delivered to users and the extent to which users are satisfied) or outcomes (the results of a program activity compared to its intended purpose), and the effectiveness of operations with regard to their specific contributions to program objectives. As stated in an NCHRP report also quoted by the FHWA Operations Performance Management Program [13], performance measurement is the use of statistical evidence to determine progress toward specific organizational objectives. This includes the evidence of both measurements of the quality of outcomes and measurement of user perception, such as would be accomplished through a user satisfaction survey.

As for transportation, regarded as a service industry, to establish a set of performance measures is essential, gauging the level of transportation services regarding their strategic and operational goals. The performance measures should quantitatively reflect where the transportation services have performed in comparison with the optimal levels. The satisfaction of users who are ultimately served by transportation systems needs to be included in performance measures, in addition to those concerns of transportation system owners or operators whose definition of the best quality is often deemed authoritative. As exhibited in Figure 2.1, effective performance measures should consist of two aspects: 1) measuring how well the service is delivered and 2) measuring how well the users feel about the service.

Figure 2.1 Two Major Aspects of Performance Measures

Properly selecting performance metrics is essential for signal timing performance evaluations. Hence, an analysis of the metrics used in the current signal timing performance measurements is presented in this chapter firstly.

The current practices and techniques for measuring signal timing performance can be classified into two categories: 1) the performance measurements based on high-resolution controller event data and 2) those based on travel-run data. A comprehensive review is presented in this chapter, additionally regarding these two types of performance measurement techniques.

2.2 Performance Measures Adopted in Current Practices

Nearly one and half centuries ago, traffic signals were first invented in order to organize the traffic operations at intersections. Today, traffic signals have become ubiquitous and much evolved. There are many signal control modes, such as pre-timed, semi-/fully- actuated, responsive, and adaptive control. There are also several types of traffic signal controllers and detecting sensors that have been invented. Technological advances can enhance the performance of traffic signal operations; however, studies indicate that sophisticated traffic signal control systems do not always deliver the expected outcomes in practice [14]. There is a misconception that some practitioners conduct performance measurements based on the number of resources invested in upgrading the facilities, assuming the more cutting-edge control systems installed, the better performance these systems can result in.

Accordingly, to implement performance measurements of signal timing still requires a focus on how well the traffic is served under the control of traffic signals, rather than by an input-oriented approach which only counts the number of investments.

Even though signal control techniques have significantly evolved and become diverse nowadays, the ultimate objective of traffic signal control is still based on one theme – assigning right-of-way within the context of safe operation also with as-little-as-possible delay generated. To specify the objectives of traffic signal operations, there exist several dimensions toward all of the traffic participants at the intersection, e.g., passenger vehicles, freight trucks, pedestrians, bicyclists, and transit buses. The *Signal Timing Manual* [15] documents several specific operational objectives that can be used individually or in combination to focus signal timing efforts, which are listed in Table 2.1.

Table 2.1 Multi-dimensional Objectives of Signal Timing Practices [15]

Objective Dimensions	Definitions	
Safety	Reduce vehicle-, pedestrian-, and bicycle-related conflicts. Provide sufficient time for all traffic participants to execute movements. Ensure those signal indications would not be distractive or confusing to drivers, pedestrians, and cyclists.	
Mobility	Capacity Allocation	Serve vehicle, pedestrian, and bicycle movements as efficiently as possible while also distribute capacity as equitably as possible across movements and modes. Prioritize some movements according to need (e.g., transit priority) without excessively delaying other movements.
	Corridor Progression	Minimize delays on high-priority movements (typically the through movements along the arterial) for vehicles, and if possible, for transits or bicycles.
	Delay Control	Control delays on the secondary movements (the turning movements along the arterial or the cross-street movements) for vehicles, and if possible, for transits or bicycles.
Environmental Impact Mitigation	Minimize the amount of induced pollution by improving the efficiency of vehicle trajectories, e.g., by reducing vehicular delays or stops. Promote high-occupancy traffic modes (e.g., transit priority).	
Queue Length Management	Prevent the formation of excessive queues on critical lane groups, such as freeway exit ramps. Avoid queue spillovers, e.g., eliminating the stops as much as possible between closely spaced intersections such as interchange signals.	
Operating Cost	Minimize stops and delays in order to reduce vehicle operating costs and to save time costs for drivers, pedestrians, cyclists, and transit passengers.	
Accessibility	Provide the ability for pedestrians and transit vehicles, including special-needs groups, to execute movements. Improve the ease of reaching the destinations and activities according to need (e.g., by reducing delays and stops along major commuting routes)	

To achieve the highest level of every objective is impractical. There are a few inherently incompatible objectives, e.g., most means to improve the quality of arterial progression (progression is the movement of users along a designated route in a manner that minimizes stops [15]) would inevitably increase the total delay on the non-progressed movements. Additionally, some objectives may mutually overlap under certain circumstances. For instance, the quality of progression not only influences the number of stops at the intersection but also closely correlates with environmental and economic impacts. Poor quality of progression results in additional stops, leading to the more idling time of vehicles at intersections, which will generate more emissions and increase fuel consumption than if vehicle platoons can smoothly traverse the intersections [16]. Hence, in real-world practices, it is common to see that only some critical objectives are selected and then measured in performance evaluations, in an effort to achieve the significant expected outcomes (e.g., for a signal re-timing project that aims to develop transit signal priority, minimizing the delay time and the number of stops for transit vehicles is the core intention, and the performance measurements will focus on evaluating the improvements regarding transit traffic). The other objectives can be either included in the critical objectives or regarded as prerequisites for successful operations, which means that any signal timing improvements according to the critical objectives can be deemed meaningful only if a good extent of the other objectives is fulfilled. For example, the travel time along the arterial can be reduced at the expense of the side-street traffic and pedestrians; however, the reduction will be unvalued if a significant delay time increase is imposed on the side-street traffic and pedestrians, especially when the people who frequently use the minor-street routes are aware of the increased delays and then complain to the agency.

When it comes to arterial operations, the critical subject of signal timing is to improve platoon progression between signals, which is achieved by signal coordination, the most iconic part of arterial-level signal operations. The remainder of this dissertation will focus on the performance measurements for signal coordination. The concepts, methodologies, and techniques for developing signal coordination were documented in the *Signal Timing Manual* [15], which will not be discussed further here.

There have been a number of metrics adopted in the current practices pertaining to signal coordination evaluations, which are summarized in Table 2.2.

Table 2.2 Performance Metrics Used in Current Practices

Metrics	Criteria for Good Operations	Data Source	Advantages	Disadvantages
Progression Bandwidth	The greater Progression Efficiency, Progression Attainability [17], or Progression Opportunity [18], the better quality of signal coordination it performs.	Time-Space Diagram	The quality of timing plans can be estimated when the timing is initially designed through visual analysis.	Progression bandwidth only displays the length of the progressive time window, which cannot pledge good platoon progression formed in reality.

Table 2.2 Performance Metrics Used in Current Practices (continued)

Percent Arrivals on Green	Percent arrivals on green is the proportion of vehicles that arrive during a green indication relative to those that arrive during a red-light indication. A high percentage demonstrates the signal coordination is effective.	Detection and controller events	It indicates how many vehicles actually benefit from signal coordination.	A specific set of detector installation and layout is required. The data collection can be significantly affected by queuing.
The ratio of Arrival on Green to Arrival on Red	The ratio of intersections that the vehicle arrives at on green to that the vehicle arrives at on red traversing along the arterial. Achieving less-than-two stops per five signals is Usually considered a good operation.	Floating-car studies	It provides an intuitive and corridor-level analysis of progression quality, which can be obtained through observations.	It is sensitive to the size of samples as well as the number of evaluated signals.
Platoon Ratio	It is calculated as percent arrival on green divided by the green-to-cycle ratio [19]. Platoon Ratio ranges between 0.3 and 2.0. A platoon ratio of 0.3 represents Arrival Type 1 [20], which can be caused by the inferior quality of progression. A platoon ratio of 2.0 indicates an exceptional quality of progression.	Detection and controller events	It averts inappropriate timing designs, which are mostly in favor of arterial traffic. Platoon Ratio of 1.0 is a handy baseline by which to judge whether signal coordination is beneficial or not	Platoon Ratio is a link-based metric and requires a specified detection configuration. It is also sensitive to queuing.

Table 2.2 Performance Metrics Used in Current Practices (continued)

Travel Time /Travel Speed	Good quality of progression can be demonstrated by the reduced travel time or the increased travel speed.	Travel-run trajectories	Travel time or average speed can be an intuitive performance measure to the public, operators, planners, and maintenance staff.	It can be influenced by non-signal-timing factors such as congestion levels along arterials. It does not reveal travelers' perceptions.
Number of Stops per Mile	The fewer stops per mile, the smoother platoon operation achieved, which indicates a better quality of signal coordination.	Travel-run trajectories	It closely relates to fuel consumption, polluting emissions, and the underlying feelings of travelers	It is sensitive to signal density, and the number of stops is not differentiated regarding stop duration
Vehicle Delay	Good quality of signal coordination typically can reduce the average delay of the vehicles in the system. Some measures were developed based on vehicle delay, e.g., Performance Index [21] which is a combination of cumulative delay and the number of stops incurred on the trip (usually one stop equals to 20-second delay time)	Simulation studies or mathematic calculations	Average vehicle delay is a network-level metric, which covers traffic on the side streets	It is challenging to measure vehicle delay time in the real world.

As described in Table 2.2, the quality of progression can be mainly reflected by the link-level metrics (e.g., Percent Arrivals on Green/ Platoon Ratio), and by arterial-level metrics (e.g., the number of stops per mile/ travel time or travel speed along the arterial), which are computed by using controller event data and travel-run trajectories.

Both types of metrics are useful for evaluating the quality of signal timing and developing timing improvements; however, some vital information of signal operations may be missing if the only type of metrics is adopted. Figure 2.2 illustrates the Time-Space Diagrams of two signal timings for three intersections. Only the offset and the phase sequence at intersection 1 is different between the two timings, while two different types of progression are caused. For the link-based progression, the links between the neighboring intersections all possess very good progression bandwidths, but the vehicles driving through the three intersections would inevitably make one stop (the inbound direction: the arterial traffic is stopped at the intersection 1, and the outbound direction: the arterial traffic is stopped at the intersection 3). If the two timings are implemented, the resulting values of Percent Arrivals on Green could be similar between on the movements A and B as well as between on the movements C and D, which implies that the link-level metrics may understate the characteristics (including the information about the number of stops, stopped time, and queue length, which correlate with not only the effectiveness of signal coordination but also the driver satisfaction of trip quality) that can be captured by certain travel-run trajectories, e.g., the trajectories α and β shown in Figure 2.2.

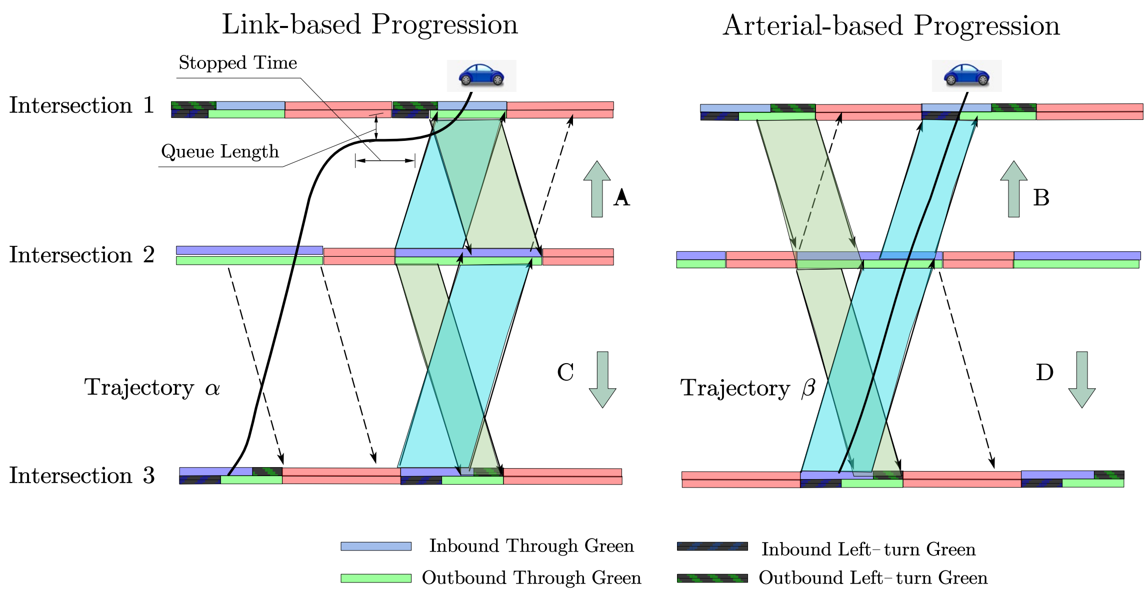


Figure 2.2 Comparison between Link-based and Arterial-based Progression

Compared to link-level performance metrics, the implementation of arterial-level metrics is limited due mostly to data availability. A vast amount of data collection along different routes of interest is required before a representative sampling can be obtained. Therefore, many studies and innovations have been conducted on the basis of link-level metrics [22]. In contrast, arterial-level metrics related research is currently scarce [23, 24, 25] to the best knowledge of the author. In recent years, more and more emerging technologies have been developed and implemented, which hold promise for ubiquitously collecting high-resolution travel-run trajectories throughout the roadway network. This promotes the use of trajectory data in signal timing performance measurements, also showing an urgent need for related research.

To fill the gap, this dissertation mainly focuses on exploring potential arterial-level performance metrics by leveraging trajectory data in order to enhance the current signal timing performance measurements.

2.3 Performance Measurements based on Controller Event Data

Nowadays, most traffic controller products have capabilities to describe and preserve the information of controller events (e.g., phase turning green, turning red, as well as detector calls received or lost) with timestamps that contain the time of the event with the time resolution up to 0.1 seconds. Therefore, the data logged by controllers can be used to reproduce the control state in a combination of signal control operations (which/when/how long phase(s) turn on green) and traffic flow operations (when/how many vehicles pass over the road detector), at any given moment during the time of signal operation. Some studies have been conducted [26, 27, 28] to create signal timing performance measures by leveraging these data sources, and upon which, several techniques have been developed such as Automated Traffic Signal Performance Measures (ATSPMs) [29] and SMART-SIGNAL [30].

ATSPMs is one of the most noted performance measurement systems for signal timing based on the high-resolution data-logging capability added to existing traffic signal infrastructure. It provides several data analysis techniques to conduct evaluations of communication, detection, timing, and coordination for traffic signals. Agency professionals can use the information given by ATSPMs to identify and correct deficiencies in signal operations.

The U.S. Department of Transportation and the Federal Highway Administration (FHWA) are currently promoting ATSPMs as a means to improve on the traditional retiming processes by providing continuous performance monitoring capability [31]. According to the information published by the FHWA, approximately 26 transportation agencies at both state and local levels are involved in implementing ATSPMs. Recently, an open-source software package was developed in a study sponsored by the Transportation Pooled Fund Program, "Traffic Signal Systems Operations and Management" [32], which publishes a framework for continued innovation in data analysis techniques.

The ATSPMs system contains several measures, such as 1) Purdue Phase Termination, 2) Split Monitor, 3) Pedestrian Delay, 4) Preemption Details, 5) Purdue Coordination Diagram (PCD), 6) Approach Volume, 7) Approach Delay, 8) Arrivals on Red, 9) Approach Speed, and 10) Purdue Split Failure [33]. Data visualization is also enabled upon several diagram themes [7]. These automatically collected and generated measures depict the status of the signal operation in real-time and help practitioners monitor whether the agency's objectives have been achieved.

Among the diagrams of ATSPMs, there is one specifically addressing the issues of arterial signal timing, called the "Purdue Coordination Diagram (PCD)." The PCD is a useful tool that offers a quick visualization of how well a signal system is coordinated. The effectiveness of signal coordination on certain movements is demonstrated through quantified and graphical indicators. As shown in Figure 2.3, detection events (black dots: each of the dots indicates the moment when a vehicle triggers the road detector, implying a vehicular arrival at the time) and signal phase status (red/green dots shown as two lines: each of the dots represents the moment when the signal light turns red or green, implying the duration times of the green-light and red-light intervals) are depicted in a PCD chart. Through this combination, both visual and quantitative figures of the proportion of vehicles that arrive at the intersection during red-light and green-light time intervals are provided.

The PCD chart plots the time of day on the horizontal axis and the time in the cycle along the vertical axis. The vertical strips represent single cycles divided by BOG (begin of green, shown as green dots on the diagram) and EOG (end of green, shown as red dots on the diagram). The region between the line of BOG and EOG dots delineates the time window when vehicles can move through the intersection without stopping. The more vehicles showing up within this time window, i.e., the higher portion of black dots located in the region, the better quality of coordination typically attained. The EOG line also portrays the fluctuation of cycle times as the cycle length would be varying during non-coordinated time periods and coordinated time periods whereas transitions happen.

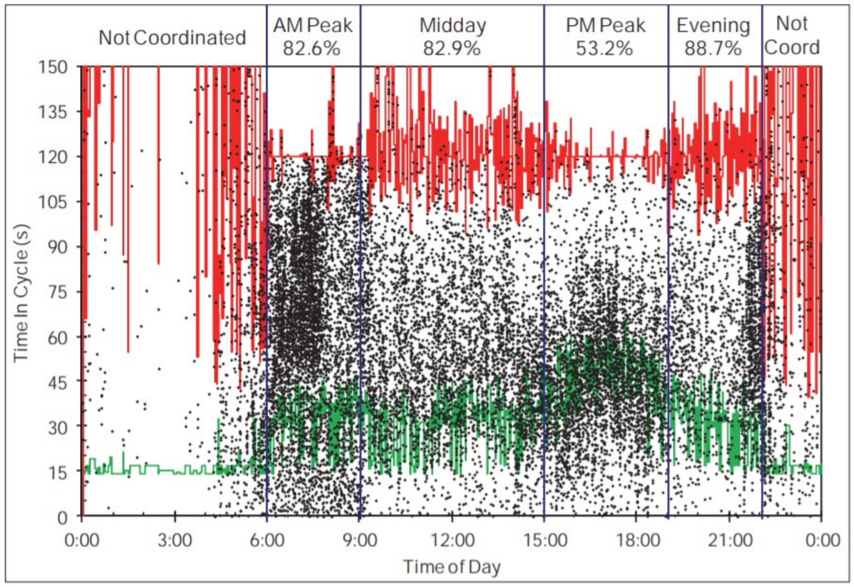


Figure 2.3 Purdue Coordination Diagram（Source: Signal Timing Manual [15] ）

The practitioner can intuitively assess the level of performance through a comparison between the dot densities above and below the green line. In addition, some numerical measures are provided based on the distribution of the dots. For example, the percentage of vehicles arriving amid the green indication (the arrivals on green) for each coordination plan is shown above the PCD chart in Figure 2.3, such as "AM Peak 82.6%" that indicates 82.6% of the vehicles can traverse the intersection making no stops for the AM-peak coordination plan.

Furthermore, the percentage of arrivals on green divided by the green-to-cycle ratio yields a new metric called the "Platoon Ratio," which accounts for the fact that the longer green time of the cycle assigned to a phase, the more likely it is for vehicles on that phase to arrive on green while the less green time received by the vehicles on other phases to pass the intersection. The Platoon Ratio, therefore, tends to reward the signal coordination that is performed with shorter greens on coordinated phases, and to scale down the results if unnecessarily long green times are allocated to the coordinated phases despite a high percentage of vehicles arriving on green achieved.

A Platoon Ratio of 1.0 is a threshold which indicates that the effectiveness of signal coordination is insignificant (the Platoon Ration equals to 1.0 anyway if vehicles arrive randomly). Numbers higher-than-one represent that traffic signals are favorably coordinated, and numbers lower-than-one indicate that there could be detrimental signal timing settings that lead to unfavorable arterial operations.

With the capabilities of automated data collection, the ATSPMs system allows 24-7 surveillance aiding in both supervisory and perceptive capabilities of the agencies, which are being tested or deployed in many states across the U.S. [34, 35, 36]. However, there are still some issues with ASTPMs that need to be addressed.

First, the implementation of ASTPMs requires a certain infrastructural configuration [37], which makes the system less appealing to those agencies with limited funding resources. The ASTPMs-enabled traffic signal controllers must be equipped with high-resolution data loggers. Most controller manufacturers such as Econolite, Peek, Siemens, Intelight, and Cubic (Trafficware or Naztec) have integrated controller data loggers with their up-to-date products; however, the agencies may need to spend additional funds on replacing or upgrading their existing controllers. The ASTPMs systems also rely on specified detection setups.

For example, to produce the PCD diagrams, advance detectors (or called setback detectors) are supposed to be deployed and well-maintained at the intersections, whereas current practices on detector configuration vary considerably from one agency to another [38]. Hence many agencies may need to convert their detection configurations. Besides, operating and maintaining the web service and database of ATSPMs require additional technical personnel.

Another limitation of ATSPMs lies in queuing. Generating PCDs requires the data collected by the setback detectors. But, as illustrated in Figure 2.4, when the queue spills and reaches the position of the setback detector, a false platoon may be created on the PCD chart [39], and the resulting percent arrivals on green and platoon ratio can be incorrect.

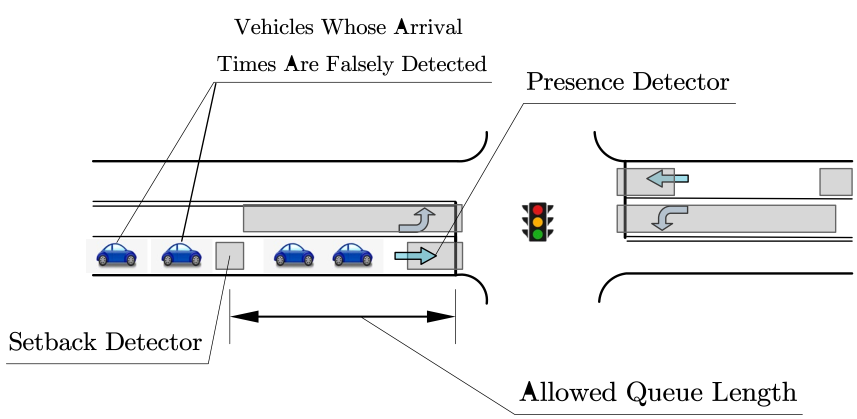


Figure 2.4 Queuing Issue of ATSPMs

More critically, the ASTPMs-based performance evaluations are mostly based on link-level metrics focusing on independent traffic movements, not arterial-performance-oriented. Consequently, some characteristics which indicate the quality of arterial signal timing (e.g., travel speed and stops driving along the arterial) cannot be well described on the basis of ASTPMs [40].

The use of ATSPMs is complex and obscure in daily practice sometimes. As for the PCD charts, one of the comments from the current users of ATSPMs was noted – "PCD charts show too many dots" [41]. Notwithstanding, the PCD is considered an effective tool for performance measurements; however, practitioners often encounter challenges when attempting to quantify the signal timing performances based on the metrics provided by PCD. In order to establish a defensible standard of "good quality of timing for arterial operations", the researchers have been trying to aggregate the data provided by ATSPMs into letter-grade assessments. The overall grades of the quality of signal operation are categorized as "A," "B," "C," "D," and "F" in a format similar to the Level of Service (LOS) framework documented in the *Highway Capacity Manual* [42]. The practitioners can monitor and manage signal operations based on the grades, e.g., to recognize re-timing needs through grade ranking.

In terms of arterial operations, one additional consideration, volume-to-capacity ratio, was integrated with Platoon Ratio in the grading methodology. The thresholds of the grades A, B, C, D, and F may change according to the congestion level at the intersection. For instance, when the volume-to-capacity ratio nears zero, an A-grade performance of signal coordination is attained if the Platoon Ratio is greater than 1.3; however, any Platoon Ratios greater than 1.1 can result in grade-

A quality when the volume-to-capacity ratio increases to 0.9, as illustrated in Figure 2.5.

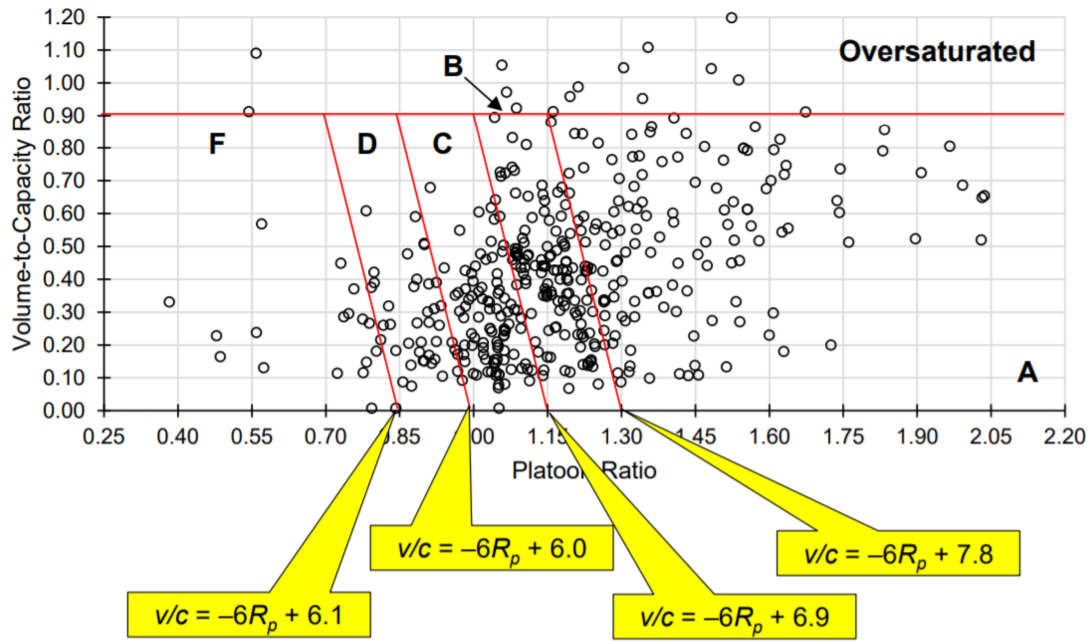


Figure 2.5 Grading Methodology of Progression Quality Used by ATSPMs [43]

Many of the studies related to ASTPMs are still ongoing [44], and more information about how ASTPMs can consolidate signal timing management will be presented in the deliverables of the NCHRP Project 3-122: Performance-based Signal Management of Traffic Signals. The final reports are expected to be released by the year 2021.

SMART-SIGNAL is another technique for gauging signal timing performances based on high-resolution controller event data. Different from ATSPMs, SMART-SIGNAL evaluates arterial signal timing performances based on travel time, and the travel time is estimated through the generated virtual probe vehicle trajectories [45, 46], as shown in Figure 2.6. In other words, SMART-SIGAL uses controller event data to conduct signal timing performance measurements, but its performance measurement method depends on trajectories.

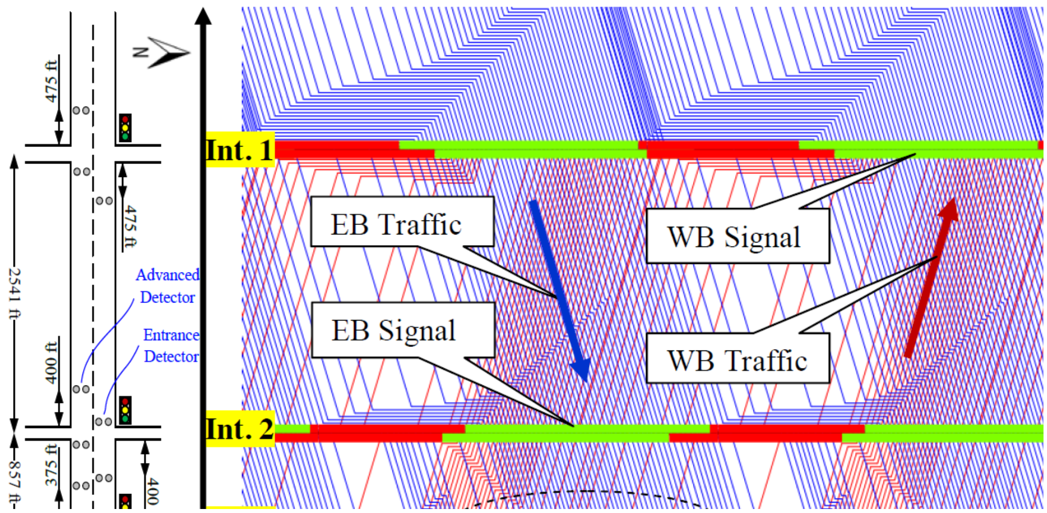


Figure 2.6 Virtual Trajectories Generated by SMART-SIGNAL [45]

2.4 Performance Measurements based on Travel-run Data

Besides high-resolution controller event data, travel-run data like GPS trajectories are attracting more and more interest among researchers and engineers recently regarding the usage in signal timing [47, 48, 49]. High-resolution GPS trajectories can be applied to signal timing performance measurements with great promise due to being descriptive of detailed and consecutive vehicular motions, which indicate where and when the vehicles proceed or stop.

Some attempts have been made to develop signal timing performance measures using trajectory data. The Orange County Transportation Authority

(OCTA), in collaboration with the California Department of Transportation (Caltrans) and the local agencies within the county, initiated the Signal Master Plan for the countywide synchronization endeavor in 2009. The Signal Master Plan has defined a new parameter to gauge the performance of signalized arterials, which is called the Corridor Synchronization Performance Index (CSPI) [50, 51].

CSPI is used in a score-based methodology which evaluates the performance of signal timing for arterial operations based on 1) average speed, with the highest possible score of 36; 2) the ratio of the number of greens versus reds through signalized intersections, with the highest possible score of 40; and 3) the average number of stops per mile, with the highest possible score of 33, as exhibited in Figure 2.7. By combing the three scores, the overall CSPI can be computed, ranging from 0 to 100.

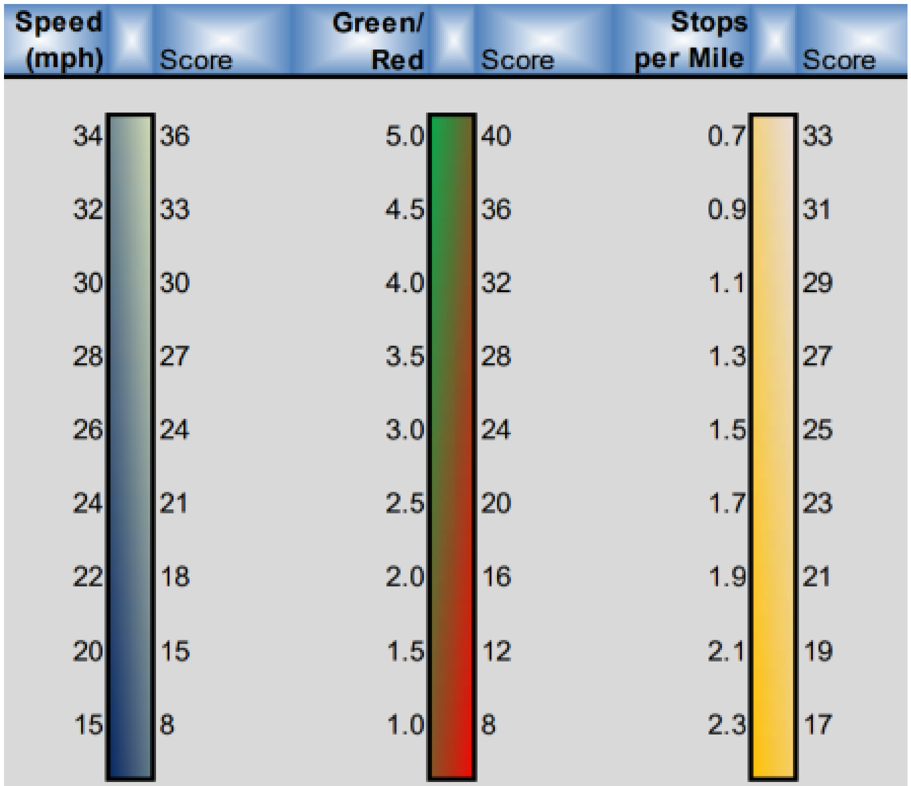


Figure 2.7 CSPI Scoring Methodology based on Three Measures [51]

In accordance with the CSPI scores, evaluation criteria were developed, which categorize the quality of signal timing into five tiers – Tier 1, 2, 3, 4, and 5, as shown in Figure 2.8. Tier 1 indexed by CSPI scores at or above 80 refers to very good signal coordination qualities. Tier 5 corresponds to CSPI scores lower than 50, indicating that the corridor would greatly benefit from signal timing improvements and suggesting a signal re-timing project.

CSPI Score	Signal Synchronization Description	Level
>=80	Very good progression – traveling through signalized intersections with minimal stops and favorable travel speeds.	Tier 1
70-80	Good progression – traveling through signalized intersections with few stops and good travel speeds.	Tier 2
60-70	Fair progression – traveling through signalized intersections with moderate stops and fair travel speeds.	Tier 3
50-60	Limited progression* – traveling through signalized intersections with moderately high stops and slower travel speeds.	Tier 4
< 50	Very limited progression* – traveling through signalized intersections with frequent stops and slow travel speeds.	Tier 5

Figure 2.8 CSPI Corridor Synchronization Performance Criteria [51]

The advantages of CSPI are obvious. CSPI was initiated by expert practitioners who have a good understanding of the needs of local communities. The methodology of CSPI is concise and clear, which helps people who have limited traffic engineering knowledge to be convinced in an intuitive sense. In addition, CSPI does not require additional infrastructure and labor investments apart from the GPS devices and the resource for conducting the probe vehicle investigation.

It should be noted that there are some limitations for not only using CSPI but also adopting the most established trajectory-based performance measures [23, 49]. Firstly, the arbitrarily prescribed parameters may affect the accuracy of performance measurements. For instance, the highest Speed Score of CSPI is attained when the average speed is greater than 34 mph because most arterials in Orange County have speed limits that are 40 mph. As a result, the CSPI methodology may give false results when evaluating signal timing performances in the context of speed limit under 40 mph. And CSPI does not consider the factors that can influence the evaluation result which are not related to signal timing, such as arterial congestion level. Accordingly, the methodology is flawed as the signal timing performance would never be graded as Tier 1 due to oversaturated traffic demands even if the timing design is already the best. Last but not least, CSPI does not account for the travelers' perceived quality of signal timing, which should be a major consideration in signal timing performance measurements.

2.5 Summary

The current research and practices related to signal timing performance measurements were reviewed and analyzed in this chapter. A comprehensive analysis of signal timing performance metrics was conducted, which demonstrated the research need for developing signal performance measurement methods based on arterial-level metrics as the detailed shapes of travel-run trajectories allow the nuanced evaluations that may uncover the signal timing issues omitted through link-level performance measurements. Therefore, the proposed performance measurement methodology is based on arterial-level metrics using travel-run trajectory data.

In addition, the emerging signal performance measurement techniques were reviewed, in which ATSPMs based on high-resolution controller event data and CSPI based on GPS trajectory data were mainly analyzed.

According to the review findings, the performance measurement method proposed in this dissertation seeks to address the existing shortcomings that existed in the current practice in the following aspects:

1) The proposed performance evaluation framework is in an HCM-like format, which categories the quality of signal timing into five levels, namely A, B, C, D, and F. This can help practitioners to capture the quality of signal timing immediately and intuitively;
2) Compared to ATSPMs, the proposed performance measurement methodology focuses on arterial-level metrics extracted from vehicle trajectory data. The trajectory data can be obtained through a few accessible data sources such as sparse probe vehicle investigations, federal or third-party databases, and connected-vehicle applications. Hence, the proposed performance measurement methodology is applicable regardless of what kind of signal control mode, detection configuration, and controller facility may be used at the evaluated intersection.
3) Compared to CSPI, the proposed method adopts a similar framework but possesses two significant refinements. The non-signal-timing factors (e.g., arterial congestion level) are excluded from the evaluation methodology so that the performance measurement results can accurately reflect the quality of signal timing under various scenarios like unsaturated or congested traffic operations. And, the travelers' perception of traffic signal timing is considered in the evaluation methodology, which mainly focuses on whether the quality of signal timing can satisfy the travelers by meeting their need

for efficiently and safely traveling along the arterial. Hence, CSPI is enhanced by the proposed performance measurement methodology.

Chapter 3 METHODOLOGY DEVELOPMENT

3.1 Overview

This chapter presents the development of the proposed performance measurement methodology.

Two performance metrics are defined at first, which can effectively scale the quality of signal timing service as well as the extent of users' satisfaction. The calculations of the two metrics are based on the vehicle speed and stop information extracted from travel-run GPS trajectories. The two metrics have respective emphases, as speed characteristics mainly show the effectiveness of platoon progression while the stop characteristics can imply the users' perceived quality of signal timing. In addition, stop characteristics can also be related to fuel consumption and vehicular emissions, which correlate with environmental performance.

The performance measurement methodology proposed in this dissertation is independent of the selection of various signal control modes or facilities, which means the methodology should be suitable to use in most cases, given the variation in controller and detection facilities among the different jurisdictions.

3.2 Performance Measure Based on Speed Characteristics

Average speed, which refers to the average arterial operating speed or the average travel speed, has been used as a performance measure for a long period of time [52].

The average travel speed of the through-movement vehicles has been adopted by the latest version of the *Highway Capacity Manual* [42] to generate the automobile LOS (level of service) for urban street facilities (two or more segments including signals and roadway links between signals). Average speed conclusively indicates the degree of mobility achieved by the arterial operation regarding delay incurred due to signal control and other influential factors.

Average speed correlates with the quality of arterial signal timing as the time delayed by signal operations could be a significant part of the travel time along the arterial. Effective arterial signal timing can reduce the chance of vehicle platoons being stopped at the intersections, resulting in reduced average travel time by increasing average speed.

As mentioned previously, HCM automobile LOS can indicate the vehicular mobility along the arterial; however, this methodology cannot be directly adopted for signal timing performance measurement purposes because it involves many non-signal timing factors, which refer to the factors can influence arterial travel speed but are not related to signal timing. The three major non-signal-timing factors are listed below:

1) Level of Congestion along Arterials

   As the level of congestion increases, vehicles no longer can move at or near the free-flow speed in the optimal-progression context. Longer queuing distances, as well as slower queue dispersion, can be observed. The level of arterial congestion is usually gauged by the arterial volume-to-capacity ratio.

2) Arrival Flow Profile

Arterial signal timing creates a “window” of green as traffic moves along the arterial. This has beneficial effects when the “window” coincides with the time interval when most vehicles arrive at the intersections. Traffic arrivals consist of the vehicles from upstream arterial through movements and the vehicles left-turning or right-turning onto the arterial from the upstream cross-streets. The traffic arrivals of arterial-through origins and cross-street origins are distributed at different time intervals within a cycle. Typically, the progressed “window” of green is designed to accommodate the arterial through traffic, which constitutes the bulk of total arrival, but the vehicles turning onto the arterial from the upstream cross streets very likely arrive on red. Therefore, if the traffic from the upstream side street accounts for a considerable proportion of the traffic flow (e.g., the traffic volume differences among the time intervals become insignificant and arrival flow profile becomes uniform), the beneficial effects of coordinating signals will be limited and the average speed will decrease. This factor can be indicated by the proportion of the traffic volume of side-street origins to the total traffic volume along the arterial.

3) Signal Density

In practice, traffic flows are still likely interrupted by traffic signals even if signals are optimally coordinated. Hence, for a certain length of the corridor, the more signals involved in the analysis, the more likely it causes a stop somewhere that may reduce the average speed. Signal density is usually represented by the number of signals per mile.

Investigations were made during this research to characterize the effects of the abovementioned non-signal-timing factors on average speed. A hardware-in-the-loop simulation was utilized to generate a set of simulation scenarios. The simulation environment was established, as illustrated in Figure 3.1.

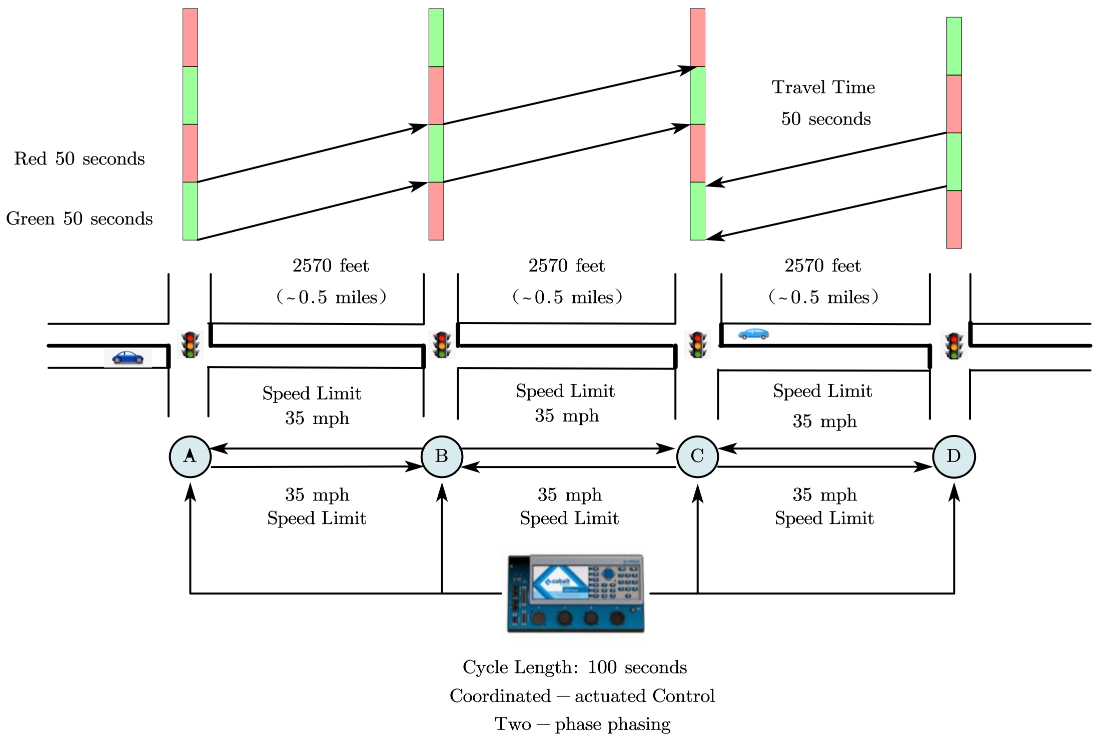


Figure 3.1 Hardware-in-the-loop Simulation Environment

An ideal arterial network was built with four equally spaced intersections in VISSIM, a microscopic simulation software package widely used across the world. The four intersections were controlled by four real traffic signal controllers (Econolite Cobalt ATC controller) connected with the VISSIM software. The hardware-in-the-loop system named PASS (exhibited in Figure 3.2) was used in order to authentically test arterial signal timings as it can perform realistic signal actuation and signal coordination functions.

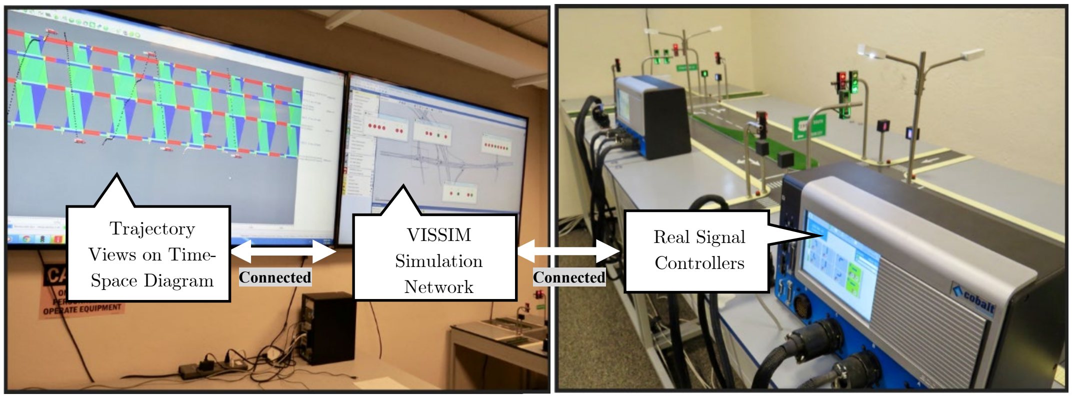


Figure 3.2 PASS: A Hardware-in-the-loop Simulation System

The phasing and timing scheme used in the simulation was simplified such that the cycle length (100 seconds) was divided into two phases, an arterial phase (50 seconds) and a side-street phase (50 seconds). With some certain values of offsets, it is possible to achieve the best two-way progression (e.g., offsets: 50-0-50-0) as displayed in Figure 3.1, or the worst two-way progression (e.g., offsets: 0-0-0-0). Other levels of the quality of progression can be achieved according to different sets of offsets.

A total of three investigations were completed regarding different scenarios of arterial congestion level, the traffic volume of side-street origins, and signal density. Six levels of progression quality were defined according to the six HCM arrival types [42], AT-1 (the worst), 2, 3, 4, 5, and 6 (the best). The ideal arterial operating speed, i.e., the free-flow speed, was 40 mph (the free-flow speed was estimated by the posted speed limit plus five mph in this research, while other advanced methods can be adopted with proper judgments [53]).

The following Sections 3.2.1 – 3.2.3 present the details of the simulation study.

3.2.1 Effect of the Level of Congestion along an Arterial

As noted previously, the level of arterial congestion can be measured by the arterial volume-to-capacity (V/C) ratio. In order to evaluate various levels of arterial congestion, 16 scenarios were created with different arterial volume-to-capacity (V/C) ratios across the four signals, as described in Table 3.1.

Table 3.1 Definition of Scenarios with Various Levels of Arterial Congestion

Scenario Numbers	$0 < V/C \leq 0.3$ (lightest congestion)	$0.3 < V/C \leq 0.55$	$0.55 < V/C \leq 0.85$	$0.85 < V/C < 1$ (Heaviest congestion)
1 Intersection Modified	1	2	3	4
2 Intersections Modified	5	6	7	8
3 Intersections Modified	9	10	11	12
4 Intersections Modified	13	14	15	16

The investigation began with an ideal operating condition that the lightest level of arterial congestion equally occurred at all the four intersections (the arterial V/C ratio was no greater than 0.3 where free-flow traffic was mostly observed). Then one intersection was selected, and its level of arterial congestion was modified from the lightest to the heaviest through four increments of the arterial V/C ratio. Holding the V/C ratios of the modified intersections unchanged at the heaviest level, the arterial congestion levels of the remaining three intersections were modified in the same manner one after another. At last, the heaviest arterial congestion appeared at all the four intersections after the 16^{th} -time modification. The arterial V/C ratios were manipulated through the increases of the arterial traffic demands at the intersections.

It should be noted that it was difficult to use the HCM arrival types as the index of progression quality in this investigation as the arrival types are designated to individual intersections, implying there could be 64 combinations of arrival types. Under certain congestion levels, some of the combinations of ATs were impossible to realize, e.g., AT-6 could almost never happen at four intersections simultaneously if the arterial congestion level was high. Therefore, an alternative index was adopted, which was the sum of the three one-way progression bandwidths between every two adjacent signals. Accordingly, six levels of the quality of progression were defined as "Type-1: the bandwidth sum ≥ 125 seconds", "Type-2: 125 seconds> the bandwidth sum ≥100 seconds," "Type-3: 100 seconds > the bandwidth sum ≥ 75 seconds," "Type-4: 75 seconds > the bandwidth sum ≥ 50 seconds," "Type-5: 50 seconds > the bandwidth sum ≥ 25 seconds," and "Type-6: the bandwidth sum < 25 seconds," mirroring the HCM arrival types.

Findings:

As shown in Figure 3.3, the following results were observed:

1) For the six levels of progression quality, when the arterial V/C ratio ranged between 0 and 0.55, increases of arterial V/C ratio only slightly changed average speed, whereas average speed was affected to an obvious extent if the arterial V/C ratio was greater than 0.55. The average speed significantly decreased as the arterial V/C ratio increased above 0.85.
2) The influential effect of arterial congestion receded along with the increases of modification times and eventually converged around 50% of the free-flow speed. The convergence point of "50% free-flow speed" might account for the circumstance in this study that the travel time between the neighboring intersections at the free-flow speed was 50 seconds, and the longest stopped time at the intersection was 50 seconds as well. When the intersections along the arterial were congested, most vehicles spent around 50% of the travel time on stops and slowdowns regardless of the quality of signal timing. In practice, the spacing between the coordinated signals typically ranges from a quarter-mile to a half-mile (the travel time often varies from 30 seconds to 60 seconds)[54], and the cycle length used generally ranges from 60 seconds to 120 seconds (the longest stop time of arterial through traffic can be regarded as a half time of the cycle, which ranges from 30 seconds to 60 seconds, in accord with the travel time). Therefore, this "50% free-flow speed" should be representative of the most convergence points observed in reality.
3) The sequential order of the selected intersections did not substantially affect the results, e.g., the resulting differences of average speed were negligible between "modifying the intersection A-B-C-D" and "modifying the

intersection D-C-B-A" (the positions of intersection are exhibited in Figure 3.1).

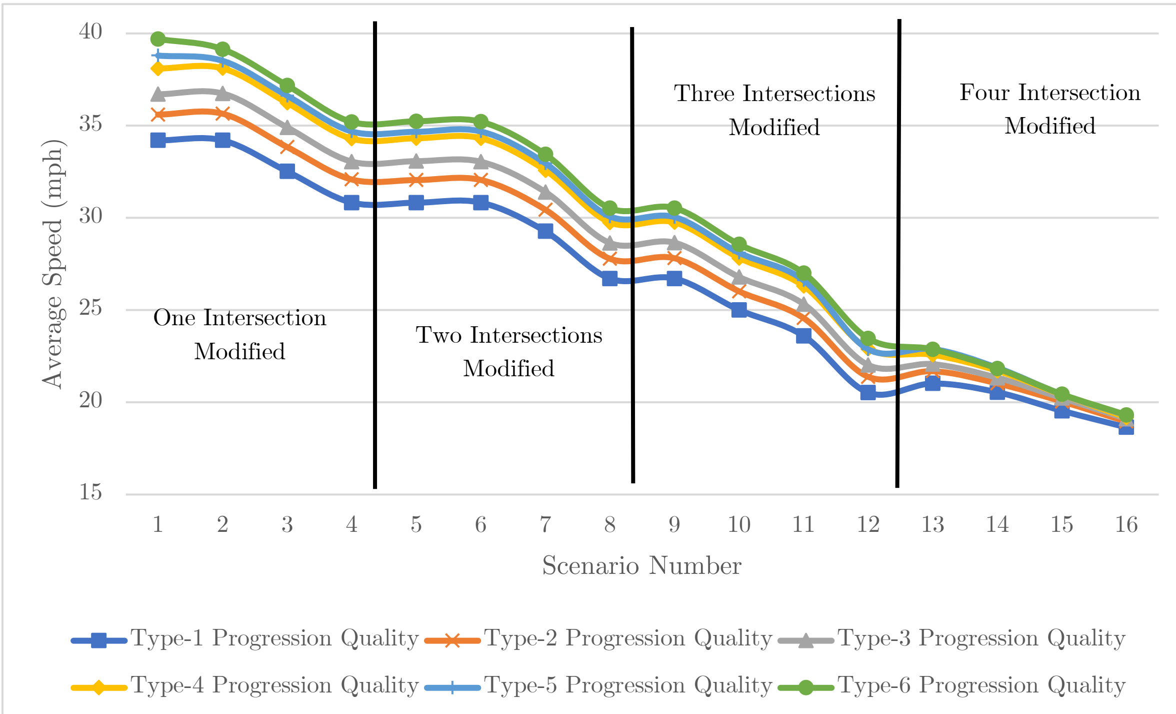


Figure 3.3 Change of Average Speed under Different Arterial Congestion Scenarios

This investigation sought to explore the effects of arterial congestion levels in the context of unsaturated operations. Under oversaturation conditions, queue spilling over can happen recurrently during which a measurable decrease of average speed can be observed, e.g., the observed average speed can be slower than 50% of the free-flow speed.

3.2.2 Effect of Traffic Volume of Side-street Origins

When timing traffic signals, practitioners need to monitor traffic arrival flow profiles to determine the offset and phase sequence. Arterial through movements are mostly progressed in practice; however, the traffic arrivals in other time intervals (e.g., the traffic of upstream side-street origins) may be stopped at the intersection. Therefore, if the proportion of the traffic of side-street origins to the overall traffic flow increases, the average speed may be reduced.

Experimental investigations were conducted in this research aiming to reveal the effect of the proportion of traffic volume of side-street origins to the overall traffic arrivals. Among the five experiments, the traffic volume of side-street origins made up five different proportions (0%, 12.5%, 25%, 37.5%, and 50%) of the total arrivals at the intersection, while the number of arrivals remained constant (arterial V/C Ratio = 0.4). The offset was adjusted in accordance with each change of the proportion to achieve the different quality of progression (to achieve Arrival Types 1-6 if possible). The simulation studies only considered the operation in one direction along the arterial and between two adjacent signals.

Findings:

As shown in Figure 3.4 and Table 3.2, some findings can be summarized as follows:

1) The amount of traffic volume of side-street origins could affect average speed, and could even cause some certain travel types to be unattainable, which

demonstrated that if the arrival flow profile became uniform, some qualities of progression would never be achieved no matter how well the signal timing could be improved.

2) The effect of the traffic volume of side-street origins on average speed could be complicated. If the offset design was inefficient (poor progression, AT-1, 2, or 3), average speed increased as traffic volume of side-street origins increased, while average speed decreased as traffic volume of side-street origins increased if the offset design was favorable (good progression, AT-4, 5 or 6).

Table 3.2 Resulting Average Speed Data under Various proportions of Traffic Volume of Side-street Origins and Progression Qualities

Average Speed (mph)	AT-1 Progression Quality	AT-2 Progression Quality	AT-3 Progression Quality	AT-4 Progression Quality	AT-5 Progression Quality	AT-6 Progression Quality
0% Volume of Side-street Origins	21.3	23.8	27.7	31.8	33.6	37.2
12.50% Volume of Side-street Origins	22.8	25.1	27.5	30.6	32.1	35.0
25% Volume of Side-street Origins	25.5	26.3	27.6	29.7	31.4	N/A
37.50% Volume of Side-street Origins	N/A	27.1	27.9	28.2	29.1	N/A
50% Volume of Side-street Origins	N/A	N/A	26.7	27.1	N/A	N/A

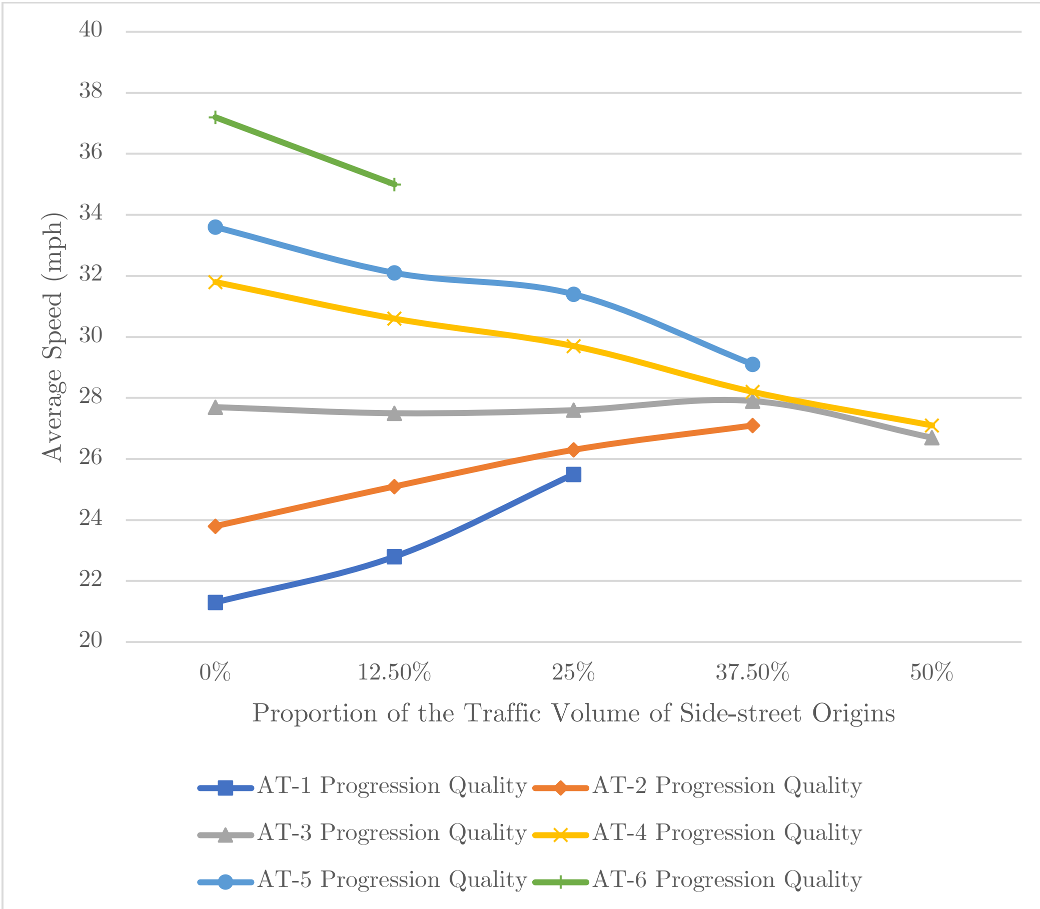


Figure 3.4 Change of Average Speed under Different Arrival Flow Profiles

3.2.3 Effect of Signal Density

Many researchers and practitioners believe that a high density of signals along the arterial can lead to the reduced average speed because the chance to make stops rises with the increased number of signals traversed. Moreover, if the signal density is low, the total time used for vehicles passing through intersection areas would just account for a minor part of the overall travel time, which means that the average speed is mostly determined by mid-block operations rather than the quality of arterial signal timing. As a result, understanding the effect of signal density on average speed is very important.

An investigation was conducted during this research based on three equally spaced intersections (the intersections A, B, and C displayed in Figure 3.1). By adjusting the separation distances, five scenarios with different signal densities were created. Holding the arterial congestion level constant (arterial V/C Ratio = 0.4) and the traffic arrival profile (traffic of the side-street origins accounted for 10% of the total arrivals) constant, the five scenarios were tested. The resulting average speed changes were gauged according to the operations in one direction along the arterial. "AT-1 progression quality" (shown in Figure 3.5) represents that AT-1 was achieved at the three evaluated intersections.

<u>Findings:</u>

As shown in Figure 3.5, some findings summarized as follows:

1) The number of signals per mile could significantly influence average speed if the offset design was inefficient (poor progression, AT-1, 2, or 3).
2) The number of signals per mile could slightly change average speed if the offset design was favorable (good progression, AT- 4, 5, or 6).
3) The impact of signal operations on average speed became less obvious if signal density was low. But, as the *MUTCD* [55] documented that traffic signals within 0.5 miles (2 signals per mile) of one another should be coordinated, there was rarely a coordinated signal system with a signal density lower than two signals per mile in practice. In this study, the signal

density of 1.33 signals per mile was analyzed, and the results showed that the quality of signal timing can still affect average speed under such low signal density.

4) The largest value of the number of signals per mile tested in this study was four signals per mile (the signal spacing was ¼ mile), representing the densest cases regarding general urban signalized arterials. Under such signal density, the quality of signal timing could significantly impact the average speed, calling for proper signal timing. In addition, in some downtown areas, streets may have denser signal spacings (e.g., less than 1,000 feet) where signals are difficult to coordinate, so an adjustment is needed when evaluating the quality of signal timing for high-signal-density arterials.

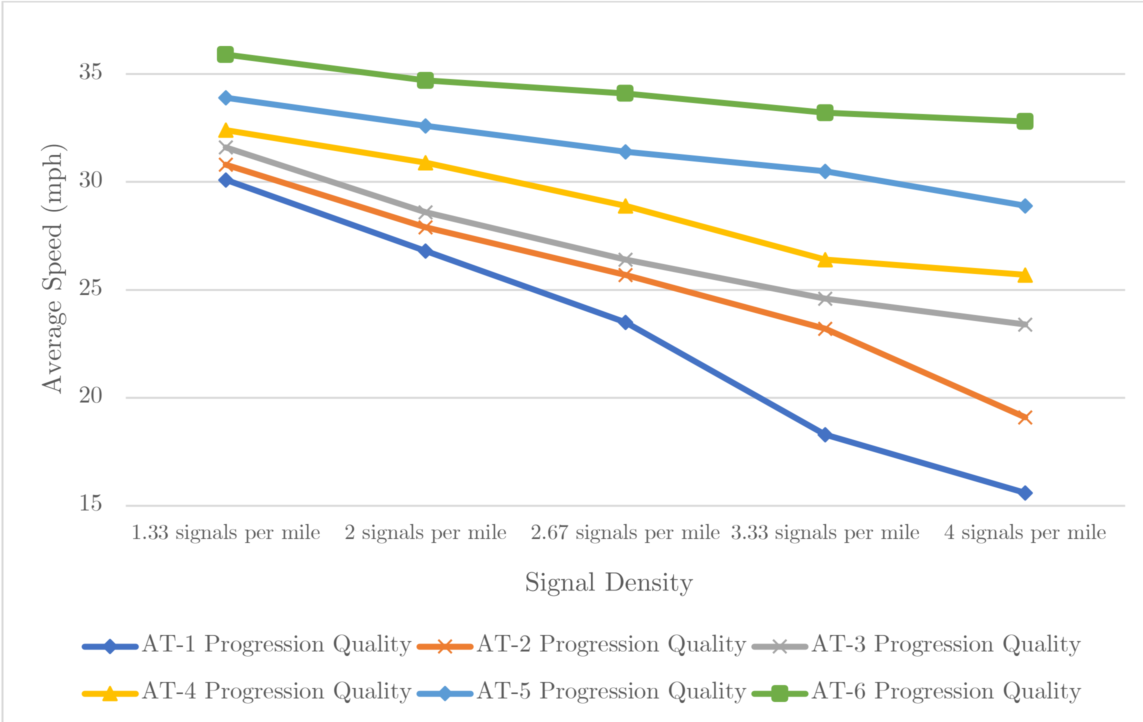


Figure 3.5 Change of Average Speed under Different Signal Densities

3.2.4 Attainability of Ideal Progression (AIP)

A new performance measure was defined in this research named the attainability of ideal progression (AIP), which is denoted by Equation 1.

$$AIP = \frac{Averge\ Speed}{Ideal\ Progressive\ Speed} \times 100\% \qquad (1)$$

where

AIP – the attainability of ideal progression; *Ideal Progression Speed* – the speed achieved under ideal progression along the arterial. The ideal progression speed can potentially equal to the free-flow speed if traffic operating conditions are optimal. According to different conditions of arterial congestion, cross-street traffic volume, and signal density, the ideal progression speed may change.

3.3 Performance Measure for Stop Characteristics

Empirical evidence has revealed that vehicle drivers are more aware of experiencing stops at the intersection [56] than they are of a reduction in travel speed. And many studies indicated that stops at the intersection could be one of the most important contributing factors to aggression and frustration of drivers, potentially causing red-light running or aggressive behaviors in driving [57, 58, 59, and 60]. Stops at intersections can also correlate with environmental performances such as fuel consumption and vehicular emissions [61]. Consequently, stop characteristics are of great importance for the evaluation of arterial signal timing performance because of the comfort, convenience, cost, and safety implications.

3.3.1 Effect of Stop Characteristics on Perceived Quality of Signal Timing

Studies have been conducted over the past 20 years, aiming to evaluate the level of service at signalized intersections based on drivers' perception [62, 63, and 64]. However, research devoted to arterial-level signal timing performance measures has been scarce. In practice, the metric of "number of stops per mile" is empirically used to scale travelers' perception of signal operations; however, there are many other characteristics of "stops" which are neglected. Solely considering the number of stops per mile may result in an incorrect estimate of the perceived quality of arterial signal timing, as many drivers may prefer two short stops of 15 seconds more than one long stop of 60 seconds whereas the evaluation based on only the number of stops indicates the opposite.

In order to further investigate the effect of stop characteristics on the perceived quality of arterial signal timing, questionnaire-based and interview-based traveler intercept surveys were conducted in Reno and Las Vegas, Nevada, during 2018-2020. A total of 67 valid sets of responses were obtained as of the time of writing. The surveys are still ongoing for gathering a larger sample size. Some socioeconomic attributes of samples are presented in Table 3.3. The survey questions are presented in the appendix.

Table 3.3 Sample Distributions Regarding Socioeconomic Attributes

Gender	Male: 62.7%	Female: 35.8%	Prefer Not to Say: 1.5%
Age	Below 25: 12%	Between 25 and 60: 82%	Over 60: 6.0%
Usage of Signalized Arterials	Frequent: 74.6%	Not frequent: 18%	I don't know:7.4%
Living Area	Urban: 62.7%	Suburban:10.6%	Rural:26.7%

Although the current sample size is limited, some preliminary findings have been acquired and summarized as follows:

1) 94% of respondents (63/67) agreed that the stop time could affect their impression of the quality of signal control service.

2) 66% of respondents (44/67) were more aware of the number of intersections that they can traverse without being stopped than the number of stops per mile.
3) A trade-off exists between the number of stops and stop time. Most respondents (27/67) considered stop times less than 10 seconds (among four options, “less than 5 seconds,” “less than 10 seconds,” “less than 15 seconds,” and “less than 20 seconds”) should be counted as minor stops. And many respondents (33/67) thought that stop times more than 20 seconds (among four options, “more than 15 seconds,” “more than 20 seconds,” “more than 35 seconds,” and “more than 40 seconds”) should be counted as full stops. Typically, 20-second stop time was counted as one stop [21];
4) The trade-off between the number of stops and stop time might vary at different intersections. Most answers from the respondents (58/67) revealed that a wait could be more tolerable when stopped at the major intersections than at the minor intersections;
5) Over half of the respondents (46/67) would like to stop twice at different intersections rather than experiencing a long wait at one intersection. A similar conclusion was drawn by a study [65]; and
6) 97% of respondents (65/67) agreed that making another stop after one stop in a short time was annoying and dangerous, and many of these respondents (31/65) considered the time interval between two consecutive stops should be at least 20 seconds.

These findings have been considered during the development of a grading evaluation framework to measure the perceived quality of arterial signal timing, which is presented in Section 3.7.

These surveys were designed to empirically characterize human perceptions of signal operations for engineering research purposes. Most of the survey questions aimed to validate the hypotheses envisioned by expert traffic signal timing professionals. Practitioners and researchers should use care when applying these findings elsewhere. The ratings of the overall perceived quality of signal timing can be measured by the number of stops per intersection, following a form of a logistic function [56].

3.3.2 Attainability of User Satisfaction (AUS)

The perceived quality of signal timing correlates directly with the attained transportation user satisfaction, and the explanatory variable is identified. The optimal user satisfaction is specified as "vehicles make no stops at any of the signals involved in the evaluation," and the user satisfaction would diminish as the stop time and the number of stops increases. A performance measure was defined during this study in light of the investigated stop characteristics. The attainability of user satisfaction (AUS) describes the probability that the travelers would rate the quality of arterial signal timing as the best, which can be computed using Equation 2.

$$AUS = 1 - P_{not\ satisfied} = 1 - \left(1 + e^{-w \times \frac{Stop\ Equivalency}{Total\ Number\ of\ Signals}}\right)^{-1} \quad (2)$$

where

AUS – the attainability of user satisfaction; $P_{not\ satisfied}$ – the probability that the travelers would not be satisfied with signal timing; *Stop Equivalency* – the standardized number of stops considering the trade-off between stop time and the number of stops, the allergy of consecutive stops in a short time, and the tolerability variation of stops at different intersections; w – a coefficient which can be determined through user satisfaction surveys.

3.4 Development of Performance Measurement Methodology

The proposed performance measurement Methodology is presented in this section. The performance measurement results are expressed in different grades, with a hierarchical ordering that varies from Level F (worst rating), Level D, Level C, Level B, to Level A (best rating). The methodology is comprised of three primary components – the AIP (the attainability of ideal progression) scoring, the AUS (the attainability of user satisfaction) scoring, and the adjustments. A typical procedure is presented in Figure 3.6, which identifies the sequence of calculations needed.

It should be noted that the proposed performance measurements should be applied to the signal systems which are coordinated already or capable of adaption. Non-coordinated signals, as well as unsignalized intersections (e.g., two-way stop control intersections, all-way stop control intersections, and roundabouts), should be excluded from the evaluation. For the unsignalized intersections such as for two-way stop intersections or roundabouts where the main-street traffic is not frequently interrupted, they can be considered as general roadway segments. If the arterial traffic is obviously influenced by these mid-block accesses, the evaluation scope should be adjusted accordingly, e.g., the arterial can be partitioned into several segments where the intersections involved are all signalized and coordinated.

The steps included in the proposed performance measurement methodology are summarized below.

Step 1: Determine Intersection Classifications

Intersection classification is a new concept created in this research. As interpreted previously, many factors (e.g., congestion level along the arterial and the proportion of traffic from side streets to the overall arrivals) should be considered in order to convert average speed and stops into the values of AIP and AUS. The conversion is usually too complicated to use in daily practice due to a large amount of data collection required, and as a result, intersection classification was developed as a means to simplify the process. Intersection classification is determined by arterial volume-to-capacity ratio and intersection geometric information. Section 3.5 describes more details of intersection classification.

Step 2: Determine AIP Scores

The scores of the attainability of ideal progression (AIP) are determined during this step.

Step 3: Determine AUS Scores

This step is to determine the scores of the attainability of user satisfaction (AUS).

Step 4: Determine Scoring Adjustments

During this step, any necessary scoring adjustments are determined to fine-tune the resulting AIP and AUS scores. Two types of scoring adjustments were defined in this research: 1) the cycle length adjustment and 2) the intersection spacing adjustment. More details are presented in Section 3.8.

Step 5: Determine Performance Grades based on Adjusted AIP and AUS scores

Based on the adjusted AIP and AUS scores, the performance grades are generated for travel-run routes.

Step 6: Determine the Quality of Signal Timing

After combining the performance grades for various travel-run routes, the quality of signal timing is finally determined.

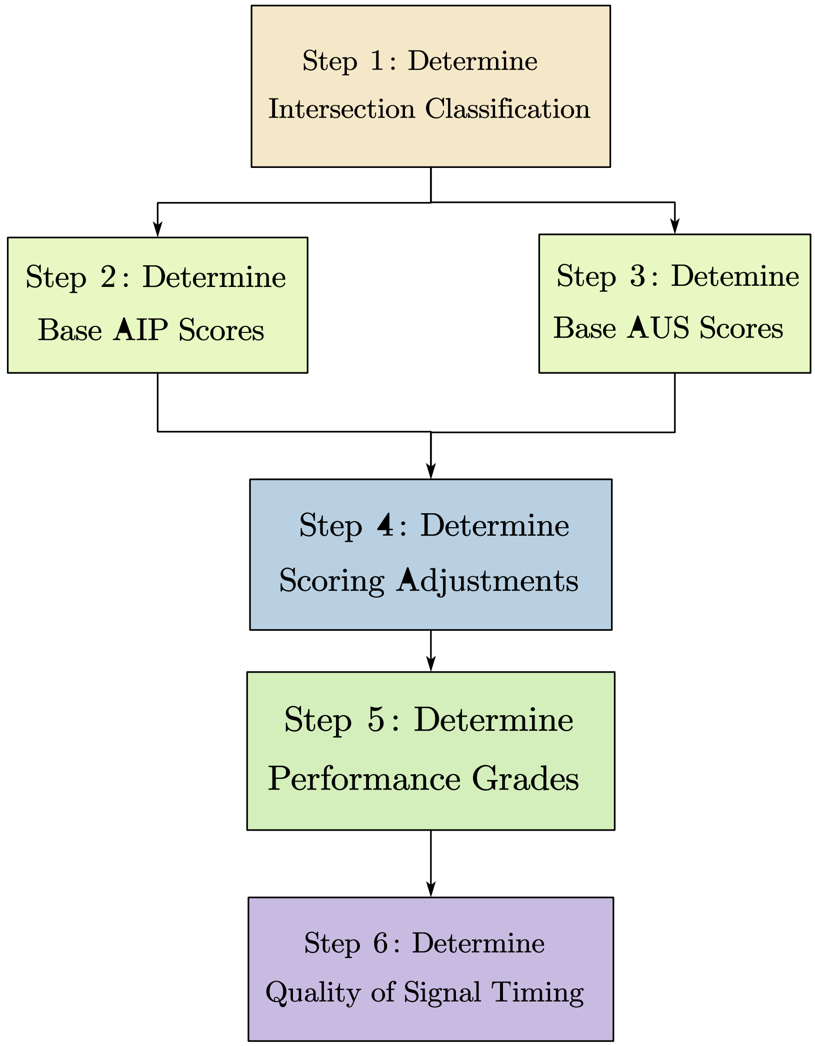


Figure 3.6 Calculation Procedure of Proposed Methodology

3.5 Determination of Intersection Classification

Excluding the influences of the non-signal-timing factors such as arterial congestion level and standardizing the number of stops by considering stop characteristics such as the stop time and the interval between two consecutive stops may involve complicated calculations, which cannot be easily implemented in real-world practice. In an effort to address this problem, a simplified method is provided to perform this process based on Intersection Classification, a newly defined parameter developed during this research.

Intersection Classifications (IC) is designated at individual intersections to describe the ease or difficulty of achieving free-flow-speed arterial operations and the optimal user satisfaction. The determination of IC is on the basis of two major considerations. For the arterials (also called the main streets), the arterial volume to capacity (V/C) ratio is primarily considered, which is computed using Equation 3.

Given a signal timing *i* at intersection *m*, the arterial volume-to-capacity ratio_i can be calculated using

$$Arterial\ V/C\ Ratio_{i,m} = \frac{q_{i,m}}{(n_m \times g_{i,m})} \times \frac{cl_{i,m}}{S} \tag{3}$$

where

$Arterial\ V/C\ ratio_{i,m}$ – the arterial volume-to-capacity ratio at intersection *m* for signal timing *i*;

$q_{i,m}$ – average hourly traffic counts in two directions along the main street at intersection *m* during the operating time of the signal timing *i*. This data can be obtained from regular traffic volume counts;

$g_{i,m}$ – the average green time of the arterial phases (through phases and left-turn phases along the main street) in two directions at intersection *m* during the operating time of the signal timing i. The value of g_i can be determined based on the logged splits history or the designed green splits (except for adaptive signals);

$cl_{i,m}$ – average cycle time during the operating time of the signal timing i. The value of cl_i can be determined based on the logged cycle time history or the designed cycle length (except for adaptive signals);

S – saturation flow per lane, typically 1800 vehicles per hour; and

n_m – number of lanes in two directions along the main street at intersection m, including the exclusive left-turn or right-turn lanes at the intersection.

For the side streets, the traffic volumes for left-turning and/or right-turning onto the arterial should be considered, which can be reflected by turning movement counts. However, turning movement count data may not be available or up-to-date in practice, and collecting turning movement counts can be costly and labor-intensive. An alternative way is to count the number of lanes on the side streets, which roughly represents the side-street traffic demand in lieu of turning movement counts.

Table 3.4 exhibits the five types of IC. The Type-I or Type-II IC at the intersection can indicate that the arterial traffic operations are approaching saturation, and the side-street traffic demands are heavy. Therefore, the highest achievable progression speed along the arterial may decrease, and drivers may tend to tolerate longer stop time at the Type-I and Type-II IC intersections than at the Type-III, Type-IV, or Type-V IC intersections.

Table 3.4 Determination of Intersection Classifications

Side Street \ Arterial	$0 <$ Arterial V/C ≤ 0.3	$0.3 <$ Arterial V/C ≤ 0.55	$0.55 <$ Arterial V/C ≤ 0.85	Arterial V/C > 0.85
Number of lanes in two directions ≤ 3	Type V	Type IV	Type III	Type II
$3 <$Lanes in two directions ≤ 7	Type IV	Type III	Type II	Type I
Lanes in two directions > 7	Type II	Type II	Type I	Type I

Supplementary Clauses:

1) Signalized freeway interchanges should be regarded as one intersection (even though some interchanges have two physical signals), and the Intersection Classification of the interchange is assigned as Type-I IC (if the arterial V/C ratio ≥ 0.55) or Type-II IC (if the arterial V/C ratio < 0.55); and
2) If the side street has been coordinated at the intersection, which means the change of offset or phasing sequence is restricted to a certain extent, the type number should be decreased by one, e.g., Type-III IC changes to Type- II IC if the side street is coordinated already.

The proposed intersection classification (IC) method is analogous to the conventional intersection classification approaches based on the annual average daily traffic (AADT) data but places emphasis on signal control related features.

Figure 3.7 shows the arterial and cross-street AADT distributions for the five groups of Type-I, -II, -III, -IV, and -V intersections identified in Reno/Sparks and Las Vegas metropolitan areas in Nevada, which demonstrates the similarities and differences between the proposed intersection classification (IC) method and the AADT-based classification approaches. The AADT data were collected through the Traffic Records Information Access (TRINA) database operated by the Nevada Department of Transportation (NDOT).

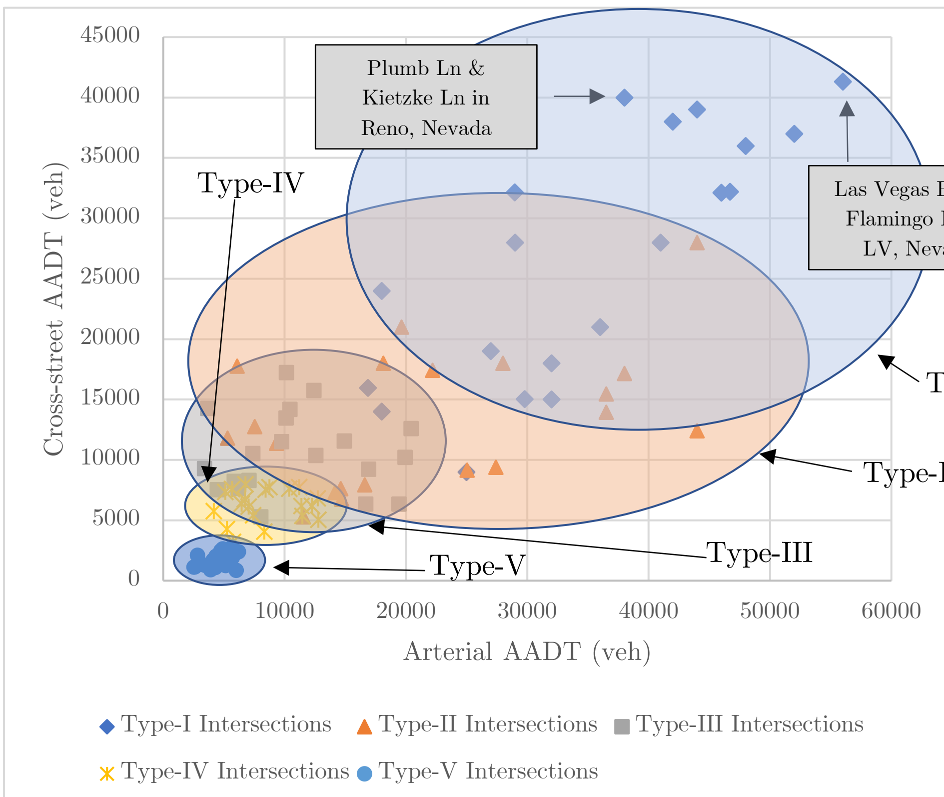


Figure 3.7 AADT Distributions of Type-I, II, III, IV, and V Intersections

In addition, Figure 3.7 indicates the potential ranges of the arterial and cross-street AADTs for Type-I, -II, -III, -IV, and -V IC intersections. The arterial and cross-street AADTs generally increase as the IC type changes from Type-V to Type-I. For instance, for Type-V IC intersections, the cross-street AADT is usually lower than 5,000, and the arterial AADT is less than 10,000. In contrast, for the Type-IV IC intersections, the arterial and cross-street AADTs are higher than 4,000 and 5,000, respectively. However, Type-II and Type-I IC intersections can have arterial AADT ranging from 10,000 to 50,000 and cross-street AADT ranging from 5,000 to 45,000. This indicates some low-volume intersections may be considered the Type-I or Type-II IC intersections; thus, the proposed IC method is different from the conventional classification approaches only using AADT data.

3.6 Determination of AIP Score

Given a signal timing i for a total of M intersections (1,2,3, ..., m) and a total of N travel-run samples (1, 2, ..., n) collected along a route j (typically route j is one of the two through movements in the two directions across the M intersections on the arterial), the AIP score can be computed using Equation 4,

$$AIP\ Score_{i,j,n} = \min\left(\frac{Average\ Speed_{i,j,n}}{Ideal\ Progression\ Speed_j} \times 100,\ 100\right) \tag{4}$$

where

$AIP\ Score_{i,j,n}$ – the score of the attainability of ideal progression for the signal timing i along the route j for the travel run n, the upper bound of AIP score is 100;

$Average\ Speed_{i,j,n}$ – the average speed for the travel run n across a total of A intersections along the route j ($A \leq M$) during the running time of the signal timing i, mph;

Ideal Progressive Speed$_j$ – the ideal progressive speed across a total of A intersections along the route j ($A \leq M$), mph, which can be calculated using Equation 5:

$$Ideal\ Progressive\ Speed_j = FFS_j \times \max(\alpha^{N_I} \times \beta^{N_{II}} \times \gamma^{N_{III}} \times \tau^{N_{IV}} \times \upsilon^{N_V}, 0.5) \quad (5)$$

where

FFS$_j$ – the free-flow speed across a total of A intersections along the route j ($A \leq M$), expressed in miles per hour (mph), which typically equals to the posted speed limit plus five mph;

N_I, N_{II}, N_{III}, N_{IV}, and N_V – the number of the type-I, -II, -III, -IV, and -V IC intersections respectively among these A intersections.

α, β, γ, υ, and τ – the coefficients which represent the impacts of intersections with different IC types on the ideal progressive speed. α and β are recommended to be 0.9 and 0.95, respectively. This means that the existence of Type-I or Type II intersections can reduce the ideal progressive speed. The greater number of Type-I and Type-II intersections involved in the study, the lower ideal progressive speed that can be used in the calculation. γ, υ, and τ are suggested to be 1, which implies the type-III, IV, and V IC intersections barely affect the average speed. These coefficients can be determined based on specific travel-run trajectories, which will be further described in the following rationale paragraphs. The recommended values of the coefficients were determined according to the data collected in Reno/Sparks metropolitan areas in Nevada.

Rationale

Equation 4 is derived according to the performance measure AIP defined in the previous section. The concept of "ideal progression speed" is highlighted as the highest achievable operating speed by signal timing optimization in the contexts of various non-signal-timing factors.

The ideal platoon speed can be affected by the arterial congestion level, arrival flow profile, or signal density, as interpreted in Section 3.2. These factors can be simplified as two explanatory variables – the IC types and the numbers of intersections of different IC types. The coefficients presented in Equation 5 can be determined through regression analyses using GPS trajectory data. The "ideal platoon speed" can be estimated through some specific trajectories such as the callouts i, ii, and iii illustrated in Figure 3.8. These trajectories indicate the travel runs without being halted by the signals, which represent the ideal operating speed that can be achieved by progression. Hence, using the trajectory data collected along multiple arterials, the correlation between the "ideal platoon speed" and the "IC types/the numbers of intersections of different IC types" can be revealed (e.g., as shown in Figure 3.8, an observation of the variable "Type-I: 2, Type-II: 0, Type-III: 3, Type-IV: 2, and Type-V: 1" is obtained as the average speed measured according to the trajectories i, ii, and iii). The trajectory segments where the vehicles move smoothly along the arterial can be used as well, e.g., the callout iv exhibited in Figure 3.8 can be regarded as an observation of "Type-I: 2, Type-II: 0, Type-III: 1, Type-IV: 2, and Type-V: 0" from which the three intersections not covered by the trajectory segment are excluded.

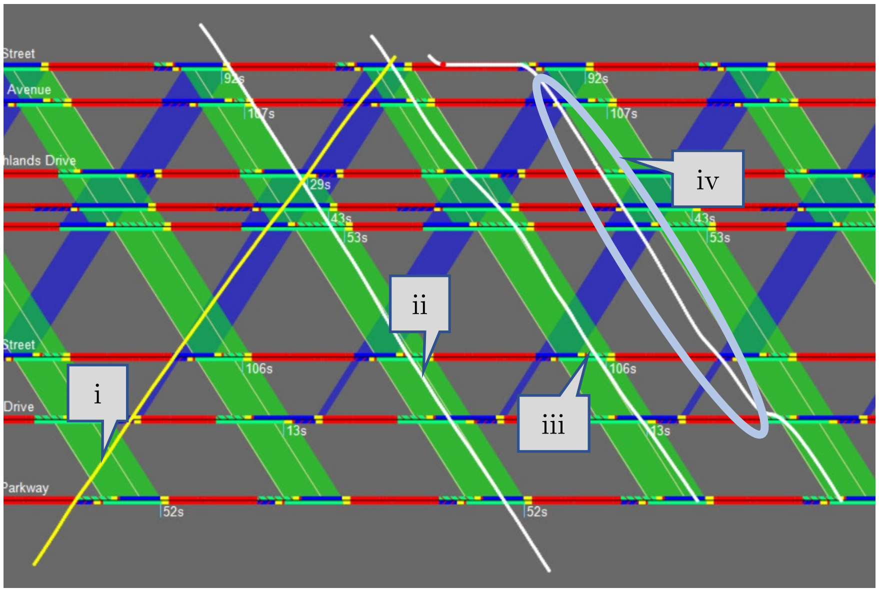


Figure 3.8 Ideal Progression Speed Measured by Trajectories

Based on the cases in Reno, Nevada, a multivariate power function was obtained as Equation 5 in which α=0.8862, β=0.9391, γ=1.024, υ=0.9877, τ=1.0080, and the constant =3.2146 (R^2=0.61). Therefore, it is suggested that α= 0.9, β=0.95, γ=1, υ=1, and τ= 1. In addition, because extreme cases such as the arterial include more than six Type-I IC intersections are scarcely found in the real world, the function may not be reliable as the numbers of Type-I or Type-II IC intersections increase. A lower bound, which is "50% of the free-flow speed," has been added to Equation 5 according to the finding presented in Section 3.2.

3.7 Determination of AUS Score

Given a signal timing i for a total of M intersections (1, 2, 3, ..., m) and a total of N travel-run samples (1, 2, ..., n) collected along a route j (typically route j is one of the two through movements in the two directions across the M intersections on the arterial), the AUS score can be calculated using Equation 6,

$$AUS\ Score_{i,j,n} = 100 - \left(\frac{50}{1 + \exp\left(- \frac{SPI_{i,j,n} \times 2 - 65}{10} \right)} \right) \tag{6}$$

where

$AUS\ Score_{i,j,n}$ – the score of the attainability of user satisfaction for the signal timing i for the route j for the travel run n. AUS scores range from 0 to 100;

$SPI_{i,j,n}$ – Stop equivalency per intersection shown by the travel-run sample n (one of the total N samples) for the signal timing i for the route j. The value $SPI_{i,j,n}$ is obtained using Equation 7,

$$SPI_{i,j,n} = \frac{\sum_{a=1}^{A} \left(Stop\ Equivalency_{a,n} \times \max\left(\frac{0.1}{Stop\ Distance_{a,a-1,n}}, 1 \right) \right)}{Number\ of\ Intersections_j,\ A} \tag{7}$$

where

$Number\ of\ Intersections_j$ – the number of intersections along the route j, expressed as A (1, 2, 3, ...a, $a \leq m$);

$Stop\ Distance_{a,a\text{-}1,n}$ – the shortest distance between the stops at the a^{th} intersection and the stops at the a-1th intersection of the route j, measured by the travel-run n, mile;

$Stop\ Equivalency_{a,n}$ – the value of the stop equivalency at the a^{th} intersection among A intersections, measured by the travel-run n, determined using Equation 8:

$$Stop\ Equivalency_{a,n} = \sum_{k=1}^{K} \begin{cases} 0 & Stop\ Time_k < 3s \\ 0.5 & 3s \leqslant Stop\ Time_k < 10s \\ 0.5 + \dfrac{Stop\ Time_k - 10}{\phi_a \times Cycle\ Length_{a,i}} & Stop\ Time_k \geqslant 10s \end{cases} \tag{8}$$

where

$Stop\ Time_k$ – the duration time of the k^{th} stop at the a^{th} intersection, assuming there are a total of K stops at the intersection ($K \geq 0$), second;

$Cycle\ Length_a$ – the average cycle length of the signal timing i at the a^{th} intersection, second. The cycle length is a constant value in most cases. But it could be varying if the signal timing i uses adaptive adjustments, and then $Cycle\ Length_a$ is determined by averaging.

Φ_a – a coefficient related to the Intersection Classification (IC) of the a^{th} intersection. Φ_a is recommended to be 0.5 if the a^{th} intersection is the Type-I IC intersection. Φ_a is recommended to be 0.25 if the a^{th} intersection is the Type-II or Type-III IC intersection. And Φ_a is recommended to be 0.15 if the a^{th} intersection is the Type-IV or Type-V IC intersections.

Rationale

Equations 6 is a modified logistic function according to Equation 2. As exhibited in Figure 4.2, if the SPI is in range A, most drivers are not aware of making stops while driving along the arterial. With the increasing value of the SPI, drivers may begin to question or complain about the quality of signal timing service; therefore, the user satisfaction considerably diminishes when the SPI is in range B. If the SPI increases into range C, most drivers' perception of the quality of signal timing will be unsatisfactory, and a score lower than 60 will result. An AUS score less than 60 represents the perceived quality of arterial signal timing is unacceptable.

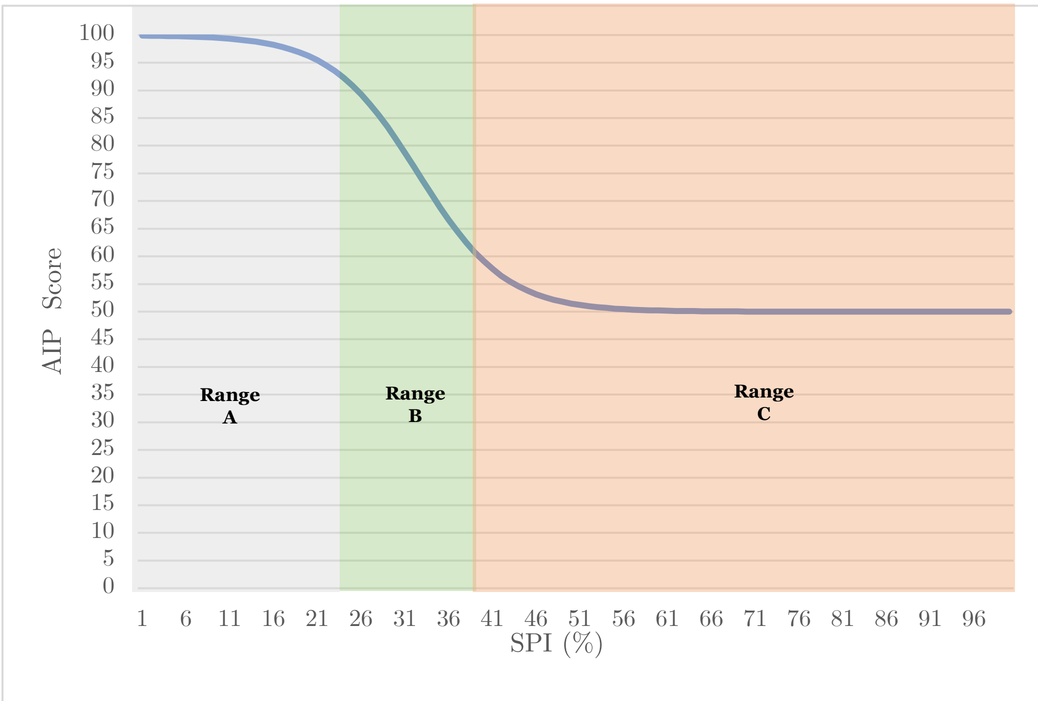


Figure 3.9 AUS Scoring Curve

Equation 6 was designed to converge at an AUS score of 50. This aims to prevent the AUS score from being affected by rare samples. Unlike the AIP score, the AUS score can measurably change due to incidental interferences, e.g., the vehicles can be stopped because the preceding vehicles are inattentive. These stops caused by incidents are not recurrent usually; however, the AUS score may significantly decrease according to the notion of satisfaction changing over the course of making stops. For example, if nine travel-run samples show AUS scores of 90, but one sample can result in an AUC score of 20 if the function is not modified, the average AUS score would be only 83, which is not convincing to most signal timing practitioners. In addition, the change of AUS score can be more sensitive when the number of evaluated signals is small, e.g., when there are only three intersections involved in the performance measurement, a trip with the stop equivalency of 1.5 may result in an AUS score below 30 whereas a travel run with the stop equivalency of 1 can cause an AUS score of 75 if a typical logistic function is adopted.

In Equation 7, a penalty is added according to the survey finding – “making two consecutive stops within a short distance or over a short time interval is annoying and dangerous.” The distance of 0.1 miles (528 feet) is selected as the threshold to differentiate the usual stops, and the short-distance stops to be penalized. Most vehicles in the United States can achieve an acceleration performance of 0-60 mph within 8 seconds [66]. The distance required for vehicle acceleration from 0 mph to 40 mph (40 mph here is considered a typical operating speed along urban arterials) is around 400 feet, which suggests that vehicles most likely just finish accelerating and then are forced to stop if the two stops are less than 0.1 miles apart. This is also considered as “two stops taken in a short time” as the acceleration plus the reaction time within 0.1 miles usually are typically less than 20 seconds, which may make most drivers may feel unsafe and annoyed.

In Equation 8, stop equivalency is calculated instead of merely counting the number of stops. Stop equivalency accounts for the fact that the longer amount of time the drivers wait at the intersection, the worse perception they typically have of signal operations. Also, based on the finding presented in Section 3.3, the calculation stop equivalency considers two variables – cycle length and Intersection Classification (IC). Figure 3.10 illustrates how the stop equivalency increases along with the increasing stop time regarding "Type-I IC," "Type-II or -III IC," and "Type-IV or -V IC," respectively, under 90-second cycle length. In practice, the possible stop times correlate with the cycle length. If the cycle length is 90 seconds, the green time allocated to the arterial through phases is typically about 50% of the cycle length, namely 45 seconds in this case. Therefore, the longest wait the drivers may experience at this intersection is 45 seconds (e.g., the vehicle is stopped just when the yellow indication turns red), which is why cycle length should be considered in the calculation of stop equivalency. In addition, as shown in Figure 3.10, the three traces of the IC types have different slopes. For the Type-I IC intersections, stop time is more tolerable, which is reflected by a flatter slope than the other two traces.

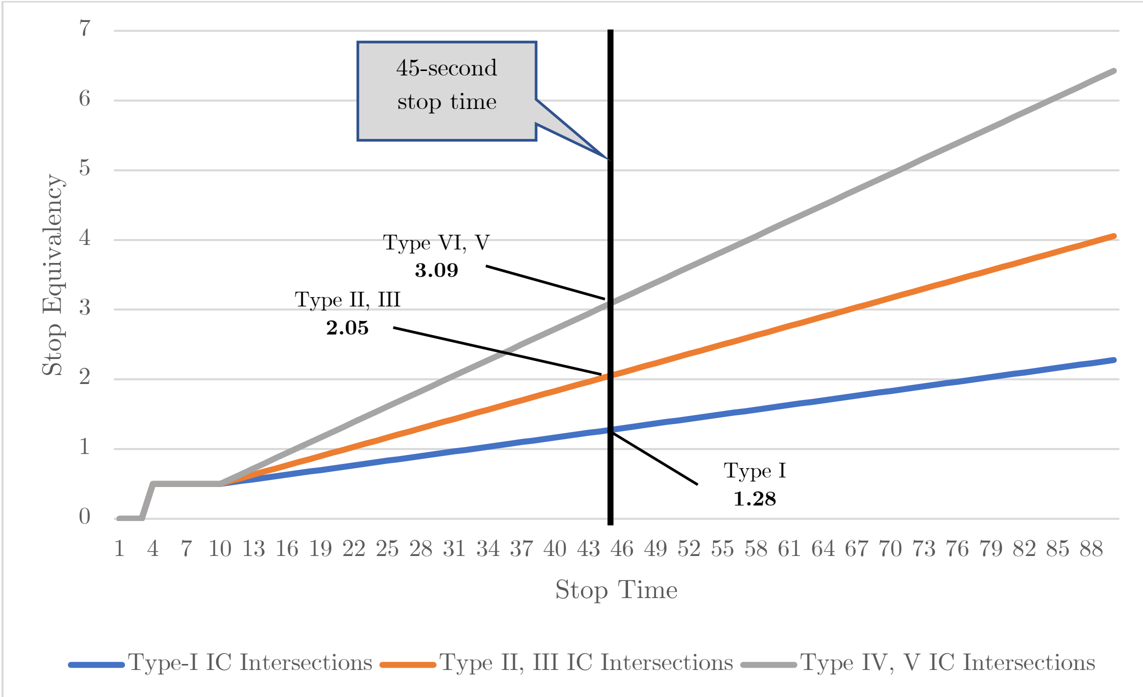


Figure 3.10 Stop Equivalency for Different IC Types under 90-second Cycle Length

3.8 Determination of Scoring Adjustments

There are two scoring adjustments used in the proposed methodology – cycle length adjustment and intersection spacing adjustment.

3.8.1 Cycle Length Adjustment

The cycle length adjustment is created to reward the quality of signal timing achieved by a short cycle length and to penalize the same quality that results from a redundantly long cycle length. A short cycle length often leads to reduced delay times for side-street traffic and pedestrians.

Cycle length adjustment is mainly determined by the value of system average cycle length (SACL), which is calculated using Equation 9. Given a signal timing i for a total of M intersections (1,2,3, ..., m)

$$SACL_{i,j} = \frac{\sum_{a=1}^{A} Cycle\ Length_{a,i,j}}{Number\ of\ Intersections_j,\ A} \tag{9}$$

where

$SACL_{i,j}$ – the system average cycle length for signal timing i for route j. Typically, route j is one of the two through movements in two directions across the M intersections along the arterial;

$Number\ of\ Intersections_j$ – the number of intersections along route j, expressed by A (1, 2, 3 ..., a; $a \leq m$);

$Cycle\ Length_{a,i,j}$ – the cycle length of signal timing i at a^{th} intersection along route j. The signals in coordination are usually timed with only one common cycle length; however, if there are the intersections where using the common cycle length is infeasible or inappropriate, alternative cycle lengths, such as one-half or two-times of the common cycle length, can be adopted so that the opportunity for a cyclic progression remains. For adaptive signals, $Cycle\ Length_{a,i,j}$ can be determined by averaging the historical cycle times recorded during the operating time interval of signal timing i because the cycle length of an adaptive signal system keeps varying according to the change of traffic flows.

Table 3.5 shows the determination of the cycle length adjustment. As can be seen, the value of the cycle length adjustment may be negative; hence, the AIP and AUS scores will be reduced if the system average cycle length is deemed too long. The practitioner may be able to shorten the value of SACL through the use of the half-cycle maneuver in order to avoid a reduction of scores.

Table 3.5 Cycle Length Adjustments

SACL (second)	**The Value of Adjustment**
SACL $>$ 160s	-5
140s $<$ SACL $\leq$ 160s	-2
90s $<$ SACL $\leq$ 140s	0
70s $<$ SACL $\leq$ 90s	+2
SACL $\leq$ 70s	+5

The values listed in Table 3.5 are derived empirically, mainly based on the judgments provided by local expert engineers regarding the general cases they have encountered in practice. Additionally, a few exemptions of the cycle length adjustment are listed as follows:

Exemptions

1) If a cycle length longer than 140 seconds is used due to capacity issues, e.g., any other cycle lengths (shorter than 140 seconds) cannot provide an adequate capacity at the intersections, negative adjustments should not be applied to the case; and
2) If a cycle length longer than 140 seconds is used due to geometric constraints, e.g., any other cycle lengths (shorter than 140 seconds) cannot accommodate

the requirements of pedestrian timing, negative adjustments should not be applied to the case.

3.8.2 Intersection Spacing Adjustment

Intersection spacing (or street spacing) is a factor that should be considered during signal timing performance measurements because intersection spacing may affect the ease or difficulty of performing bi-directional signal coordination along arterials [67]. Even if a two-way progression can be achieved under various intersection spacings by adjusting cycle length, offsets, and phase sequences [68, 69], it would be challenging to provide high-quality two-way progression along the arterial if intersections are spaced closely together.

Typically, compact intersection spacings are observed in the central business districts (CBD), where queue management is also crucial for arterial operations. Queuing control may be achieved at the expense of arterial progression. As a result, the scores should not be negatively impacted due to the need for queue management.

For determining the intersection spacing adjustment, an index was developed based on the proportion of the number of close-spacing intersections to the total number of evaluated intersections, which is computed using Equation 10. For an evaluation with a total of M intersections (1, 2, 3, ..., m),

$$PCSI_j = \frac{n_{spacing \leqslant 1,000\ feet,\ j}}{Number\ of\ Intersections_j,\ A} \tag{10}$$

where

$PCSI_j$ – the proportion of closely spaced intersections to all involved intersections along route j, and typically route j is one of the two through movements in the two directions across the M intersections along the arterial;

$Number\ of\ Intersections_j$ – the number of intersections along route j, expressed by A (1, 2, 3 ..., a; $a \leq m$);

$n_{spacing \leq 1{,}000\ feet,\ j}$ – the number of intersections where the distance between either of the two neighboring intersections is less than 1,000 feet. One thousand feet is selected as the threshold because the FHWA *Signalized Intersections Informational Guide* [54] documented that signal spacing less than 1,000 feet is difficult to coordinate.

Table 3.6 shows the determination of intersection spacing adjustments, which is an incentive adjustment, as all adjustment values are nonnegative.

Table 3.6 Intersection Spacing Adjustments

PCSI	**Value of Adjustment**
0.75 < PCSI ≤ 1	4
0.5 < PCSI ≤ 0.75	2
0.25 < PCSI ≤ 0.5	1
0 < PCSI ≤ 0.25	0

The values provided in Table 3.6 are empirically determined, mainly based on the judgments of local expert engineers.

3.9 Determination of Performance Grades

Based on AIP and AUS scores, performance grades can be determined to describe the performance of a signal timing plan along a specific route through multiple travel runs. The AIP and AUS scores obtained from individual trajectory samples need to be aggregated into the general scores.

Given a signal timing I for a total of M intersections (1, 2, 3, ..., m) and a total of N travel-run samples (1, 2, ..., n) collected along a route j (typically the route j is one of the two through movements in the two directions across the M intersections on the arterial), the aggregate AIP and AUS scores can be obtained by applying Equations 11 and 12:

$$Aggregate\ AIP\ Score_{i,j} = \frac{\sum_{n=1}^{N}(AIP\ Score_{i,j,n} + CLA_{i,j} + ISA_{j})}{Number\ of\ Samples_{i,j},\ N} \tag{11}$$

$$Aggregate\ AUS\ Score_{i,j} = \frac{\sum_{n=1}^{N}(AUS\ Score_{i,j,n} + CLA_{i,j} + ISA_{j})}{Number\ of\ Samples_{i,j},\ N} \tag{12}$$

where

Aggregate AIP/AUS $Score_{i,j}$ – the aggregate scores of AIP and AUS for signal timing I along route j;

$CLAi,j$ – the value of cycle length adjustment for signal timing I along route j;

ISAj – the value of intersection spacing adjustment along route *j*. Intersection spacing adjustment does not correlate to specific signal timings;

Number of Samples$_{i,j}$ – the total number of travel-run samples collected for signal timing *I* along route *j*.

Table 3.7 shows the determination of performance grades based on aggregate AIP and AUS scores.

Table 3.7 Determination of Performance Grades

Aggregate AIP Score / Aggregate AUS Score	AIP ≥ 90	80 < AIP ≤ 90	70 < AIP ≤ 80	60 < AIP ≤ 70	AIP < 60
AUS ≥ 90	A	A	B	C	N/A
80 < AUS ≤ 90	A	B	B	C	N/A
70 < AUS ≤ 80	B	B	C	D	N/A
60 < AUS ≤ 70	C	C	D	D	F
AUS < 60	F	F	F	F	F

3.10 Determination of Quality of Signal Timing

Performance grade is a route-based notion, but the quality of signal timing is supposed to represent the performance of the signal timing plan by combining the performance grades of the routes of interest (typically the two through movements along the arterial in the two directions). The performance grade of the routes should be aggregated by simply taking the average because the importance of each route may not be the same. Different weights may be placed on the routes accounting for the engineering judgments, e.g., if the route is along the transit path and the travel-run trajectories are collected by transit buses, the resulting performance grade should be emphasized when transit signal priority is one of the signal timing operational objectives.

An index named Route Priority Factor (RPF) is proposed in this research, which can be used for drawing the conclusion of the quality of signal timing based on multiple performance grades.

Given a total of *J* routes (1, 2, 3 ..., *j*) considered in the determination of the quality of signal timing *i*, the PRF of route *j* can be calculated using Equation 13

$$RPF_j = \frac{W_j Q_{i,j}}{\sum Q_{i,(1,2,3\ldots J)}} \tag{13}$$

where

RPF_j – route priority factor of route *j*;

$Q_{I,j}$ – traffic volume counts along route *j* during the operating time of signal timing *i*;

W_j – weight for route *j* which can be determined regarding the characteristics of route *j* such as traffic modes involved (e.g., bicycle traffic or transit traffic), whether route *j* is a part of major commuting paths, whether route *j* is a part of ingress/egress paths for special events or evacuations.

$Q_{i,\ (1,2,3\ldots,J)}$ – traffic volume counts of each route (1, 2, 3, ..., *J*) among the *J* routes during the operating time of signal timing *i*.

The weights should be carefully determined due to potential equity issues. It is common to see in practice that traffic demands of the two directions along the arterial are distinctly uneven; however, it is not appropriate to disregard the direction with lighter traffic demand favoring the other direction. In order to avoid the equity issues, Tables 3.8, 3.9, and 3.10 exhibit an example performed to determine the quality of arterial signal timing, assuming there are two routes involved.

Table 3.8 Determination of the Quality of Signal Timing (if 0.5 $\leq$ the Greater RPF between Two RPFs $<$0.7)

Minor Direction \ Major Direction	**A**	**B**	**C**	**D**	**F**
A	A	A	C	C	F
B	A	B	C	C	F
C	B	C	C	D	F
D	C	C	D	D	F
F	F	F	F	F	F

Table 3.9 Determination of the Quality of Signal Timing (if 0.7 ≤ the Greater RPF between Two RPFs <0.9)

Minor Direction \ Major Direction	**A**	**B**	**C**	**D**	**F**
A	A	B	B	C	F
B	A	B	C	D	F
C	B	B	C	D	F
D	C	C	D	D	F
F	F	F	F	F	F

Table 3.10 Determination of the Quality of Signal Timing (if 0.9 ≤ the Greater RPF between Two RPFs < 0.1)

Minor Direction \ Major Direction	**A**	**B**	**C**	**D**	**F**
A	A	B	C	D	F
B	A	B	C	D	F
C	A	B	C	D	F
D	B	C	C	D	F
F	C	D	F	F	F

Ultimately, the quality of arterial signal timing can be reached. Table 3.11 presents the descriptions of the quality of arterial signal timing grades.

Table 3.11 Description of the Quality of Arterial Signal Timing Grades

Quality of signal timing	Description
A	Excellent quality of signal timing
B	Good quality of signal timing
C	Re-timing could significantly improve the operations
D	Re-timing is strongly recommended
F	Re-timing is urgently required

3.11 Summary

This chapter mainly described the performance measurement methodology proposed in this research for evaluating the quality of arterial signal timing.

Firstly, two performance measures – the attainability of ideal progression (AIP) and the attainability of user satisfaction (AUS) were defined based on speed and stop characteristics, which can be extracted from travel-run trajectories. Non-signal-timing factors such as arterial congestion level and arrival flow profile were explored regarding the effects on arterial travel speed. In addition, driver surveys were conducted to reveal the perceived quality of arterial signal timing, and the preliminary findings were presented.

Then, the proposed signal timing performance measurement framework was stated. The framework included AIP scoring, AUS scoring, and scoring adjustments regarding system average cycle length (SACL) and the proportion of closely spaced intersections (PCSI).

A new parameter, Intersection Classification (IC) was introduced to simplify the calculations of AIP and AUS. Intersection Classification focuses on the intersection features related to signal timing, which is different than conventional intersection classifications based on AADT.

The determinations of AIP and AUS scores, as well as the cycle length and intersection spacing adjustments, were also presented.

Lastly, the performance grades could result from the adjusted AIP and AUS scores. The quality of arterial signal timing could be reached by combining the performance grade of the routes of interest.

Chapter 4 METHOD IMPLEMENTATION

4.1 Overview

This chapter outlines the implementation of the proposed signal timing performance measurement in terms of the data gathering, the data processing, and the pre-implementation examination.

4.2 Data Gathering

GPS trajectories recorded during travel runs are considered an ideal data source for the proposed methodology. Figure 4.1 shows a snippet of a travel-run trajectory recorded by a smartphone APP [70]. Compared to the conventional GPS units, the APP can be directly downloaded via the online APP stores and installed on mobile devices, which are compact and comfortable to carry. Signal timing practitioners can quickly get started with the smart device-based APP and conveniently use it while driving along the arterial. The gathered GPS data will be saved online, which also can facilitate data management and exchange.

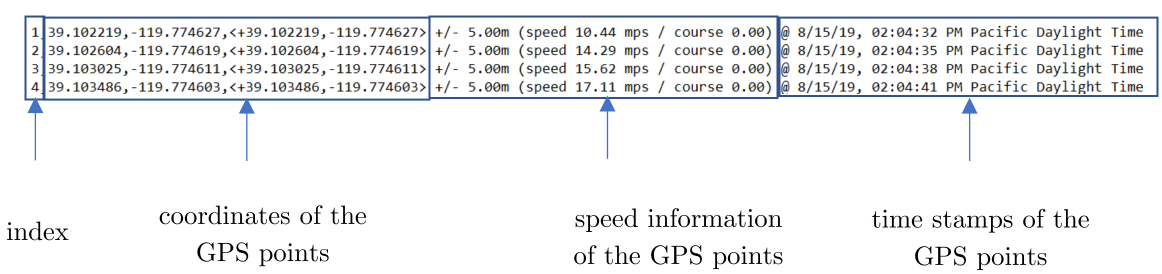


Figure 4.1 Information Provided by GPS Trajectories

Implementing the proposed performance measurement should use high-resolution GPS trajectories that can provide the information, including average speed, the number of stops, the time duration of stops, and the stop locations. Any formats of GPS trajectory that contain such information can be used as well, e.g., the segmented probe vehicle data (PVD) with time-stamped average segment speed. This data set can be obtained from some nationally accessible databases such as the National Performance Measure Research Data Set (NPMRDS) or third-party data vendors such as INRIX.

The GPS trajectories should be collected for performance measurement purposes during the operating time intervals of the signal timing plans. Practitioners can conduct the trajectory data collection by driving along the arterial back and forth with the abovementioned trajectory recording APP during the coordination periods. Or, practitioners can acquire GPS data from the third-party sources according to certain spatial and temporal focuses.

Nevertheless, some requirements should be fulfilled when the trajectory data are used for evaluating the quality of arterial signal timing, which are listed as follows:

1) Data Resolution

The trajectory data may have varying ping frequencies. Detailed travel-run trajectories can be described with the time resolution at one-per-second, which can be achieved using the previously mentioned APP. For other sourced trajectory data, at least one GPS point must be recorded every three seconds. This minimal resolution requirement aims to recognize the smallest stop equivalency. As presented in Equation 8, a stop equivalency can result when a vehicle is stopped by the signal for more than 3 seconds. Therefore, if the ping frequency of the trajectory data is more extensive than once per three seconds, some stops may be neglected.

In addition, the speed and location information included in the trajectory data should possess the proper resolution. The decimal degrees of latitude and longitude geographic coordinates should have a precision of at least six decimal places, in which feet-level motions can be unambiguously recognized. The precision of speed information usually depends on the resolution of coordinates and timestamps. It is required that the speed variation in a scale of miles per hour (mph) should be recognized as the stop is identified according to the decrease of speed. In this study, a stop is recognized when the speed drops lower than five mph [49].

2) Sampling Size

For the proposed performance measurement, the minimal requirements of the sampling size are categorized into two types – 1) ad-hoc performance measure studies and 2) daily performance monitoring.

For ad-hoc performance measure studies, this research suggests that at least four trajectories per hour per route need to be collected during the operating time of the signal timing (the data collection should be conducted within the same time-of-day periods but can be separated into different days). A conclusion drawn by one study [49] indicated that the trajectory penetration rates of less than 0.04% could be used to assess signal timing performance. Given one of the highest urban arterial AADTs in Nevada – 101,000 vehicles observed on Tropicana Avenue in Las Vegas, the minimal sample size should be 41 trajectories based on the 0.04 penetration rate. If the signal coordination plans cover 8 hours a day (e.g., from 9 AM to 5 PM) and the arterial signal timing performance measurements focus on at least the two through-traffic directions along the arterial, 64 trajectory samples will be gathered according to the suggestion provided by this research, which is larger than the 0.04% penetration rate.

For daily performance monitoring, this research suggests that at least five trajectories per route need to be collected every three months. As one study [71] suggested a trajectory penetration rate of 0.1% that may be able to provide insights into the cyclic traffic variance and changing trends in traffic flow as the trajectory data can be stacked over multiple days and months. Given 101,000 AADT as one of the highest arterial AADTs observed in Nevada, performance monitoring should be conducted on the basis of 101 trajectory samples. Based on the suggestion provided by this research, the data of 120 trajectories samples will be collected during the three years, which is qualified to be used for performance monitoring purposes.

4.3 Data Processing

Any trajectories collected within the timing plan switching periods should be excluded because most signals are forced into transition during these time intervals.

In addition, caution should be exercised when applying the methodology for oversaturated conditions. Once oversaturation is observed at an intersection, the intersection can become a bottleneck leading to the queue backing up endlessly. In this regard, the signal timing performance evaluation is meaningless for the upstream segment. But the downstream traffic operations beyond the bottleneck will not be affected; hence, the performance measurement is still needed. Oversaturation can be detected by GPS trajectories as it occurs whenever at least two trajectory samples collected within 30 minutes have two or more stops on the same approach to an intersection. Figure 4.2 exhibits two trajectories that were recorded during a time period when oversaturation occurred. The two trajectories were collected 17 minutes apart, and both the travel runs made more than two stops at the same intersection. Therefore, the oversaturation was detected.

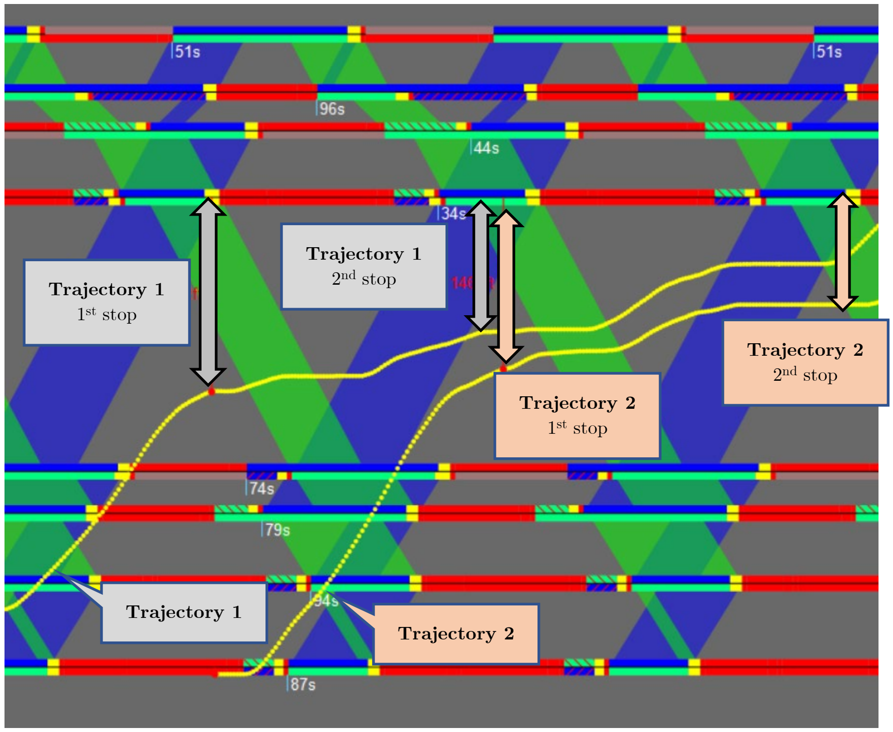


Figure 4.2 Oversaturation Identified through Trajectory Views

4.4 Examination Prior to Implementation

The proposed performance measurement should be implemented under appliable conditions. Accordingly, an examination regarding the background conditions is needed prior to the implementation.

The implementation of the proposed performance measurement requires conditions such as well-maintained detection and communication, which are summarized below:

1) Detection – the detectors should function well at the evaluated intersections, which can be verified through field observations or the signal management

software report focusing on whether signal actuation can be realized accurately and efficiently.

2) Communication – traffic signal coordination requires coordinated signals to be interconnected. If communication is lost in the field, the progression will be influenced most likely.
3) Controller operations – the signal controllers should function well at the evaluated intersections, which need to run the coordination plans or be capable of adaption.
4) Cycle length design – the cycle length should be appropriately selected, which can be validated through cycle failures (refers that all vehicles are not served within a cycle). It should be noted that cycle failures may sometimes occur due to traffic flow fluctuation.
5) Phase splits design – the phase splits should be allocated appropriately in order to serve the traffic demands at the evaluated intersections. This can be verified through phase failures (refers to the condition that one or more queuing vehicles cannot proceed through the evaluated intersection on a green indication). It should be noted that phase failures may sometimes occur due to traffic flow fluctuation.
6) Signal transition – the methodology implementation, especially the data collection, should not be performed during signal transition periods. The signal transition period occurs when most signals are in transition. The cases where some signals are forced into transition due to preemption calls or pedestrian services requests should not be excluded.

7) Traffic incidents – the data collection should not be conducted when traffic incidents are observed. These traffic incidents refer to the incidents that can measurably affect arterial traffic operations such as crashes and lane closures.

Figure 4.3 presents the workflow for performing the examination.

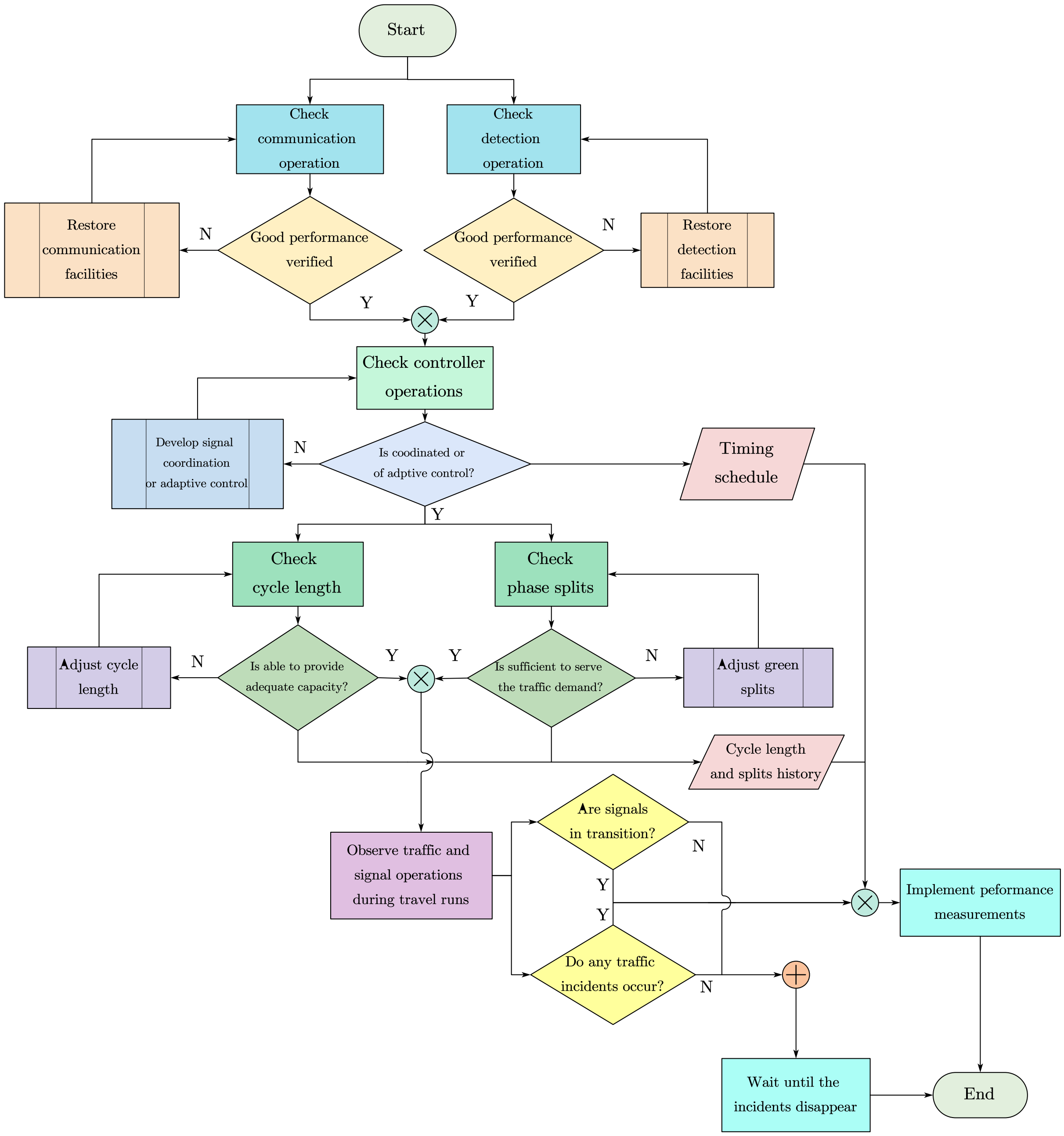


Figure 4.3 Workflow of the Examination Prior to the Performance Measurement Implementation

4.5 Summary

This chapter described the implementation of the proposed signal timing performance measurement. GPS trajectory data was studied as the data source used by the proposed methodology. The GPS data could be obtained from probe vehicle investigations and the third-party databases; however, the data should fulfill the requirements of minimal ping frequency and minimal precision of coordinates or speed. The data processing was discussed then regarding signal transition and oversaturation. An examination prior to the implementation was presented finally, highlighting the significant considerations such as detection and occurrence of traffic incidents.

Chapter 5 CASE STUDIES

5.1 Case Study 1 – RTC Washoe Signal Timing Phase 5 Project

During the years of 2017-2020, the Regional Transportation Commission of Washoe County (RTC Washoe) worked with the City of Reno and the City of Sparks to re-time 409 traffic signals in the Reno-Sparks metropolitan area. A total of 70 signalized arterials and collectors were re-timed during the project, as indicated in Figure 5.1.

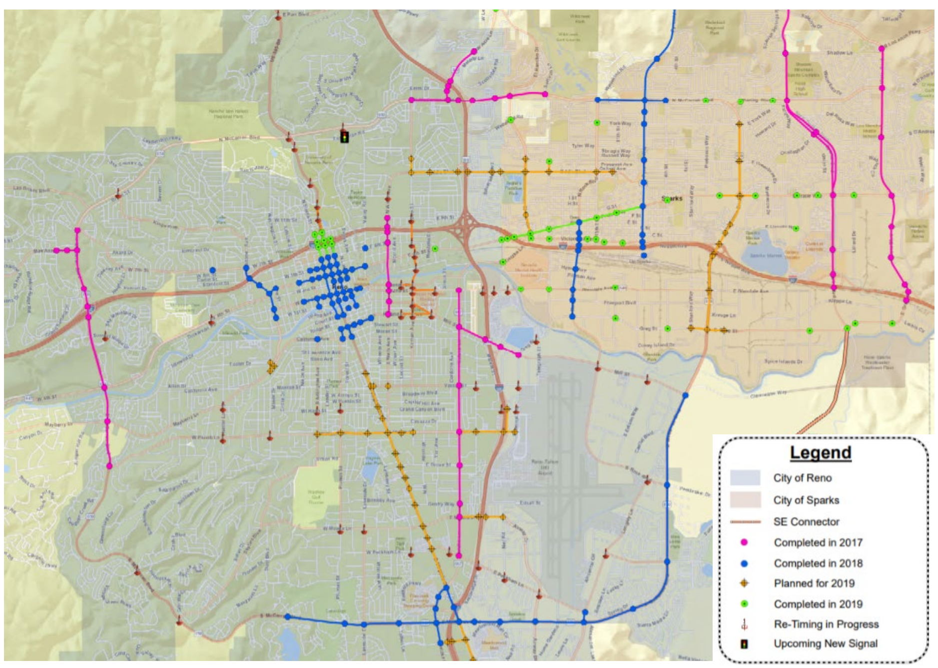


Figure 5.1 Scope of RTC Washoe Signal Timing 5 [72] (updated as of May 2019)

GPS trajectories were gathered through probe vehicle investigations along some of the re-timed arterials. The data collections were regularly conducted during the project period. Table 5.1 shows a group of GPS trajectories collected for the southbound (SB) and northbound (NB) through movements along Sparks Boulevard for the AM peak signal timing plan.

Table 5.1 GPS Data Collected along Sparks Boulevard for the AM Peak Period

Travel-run Trajectories					
	Run #	Time @ Trip End	Average Speed	No. Stops	Travel Time(sec)
SB	1	6:55:15 AM	30.5	2	426
	2	7:01:21 AM	33.8	1	379
	3	7:15:29 AM	24.1	3	537
	4	7:31:23 AM	22	4	584
	5	7:46:10 AM	19.9	6	626
	6	8:01:35 AM	27.7	2	465
	7	8:06:35 AM	36.2	2	349
NB	1	6:44:21 AM	29.3	4	444
	2	6:44:29 AM	35	1	371
	3	7:02:59 AM	35.4	1	367
	4	7:13:50 AM	31.5	3	379
	5	7:48:39 AM	28.3	4	457
	6	7:56:20 AM	25.4	4	493

Based on the gathered GPS trajectory data, the proposed performance measurements were conducted to evaluate the quality improvements for five arterials – 1) Sparks Boulevard, 2) Vista Boulevard, 3) Pyramid Highway, 4) North McCarran Boulevard, and 5) West McCarran Boulevard. Table 5.2 presents the performance data, including average speed and the numbers of stops, as well as the resulting AIP Scores, the AUS Scores, the route-based performance grades, and the overall arterial qualities of signal timing using the proposed method.

Table 5.2 Performance Measurement Results for Five Arterials in Reno and Sparks

Arterial	Plan	Route	Before Speed (mph)	After Speed (mph)	Before No. of Stops	After No. of Stops	Before AIP Score	After AIP Score	Before AUS Score	After AUS Score	Before Route Perfor-mance Grade	After Route Perfor-mance Grade	Before Quality of Arterial Signal Timing	After Quality of Arterial Signal Timing
Sparks Blvd	AM-1	NB	30.8	34	3	1.7	86	94	62	91	C	A	D	A
		SB	27.7	33.9	2.9	1	77	94	65	100	D	A		
	AM-2	NB	25.7	38.4	3	0.6	71	100	64	100	D	A	C	A
		SB	33.3	35.8	1.7	1	93	95	95	100	A	A		
	MD	NB	29	34.1	3	1.3	81	95	68	97	C	A	C	A
		SB	28.4	36.7	1.3	0.7	79	100	95	100	B	A		
	PM	NB	25.7	31.8	3.4	1.7	71	88	58	92	F	A	F	B
		SB	27.7	28.4	3.4	3.1	77	79	56	64	F	D		
Vista Blvd	AM	NB	28.4	32.8	2	1.25	84	96	89	100	B	A	C	B
		SB	24.6	24	2.75	2.69	72	71	68	71	D	C		
	MD	NB	28.1	32.3	1.98	1.25	78	89	88	100	B	A	B	A
		SB	25.4	29.3	2.25	1.38	71	81	86	97	B	A		
	PM	NB	22.1	32.1	4	1.13	65	94	50	100	F	A	F	A
		SB	22.1	28.3	4.1	1.89	65	83	50	92	F	A		
Pyramid Hwy	AM	NB	31.5	46.8	2	0.5	66	99	50	98	F	A	D	A
		SB	44.9	48.7	1	0.33	82	89	87	100	B	A		
	MD	NB	43.2	47.1	1	0.25	79	86	88	100	B	A	B	A
		SB	47.6	48.1	0.6	0.25	86	87	95	100	A	A		
	PM	NB	35	43.2	1	0.33	78	95	86	100	B	A	C	A
		SB	31.6	38.3	2.6	1	70	84	50	88	F	B		

Table 5.2 Performance Measurement Results for Five Arterials in Reno and Sparks (continued)

N. McCarran Blvd	AM	NB	33.9	33.4	0	0.67	89	88	100	100	A	A	C	A
		SB	16.8	26.3	3	1.4	58	86	50	87	F	B		
	MD	NB	24.7	31.3	1	0.5	70	89	86	100	B	A	B	A
		SB	24.8	31.2	1.5	0.83	71	89	90	98	B	A		
	PM	NB	18.7	23.7	2	1	58	76	78	95	F	B	C	B
		SB	23.9	24.4	1.2	1.4	77	79	92	88	B	B		
W. McCarran Blvd	AM	NB	40.4	40.8	0.7	0.3	100	100	91	95	A	A	A	A
		SB	37.1	40.4	1.3	0.2	93	100	86	97	A	A		
	PM	NB	28.8	35	2.39	1.5	72	88	77	88	C	B	C	B
		SB	26.9	31	2.45	1.8	67	80	74	86	D	B		

* ***Sparks Blvd*** *– number of signals: 8; arterial segment length: 18,972 feet (3.6 miles); speed limit: 40 mph*

Vista Blvd *– number of signals: 9; arterial segment length: 14,479 feet (2.7 miles); speed limit: 40 mph*

Pyramid Hwy *– number of signals: 4; arterial segment length: 12,278 feet (2.3 miles); speed limit: 55 mph*

N. McCarran Blvd *– number of signals: 7; arterial segment length: 7,484 feet (1.4 miles); speed limit: 55 mph*

W. McCarran Blvd *– number of signals: 8; arterial segment length: 13,298 feet (2.5 miles); speed limit: 50 mph*

As shown in Table 5.2, after the signals were re-timed, the arterial performance measures were significantly improved. Meanwhile, the qualities of arterial signal timing were rated at level A or level B, which demonstrated the effectiveness of the proposed performance measurement methodology gauging the project effort and performance improvement.

In addition, some agencies adopted the HCM LOS methodology to evaluate signal timing performance, which is exhibited in Table 5.3.

Figure 5.3 LOS Criteria established for the automobile mode on urban streets [42]

Travel Speed as a Percentage of Base Free-Flow Speed (%)	LOS by Critical Volume-to-capacity Ratio*	
	<1.0	>1.0
> 85	A	F
> 64-85	B	F
> 50-67	C	F
>40-50	D	F
>30-40	E	F
<30	F	F

* *The critical volume-to-capacity (V/C) ratio is based on consideration of the through movement volume-to-capacity (V/C) ratio at each boundary intersection in the subject direction of travel. The critical volume-to-capacity (V/C) ratio is the largest ratio of those considered.*

The proposed performance measurement methodology could be more suitable than the HCM LOS methodology when used for signal timing evaluation purposes, which can be shown by the case of W. McCarran Blvd, PM timing plan, the southbound (SB) route. During the PM time-of-day period, the southbound traffic operation along W. McCarran Blvd was near-saturated. According to the HCM arterial LOS methodology, the level of service along this route was only level D (31 mph = 48% of the free-flow speed, 55 mph) even if the arterial signal timing was significantly improved. Conversely, the quality of arterial signal timing was rated as level B (AIP score: 80 and AUS score: 86, which was showing a performance close to level A) using the proposed methodology.

The signal re-timing for W. McCarran Blvd reduced the measurable stopped delay, as indicated in Figure 5.2. Figure 5.2 presents a comparison of the travel delays portrayed on the Google map during the same time-of-day period between the before signal re-timing (imaged at 5:00 PM, Wednesday, April 19th, 2017) and the after signal re-timing (imaged at 5:00 PM, Wednesday, August 23th, 2017). Road segments colored in orange or red indicate different levels of congestion.

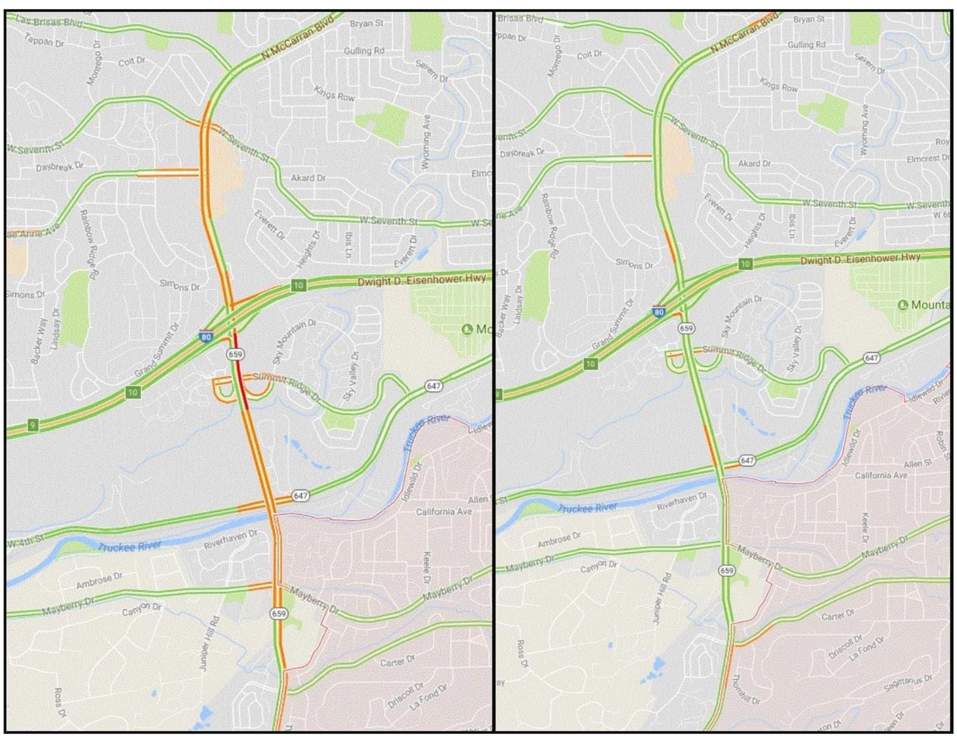


Figure 5.2 Travel Delay Improvements Shown on Google Map by Signal Re-timing

5.2 Case Study 2 – Carson Street Signal Re-timing Project

Eight signals in Carson City, Nevada along South Carson Street between Koontz Lane and Mica Drive were re-timed in 2019, as exhibited in Figure 5.3. The evaluated arterial segment is 3.1 miles long, and the posted speed limit is 50 mph.

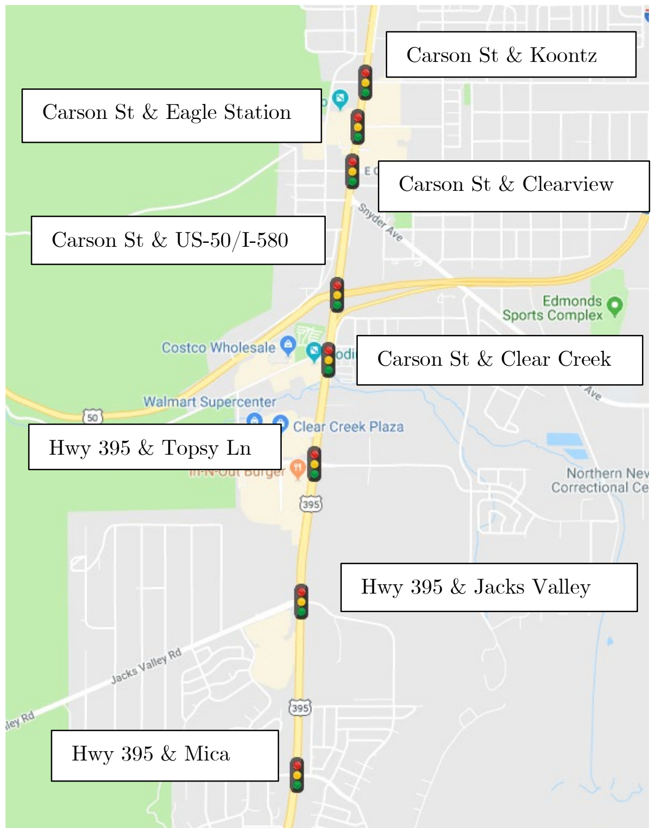


Figure 5.3 Evaluated Signals along Carson Street in Carson City, Nevada

For the performance measurement, a total of 64 travel-run trajectories were collected through probe vehicle investigations. A smartphone APP [70] was used to record the travel-run trajectories. The trajectory data were gathered in 12 days, including 9 weekdays on Tuesdays, Wednesdays, and Thursdays, as well as 3 weekend days on Saturdays and Sunday. The data were proportionally collected for four different time-of-day periods, as presented in Table 5.4.

Table 5.4 GPS Data Gathered during Time-of-day Periods

Time-of-day Periods	Number of Gathered Trajectories	Time Interval
Weekday AM	18	6:15 AM – 9:00 AM
Weekday MD	24	9:00 AM – 4:00 PM; 5:45 PM – 8:00 PM
Weekday PM	26	4:00 PM – 5:45 PM
Weekend Daytime	18	8:30 PM – 6:30 PM

Besides the data collected through floating car investigations, third-party probe vehicle data (PVD) were used as well. The data were in the format shown in Table 5.5.

Table 5.5 Third-party Trajectory Data Format (An Example of Trajectory 85207)

journeyId	capturedTimestamp	latitude	longitude
85207	2019-08-15T14:20:16.000-0700	39.145619	-119.767691
85207	2019-08-15T14:20:22.000-0700	39.144531	-119.767893
85207	2019-08-15T14:20:25.000-0700	39.143993	-119.768003
85207	2019-08-15T14:20:28.000-0700	39.143436	-119.768106
85207	2019-08-15T14:20:31.000-0700	39.142859	-119.768212
85207	2019-08-15T14:20:34.000-0700	39.142285	-119.768314

The acquired data had a ping resolution mostly higher than once per three seconds, and the precision of the coordinates fulfilled the requirement (6 decimal places). Therefore, the data source was qualified to be used for measuring the quality of arterial signal timing. Figures 5.4 and 5.5 show the vehicle speed variance chart and the trajectory views extracted from the third-party probe vehicle data.

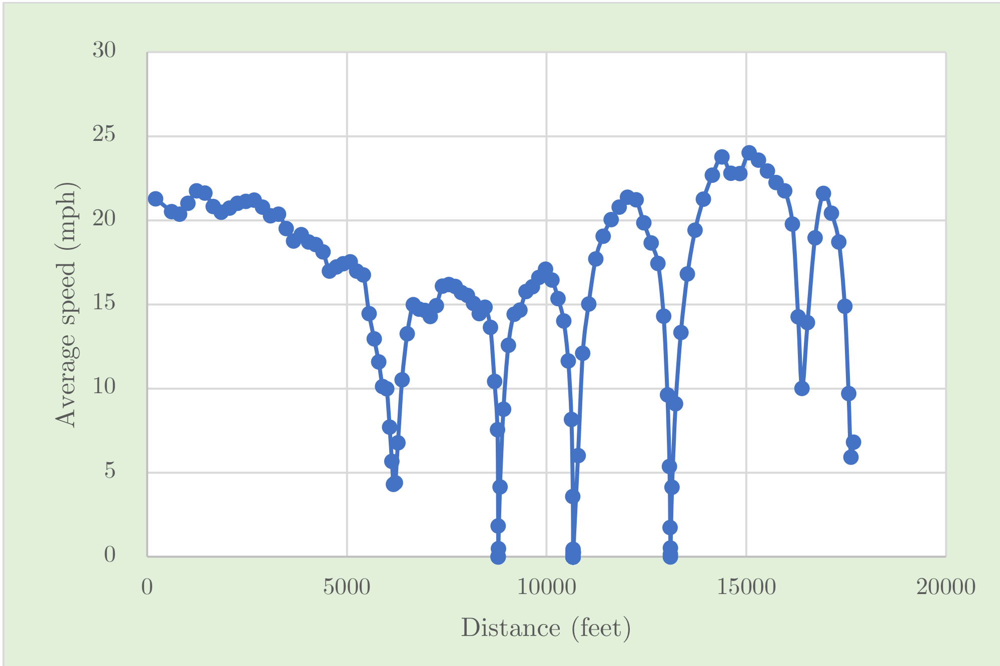


Figure 5.4 Speed Variance Shown by PVD

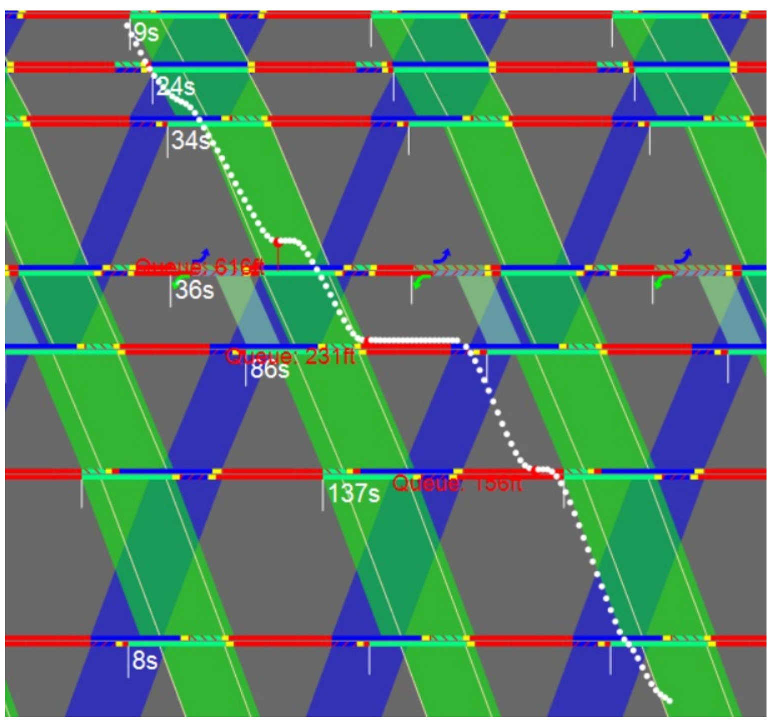


Figure 5.5 Trajectory View Obtained through PVD

Combining the data gathered from probe vehicle investigations and acquired from the third-party, a total of 146 travel-run trajectories were obtained.

The travel run covered three major routes of interest: 1) the northbound arterial through movement, 2) the southbound arterial through movement, and 3) the freeway movement left-turning onto Carson Street at the intersection of Carson St. & US-50/I-580 and going south, which are illustrated in Figure 5.6.

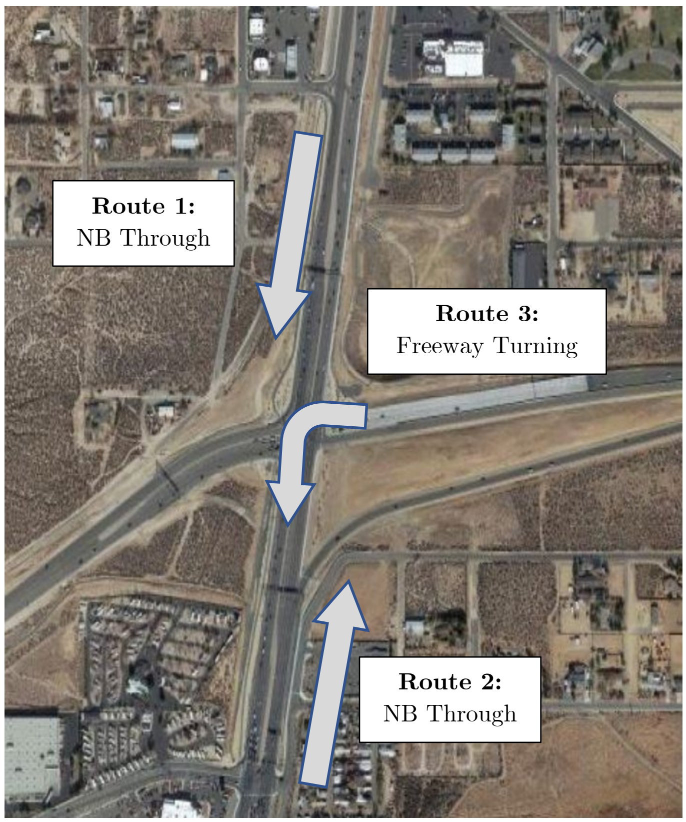


Figure 5.6 Three Evaluated Routes

Table 5.6 presents a comparison between the before signal re-timing and the after signal re-timing regarding the resulting travel times and the numbers of stops during the AM, MD, and PM time-of-day periods.

Table 5.6 Performance Data for the Before and After Timing Plans

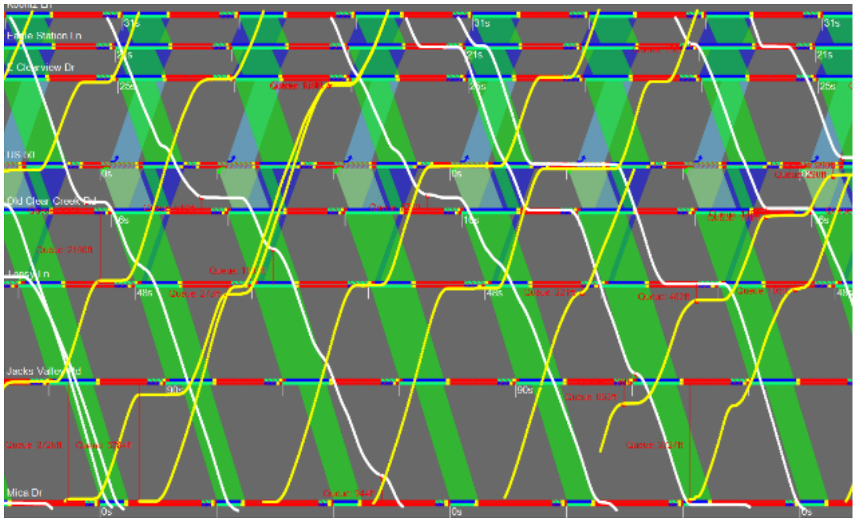
		Previous (Before) Timings		**New (After) Timings**			
		Travel Time (minute)	No. of Stops	Travel Time (minute)	Improved by	No. of Stops	Reduced Stops
AM	*NB*	8.61	3.7	5.02	41.7%	0.8	2.9
	SB	6.12	2.6	4.53	26.0%	0.4	2.2
	Freeway	2.72	0.3	2.88	-5.9%	0.5	-0.2
MD	*NB*	6.27	2.8	4.89	22.0%	0.3	2.5
	SB	6.13	2.5	4.61	24.8%	0.5	2.0
	Freeway	2.61	0.1	2.96	-13.4%	0.15	-0.1
PM	*NB*	9.67	4.2	5.58	42.3%	1	3.2
	SB	12.85	5.6	7.51	41.6%	1.6	4.0
	Freeway	6.25	1.2	6.83	-9.3%	1.9	-0.7

The newly developed arterial signal timings reduced the travel times and the numbers of stops along the NB and SB arterial through movements. However, the performances for the freeway route slightly decreased after the signal re-timing. This is because the previous timings were designed in favor of the freeway traffic, but the new timings were mainly to coordinate the arterial through traffic while maintaining some progression opportunities for the freeway traffic. By considering the trajectory data collected along the freeway route, the proposed performance measurement was able to accurately rate the quality of arterial signal timing under this circumstance. Table 5.7 presents a comparison of the resulting levels of quality of arterial signal timing with or without considering the freeway route.

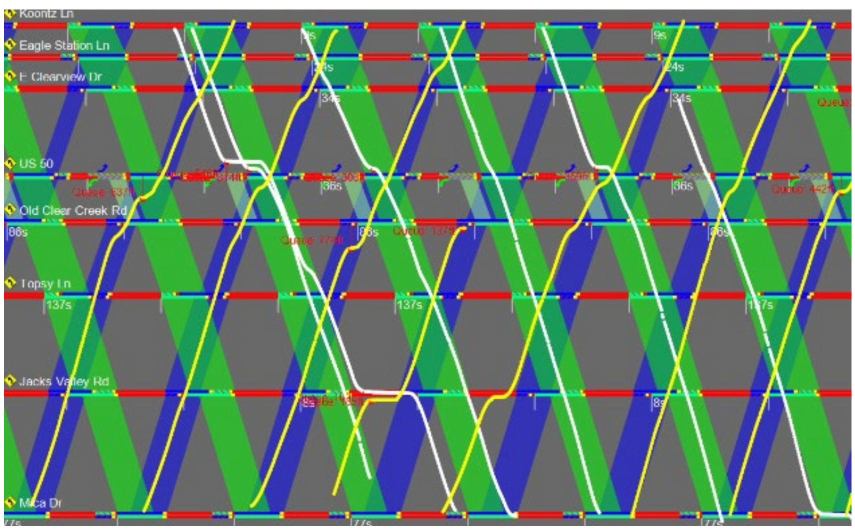
Table 5.7 Quality of Signal Timing with/without Considering Freeway Route

	Before Timings (Without Considering Freeway Route)	New Timings (Without Considering Freeway Route)	Before Timings (Considering Freeway Route)	New Timings (Considering Freeway Route)
AM	D	A	C	A
MD	F	A	D	A
PM	F	A	C	B

Figure 5.7 presents the trajectory views on the Time-Space Diagrams. Figure 5.7 (a) is the before timing which was rated at Level C the quality of arterial signal timing, and Figure 5.7 (b) is the after timing which was rated at Level A. Through intuitively visual analysis, it can be found that the trajectories shown in Figure 5.7 (a) were frequently halted at the intersections, whereas the trajectories shown in Figure 5.7 (b) were mostly straight indicating traffic flows were smoothly moving along the arterial. This figure is informative for developing signal timing improvements, showing how proposed timing plan adjustments can improve trajectory runs.



(a)



(b)

Figure 5.7 Trajectories Views on Time-Space Diagrams (a: Before Timing – Quality of Signal Timing: C; b: After Timing – Quality of Signal Timing: A)

Figure 5.8 displays six trajectories and associated performance grades, providing a visualization of the relationship between trajectories and quality levels. These trajectories presented in Figure 5.8 were collected for one signal timing plan along the northbound through route. Even if the travel-run trajectories were collected under the same operational conditions, the resulting quality levels of arterial signal timing could be different. The trajectory i represents that a vehicle was moving along the NB arterial through route and traversed the first signal (the bottom signal shown in Figure 5.8) by the end of the green indication. This travel run only experienced one 15-second stop and was rated as Level A performance grade. The trajectory ii was collected by a vehicle moving along the NB arterial through route but traversed the first intersection by the start of the green indication. The travel run was stopped twice for 38 seconds and 31 seconds, respectively, which was rated at Level C performance grade. The trajectory iii, iv, and v were gathered by vehicles that traversed the first signal in the middle of the green indication and rated at Level B performance grade. The trajectory vi was collected by a vehicle that turned onto the arterial at the first signal and then moved along the NB arterial through route. The travel run was stopped three times for 53 seconds, 23 seconds, and 11 seconds, respectively, and rated at Level D performance grade. Determining the overall performance grade for the northbound arterial through route should comprehensively consider these trajectories, which indicated different levels of travel-run performance.

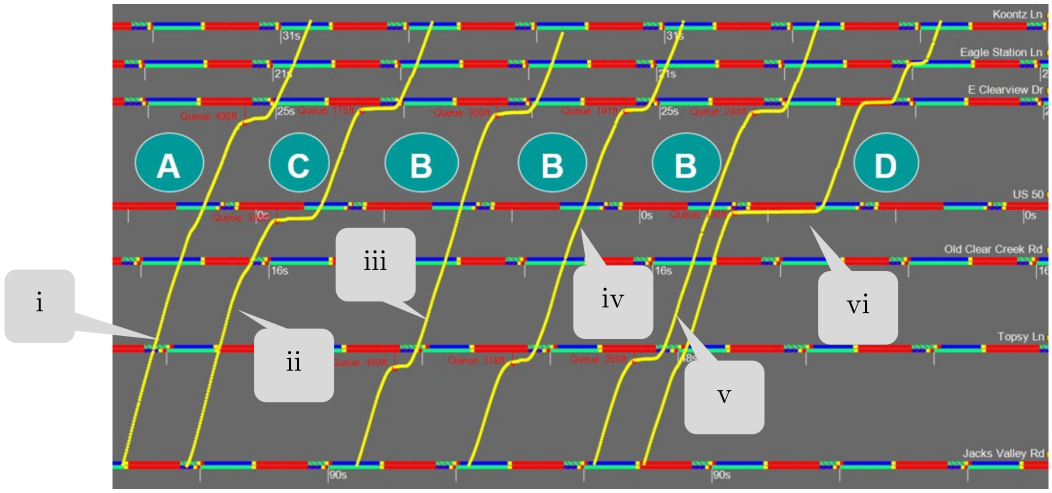


Figure 5.8 Trajectory Shapes Rated at Various Performance Grades

5.3 Summary

This chapter included two case studies based on real-world signal timing projects where the proposed performance measurements were implemented.

The first case study was based on the Washoe RTC Signal Timing Phase 5 Project. Five signalized arterials were evaluated, and the results demonstrated that the proposed performance measurement methodology could be used to adequately rate the quality of arterial signal timing and reflect the effectiveness of the signal re-timing efforts. In addition, according to a near-saturated case observed along the W. McCarran Blvd southbound through movement, the differences between the proposed performance measurement methodology and the HCM arterial LOS methodology were revealed. It indicated that the proposed methodology could be more accurate for signal timing performance measurement purposes.

The second case study was based on the Carson Street Signal Re-timing Project. Besides the trajectory data collected using the mobile devices, third-party probe vehicle data were also analyzed and adopted for the evaluations. The signal re-timing project involved three major routes of interest, including an additional freeway turning movement, which was different from the conventional signal timing practices that mainly focus on arterial through movements. The proposed performance measurements were conducted under two scenarios – 1) considering the freeway turning movement and 2) without considering the freeway turning movement. The evaluation results showed that the proposed performance measurements could accurately indicate the quality of arterial signal timing under such conditions, which showed that the proposed method could be used according to various signal timing considerations such as progressing additional traffic routes or prioritizing specific traffic routes due to bicycle traffic or transit traffic.

Trajectory views on the Time-Space Diagrams and various trajectory shapes were additionally presented, based on which, the quality of arterial signal timing obtained using the proposed methodology could be visually verified.

Chapter 6 SUMMARY AND FUTURE RESEARCH

6.1 Research Summary

In this research, a method to measure the quality of arterial signal timing was developed, and the implementation of the proposed performance measurement was outlined.

The background introduction and a review of current research and practice were presented. This identified a need for research to develop an arterial-level signal timing performance measurement methodology. This research is especially valuable when vehicle trajectories can be broadly collected across the road network through emerging technologies such as connected vehicles. Compared to the conventional signal timing performance metrics that were derived from at-intersection detectors, the notion of arterial-level performance metrics was described along with the advantages of using continuous vehicle trajectories. Also, through investigations on the current signal timing performance measurement techniques, major contributions of this research can be summarized by: 1) the proposed performance measurement methodology can be applied without strict preconditions (e.g., specific infrastructure configurations) and at flexible costs, allowing agencies with limited budgets and staffing to conduct regular arterial signal timing performance measurements; 2) the proposed methodology can be easily understood by experienced signal timing practitioners, elected officials, and the public.

The two arterial-level performance metrics developed in this research based on the travel-run speed and stop characteristics are: 1) attainability of ideal progression (AIP) and 2) attainability of user satisfaction (AUS). In practice, average travel speed is commonly used to measure the arterial signal timing performances; however, this measure alone may not be proper as there could be several non-signal-timing factors that can influence average travel speed, e.g., arterial congestion level and arrival flow profile. Investigations were conducted to characterize the effects of these non-signal-timing factors, and the AIP metric was defined based on travel speed but excluded the non-signal-timing factors. The AUS metric was defined to describe the drivers' perceived quality of arterial signal timing, which correlated to the number of stops and stopped time at intersections. Preliminary surveys were conducted to identify some factors related to drivers' satisfaction while moving along an arterial.

A final performance measurement methodology was derived according to the AIP and AUS metrics, as well as a number of additional parameters. A new parameter, Intersection Classification, was proposed to simplify the calculations to exclude the effects of non-signal-timing factors as well as scaling the change of traveler satisfaction under the various circumstances. The performance measurement framework included the AIP scoring, the AUS scoring, and the scoring adjustments. The cycle length adjustment and intersection spacing adjustment were based on considerations of side-street and pedestrian delays as well as arterial geometric conditions. The quality of arterial signal timing can be rated at levels of A, B, C, D, and F. Such letter-based grades are intuitive and can greatly facilitate information exchange, recognizing re-timing needs, and monitoring the quality of regional arterial management programs.

The implementation of the proposed method was outlined, including data collection, data processing, and a pre-implementation examination. The required data resolution and sampling for GPS trajectories were described. GPS trajectory data can be obtained through two approaches – 1) conducting floating-car investigations using mobile GPS recording devices and 2) acquiring data from third-party data service companies. Hence, practitioners could select different data sources according to budgetary conditions and purposes, i.e., either ad-hoc performance studies or daily performance monitoring. Travel runs during signal transition or under oversaturated conditions must be carefully examined as including such travel run data could bias the performance results. Before conducting performance measurements, several conditions, such as detection maintenance and occurrence of traffic incidents, also need to be investigated.

Two case studies were documented lastly to prove the validity of the proposed performance measurement methodology. The first case study involved re-timing of five arterials as part of the RTC's regional signal re-timing project – phase 5. The second case study involved re-timing of 8 signals on Carson Street in Carson City, Nevada. Both case studies demonstrated that the proposed performance measurement could accurately gauge the quality of signal timing in the contexts of various background traffic volume conditions and signal timing considerations.

The proposed signal timing performance measurement method can improve the current signal timing practice by providing: 1) a scalable performance measurement framework which can be implemented under a wide range of budgetary conditions and for diverse signal timing considerations; 2) accurate performance evaluation results that can demonstrate the effectiveness of the signal re-timing efforts, assist practitioners in identifying signal re-timing needs, and facilitate signal timing improvements, especially when the arterials are congested; 3) an intuitive indicator that can be easily understood and adopted by decision-makers, practitioners, and the public, which can promote progress reporting, the development of expertise, and public involvement during signal timing projects.

6.2 Future Extensions

The proposed method can be enhanced in the following areas through future research:

1) Integration with controller event data – at present, GPS trajectories are collected mostly through manual processes; thus the sample size is limited (penetration rate $< 1\%$). Using the information extracted from automatically gathered controller event data (penetration rate at almost 100%) will significantly supplement the trajectory data sampling.
2) Conducting additional surveys on travelers' perceptions – the preliminary findings documented in this dissertation may be arbitrary and biased as the surveyed samples were informal and very limited. Further investigations that incorporate more formal and broader surveys should be performed. In

addition, the question of why drivers make a complaint phone call was still not answered. Future collaborative studies are needed with local agency engineers and community representatives.

3) The proposed methodology involved many coefficients that need to be calibrated using real-world data. The recommendations given by this research were developed based on the data collected in urban areas in Nevada. Additional data collected elsewhere across the nation are desirable to further improve and validate the proposed methodology.

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APPENDIX: SURVEY QUESTIONS

Traveler Satisfaction Survey

This is a traveler satisfaction survey on the quality of arterial signal timing. In this survey, you will be asked questions about your attitudes and feelings under different circumstances while driving along an urban arterial. Please fill out the form, and thank you for your time!

Next

Part I: Basic Information

Gender

- Male
- Female
- Prefer not to say

Age

- < 20
- 20 - 60
- > 60

Living Area

- ○ Urban
- ○ Suburban
- ○ Rural

Usage of Signalized Arterials

- ○ Frequent
- ○ Not Frequent
- ○ I Don't Know

Part II: Driving Experience

Do you think your experience can be influenced according to how long you are stopped by signals?

- ○ Yes
- ○ No

Which one you are most concerned about while driving along an arterial?

- ○ Number of stops that I have made per mile
- ○ Number of intersections that I have passed without any stops

When do you start to be aware of a stop at the intersection?

- ○ Stopped more 5 seconds
- ○ Stopped more 10 seconds
- ○ Stopped more 15 seconds
- ○ Stopped more 20 seconds

When do you start to feel dissatisfied while being stopped by signals

- ○ Stopped more 15 seconds
- ○ Stopped more 20 seconds
- ○ Stopped more 30seconds
- ○ Stopped more 40 seconds

Do you agree that making two short stops (e.g., two 20-second stops) is better than making one long stop (e.g., one 40-second stop)?

- ○ Yes
- ○ No
- ○ Maybe

Do you agree making two stops in a short distance and a short time interval is dangerous and annoying?

- ○ Yes
- ○ No
- ○ Maybe

Rate your satisfaction degree

When you are stopped more 10 seconds at a big and busy intersection

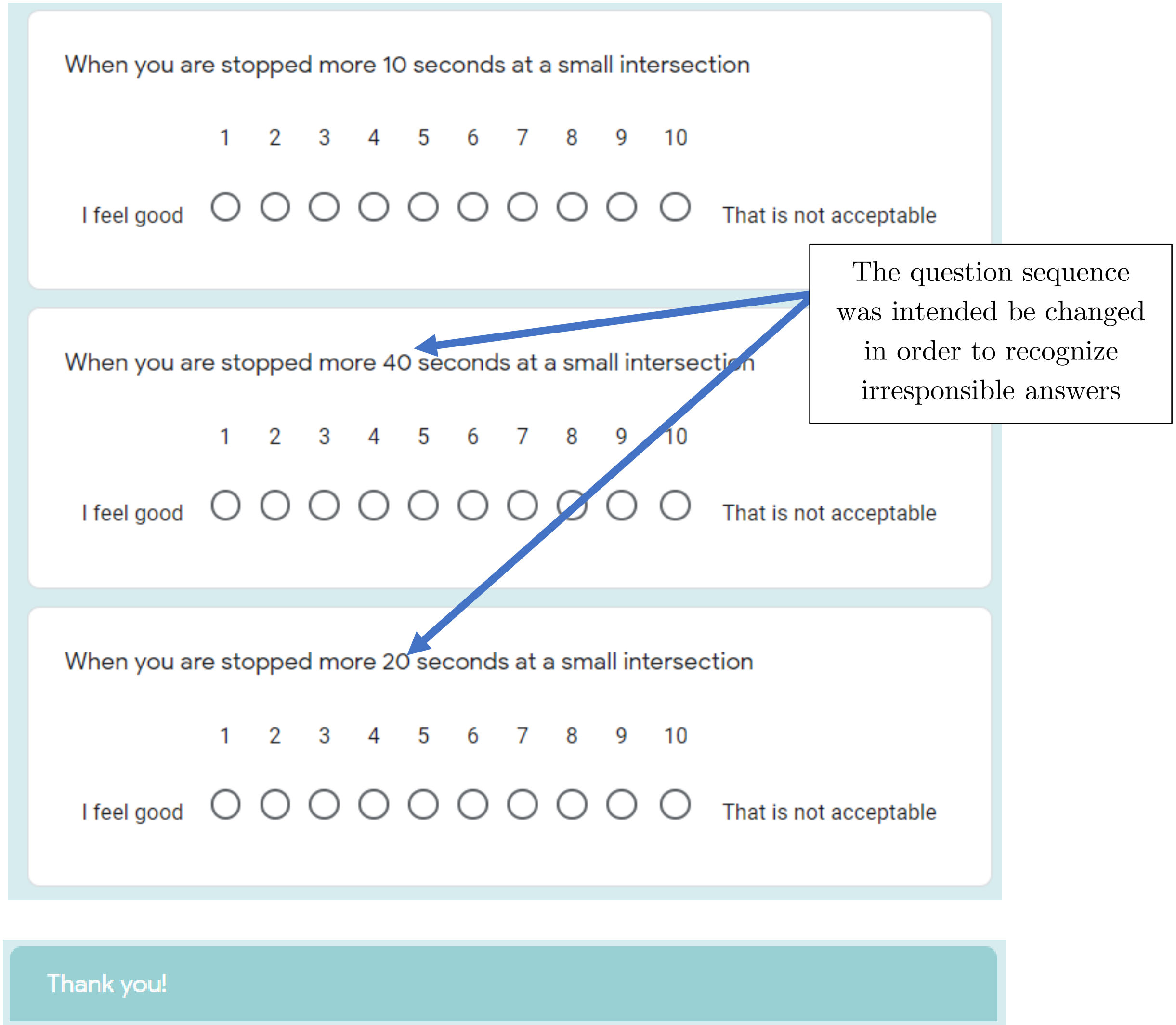
	1	2	3	4	5	6	7	8	9	10	
I feel good	○	○	○	○	○	○	○	○	○	○	That is not acceptable

When you are stopped more 20 seconds at a big and busy intersection

	1	2	3	4	5	6	7	8	9	10	
I feel good	○	○	○	○	○	○	○	○	○	○	That is not acceptable

When you are stopped more 40 seconds at a big and busy intersection

	1	2	3	4	5	6	7	8	9	10	
I feel good	○	○	○	○	○	○	○	○	○	○	That is not acceptable


When you are stopped more 10 seconds at a small intersection

	1	2	3	4	5	6	7	8	9	10	
I feel good	○	○	○	○	○	○	○	○	○	○	That is not acceptable

When you are stopped more 40 seconds at a small intersection

	1	2	3	4	5	6	7	8	9	10	
I feel good	○	○	○	○	○	○	○	○	○	○	That is not acceptable

When you are stopped more 20 seconds at a small intersection

	1	2	3	4	5	6	7	8	9	10	
I feel good	○	○	○	○	○	○	○	○	○	○	That is not acceptable

Thank you!