TRAVEL TIME ESTIMATION FOR SIGNALIZED ARTERIALS USING PROBABILISTIC MODELING

By

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by

Xiao Cui
To all who supported and helped me through the years in my research and my life
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LIST OF ABBREVIATIONS

$ATT$  Arterial Travel Time, total travel time on the arterial

$T_M$  Time in Motion, time period when the vehicles travel before they join the queue

$T_Q$  Time in Queue, time period from when the vehicles join the queue to when they leave the queue

$WT_Q$  Waiting Time in the Queue, time that vehicles have to wait in the queue

$MT_Q$  Moving Time in the Queue, time that vehicles move when they are in the queue

$w_{AB}$  Shockwave speed between state A and B

$w_{BC}$  Shockwave speed between state B and C

$w_{AC}$  Shockwave speed between state A and C

$q_A$  Arrival rate (actual flow rate)

$q_C$  At capacity flow rate

$k_A$  Density at arrival rate, can be converted from flow divided by speed

$k_B$  Jam density

$k_C$  Density at capacity

$C$  Cycle length

$r$  Effective red time

$s$  Total length of the link (exclude the length of intersection)

$s_A$  Accelerating distance

$s_C$  Constant speed distance

$s_D$  The distance from the end point of $S_2$ till the end of the link

$D_q$  Distance over which the queue extends
$V_{i\_link1}$  Vehicles’ entering speed (initial speed) when they enter link 1

$V_m$  Vehicles’ maximum operating speed when they travel on the link

$a_a$  Vehicles’ acceleration rate

$a_d$  Vehicles’ deceleration rate

$D_{q\_link1}$  Distance over which the queue extends at link 1

$V_{d\_link1}$  Discharging speed at link 1

$\text{avg.} D_{q\_link1}$  Average distance over the queue extends at link 1

$Q_M$  Maximum queue length

$V_{f\_link0}$  Leaving speed (final speed) for link 0

$D_{q\_link0}$  Distance over which the queue extends at link 0

$V_{d\_link0}$  The discharging speed from link 0

$Q_R$  The residual queue

$Q_M$  The maximum queue

$\text{Diff}_{\text{cond 2}}$  The difference between the completed Condition 2 and the terminated Condition 2

$f_{i\_link1}$  Entering (initial) flow rate at link 1

$f_{f\_link0}$  Final flow rate at link 0

$f_{int1\_WR}$  Flow rate at intersection 1 from westbound right turning movement

$f_{int1\_WL}$  Flow rate at intersection 1 from westbound left turning movement

$f_{int1\_NR}$  Flow rate at intersection 1 from southbound right turning movement

$f_{int1\_SL}$  Flow rate at intersection 1 from northbound left turning movement
$f_{i\_link2}$  Entering (initial) flow rate at link 2

$f_{f\_link1}$  Final flow rate at link 1

$f_{\text{int}_2\_WR}$  Flow rate at intersection 2 from westbound right turning movement

$f_{\text{int}_2\_WL}$  Flow rate at intersection 2 from westbound left turning movement

$f_{\text{int}_2\_NR}$  Flow rate at intersection 2 from southbound right turning movement

$f_{\text{int}_2\_SL}$  Flow rate at intersection 2 from northbound left turning movement
Travel time is an important performance measure for many transportation facilities in the sense that it is essential information in establishing Advanced Traveler Information System (ATIS), it is easy to communicate with travelers, and it can be used by operators to evaluate the network performance. However, most of travel time studies focus on freeways; there are only a few studies on arterial travel time. The objective of this research is to develop a model for estimating travel time for signalized arterials using probabilistic modeling. The dissertation reviews the literature related to arterial travel time and presents the methodology developed for estimating arterial travel time. In the methodology, travel time is defined as the sum of time spent in motion and time spent in queue. Time spent in queue can be further divided into moving time in queue and waiting time in queue. Vehicles are assumed to arrive at an intersection with three different conditions according to different traffic signal status; each condition has a certain probability of occurrence. The method estimates the travel time as a function of the signal status, and then considers all possible options to estimate the expected arterial travel time. To validate this model, field travel times were collected at 4 sites from State College, PA, and Gainesville, FL, during several time periods. There are a total of 14 cases with each of them representing a certain time period in a certain site. The analytical model was applied and validated and...
sensitivity analysis was performed for each case to test how sensitive the analytical model is to several key factors. The analytical model provides travel time which is based on the green time, cycle length, link length, maximum operating speed, offset, acceleration/deceleration rate, and the entering flow rate at each intersection. The model does not consider delay caused by driveways, and it does not analyze actuated and semi-actuated signal timing. It also does not provide travel time estimation for O/D pairs other than those for the through arterial movements.
CHAPTER 1
INTRODUCTION

Arterials are major roadways, typically in urban areas, that serve through traffic and provide access to surrounding properties. Arterials can be one-way or two-way streets, and intersections along the arterial can be un-signalized (including four-way stop, two-way stop) or signalized.

The focus of this research is on travel times for the signalized arterial. Travel time estimation depends heavily on the signal settings. Thus, the types of signals are briefly discussed here: there are three types of signal control: pre-timed, actuated, and adaptive. In pre-timed control the green interval is fixed. There are two types of actuated control, one is fully-actuated and the other is semi-actuated. In the fully actuated, the green interval changes from cycle to cycle in accordance with traffic flow on both major and side streets, and there are detectors on the major and the minor streets which determine the traffic demand. In semi-actuated control, normally there are detectors on the side streets only, and the signal adjusts the green interval according to the demand on the major street only. The signal gives green time to side streets only when traffic is detected on the side streets. Like the pre-timed signal control, semi-actuated control allows for coordination along the arterial. Vehicles traveling on the arterial have to stop or travel through each intersection. This depends on the signal timing and the time when one particular vehicle arrives at the intersection. Therefore, vehicle travel time can be affected dramatically by signal timing.

1.1 Travel Time Estimation on Signalized Arterials

Travel time is very important since it is considered a promising performance measure in evaluating traffic operations on various transportation facilities. Travel time estimation is essential, especially for new facilities where travel time cannot be measured. While several
research projects such as the Freeway Travel Time Reliability project funded by the National Science Foundation (NSF) (Elefteriadou et al, 2004) have started or have already been completed with focus on freeway travel time, few of them focus on arterial travel time. The arterial, however, is an essential transportation facility and accurate travel time estimation is important.

Travel time is very important in establishing a traveler information system; accuracy of this information can be essential in delivering real time information to travelers and can affect travelers’ route choice. With accurate estimation and immediate information delivery by radio stations and Internet, travelers would have better information about current traffic conditions. This helps travelers avoid congested routes if there are any. If travelers do not enter the congested route, traffic congestion can be relieved faster, and overall delay in the whole traffic network can be reduced. Thus, travel time or speed has to be estimated accurately.

In literature such as the Highway Capacity Manual (HCM), travel time along arterials was estimated by adding the link travel time and the isolated intersection delay. This estimation however is not accurate since the link travel time is calculated by assuming a constant vehicle speed and instant speed changes. The intersection delay is typically estimated by an analytical model for isolated intersection delay, which does not consider the effect of upstream and downstream intersections. More recently, a few studies begin to estimate the arterial travel time more realistically. Among these studies, some of them tried to measure the travel time on arterials by using technologies such as loop detectors and probe vehicles. The drawbacks of the methods are that they are site-specific and most of them cannot be converted and applied at other locations. Thus, there is no theoretical model that can be applied generally to arterial facilities for predicting travel time accurately.
1.2 Objectives of the Research

The objective of this research is to develop an analytical model for estimating arterial travel time. Travel time is a variable which can be affected by many factors. Vehicles entering at different points in time can meet with different traffic demand and signal status, which in turn can affect travel time greatly. To take variability into consideration, first, traffic demand should be considered as one of the factors in the model. Second, the signal status should be known and be used in the estimation method. The tasks of this research are as follows:

- To develop an analytical model for estimating the expected arterial travel time, the variance of travel time, and travel time distribution based on probabilistic modeling. The probability of encountering a specific signal status can be found and applied in the model so that a certain travel time occurs with certain probability.

- To compare these estimates to field data. Simulation was also used to evaluate model estimations.
CHAPTER 2
LITERATURE REVIEW

In this literature review, papers and research reports related to different arterial travel time estimation methods were reviewed according to the types of methods. Analytical models and simulation methods were reviewed. In the review of analytical models, two types of models were reviewed. One is the composite analytical models based on intersection delay estimation or not. The other is arterial travel time models based on in-field detection technology. In the review of simulation methods, the delay models in simulation packages and the travel time estimation by simulation were reviewed. The studies on freeway travel time estimation are not reviewed in this chapter since their methodologies are not related to this research and do not contribute to the development of methodology for this research.

2.1 Models for Estimating Arterial Travel Time

Several models have been developed for estimating arterial travel time. Some of them do not need the inputs from in-field detection technologies and others do. Thus, in this section, these two types of models are discussed.

2.1.1. Composite Analytical Models

In the HCM 2000 Chapter 15; travel time on arterials is estimated by the following equations:

\[ T.T. = \frac{L}{S_A} \]  \hspace{1cm} (2-1)

\[ S_A = \frac{3600L}{T_R + d} \]  \hspace{1cm} (2-2)

Where
- \( T.T. \): travel time, sec
- \( L \): link length between two intersections, ft
- \( S_A \): average speed along the arterial, ft/s
- \( T_R \): running time, sec
- \( d \): control delay for through movements, sec
The basic method for estimating the link travel time in the HCM is to divide the link length by the average speed which is adjusted from the free flow speed. The average speed is calculated by using two terms: one is the running time which is the ideal travel time. The other is the control delay for the through movement encountered at intersections. The HCM uses the isolated intersection delay model to represent this control delay. It is a rough method for estimating arterial travel time; first, speed is changing over time, it cannot be accurately estimated by the average speed. Second, the isolated intersection control delay is not the actual control delay that vehicles encounter at intersections with consideration of upstream and downstream traffic flow impacts. However, the HCM method presents a popular approach for calculating the arterial travel time, which is to calculate the arterial travel time as a function of intersection delay, either control delay or stop delay.

**2.1.1.1 Intersection-delay based arterial travel time models**

Before discussing the travel time models based on intersection delay, the types of intersection delay and their definitions have to be defined first. There are two types of intersection delay that are often used: one is control delay; it is the time period when the vehicle decelerates, stops, and accelerates at an intersection. The other is the stop delay, which is the time when the vehicle completely stops at an intersection. In the intersection-delay based arterial travel time models, arterial travel time can be decomposed into two parts: vehicle’s link travel time and the control delay caused by intersections. Some previous research considered these two parts and tried to estimate them separately. Thus, intersection delay estimation is essential in determining travel time along the arterial. There are several methods for estimating signalized intersection delays: the HCM intersection delay model, shockwave analysis, and stochastic queuing analysis. These methods can provide different estimates for intersection delay and they are reviewed in this section.
Different delay models were developed over the years; these models are both for isolated intersections and for non-isolated intersections. This section presents some delay models and studies related to intersection delay.

Intersection delay estimation is a broad field and hundreds of studies have been completed since the 1950’s. Many delay models have been presented in the previous studies and are briefly reviewed here:

Rouphail, Tarko, and Li (1992) performed a study on traffic flow at signalized intersections. In this publication, they reviewed some of the previously developed delay models for signalized intersections.

For example, Rouphail, Tarko, and Li (1992) referred to the Webster’s delay equation. Webster developed a comprehensive delay model in 1958 (Equation 2-1) which assumes vehicle arrivals are random and departures are constant. This model is widely used today:

\[
d = \frac{C(1 - \lambda)^2}{2(1 - \lambda x)} + \frac{x^2}{2q(1 - x)} - 0.65\left(\frac{C}{q^2}\right)^{\frac{1}{3}} x^{(2+x)}
\]

Where
\[
\begin{align*}
d & : \text{stop delay, sec/veh} \\
C & : \text{cycle length, sec} \\
\lambda & : \text{green ratio,} \\
x & : \text{degree of saturation,} \\
q & : \text{flow rate, veh/hr}
\end{align*}
\]

These two terms are the theoretical part of the equation, and the last term is the empirical correction factor. Webster found that the correction factor contributes to 5 to 15 percent of delay.

After Webster, several delay equations were developed. Among them, according to Rouphail, Tarko, and Li (1992), Miller’s and Hutchinson’s delay model are often cited.

Thus, delay models were developed and modified over time to evaluate the traffic operations at intersections. Since 1985, the HCM delay calculation has been used as the standard
delay estimation for intersections; currently, the HCM 2000 chapter 16 presents the estimation method for isolated intersection delay. The models are based on Webster’s equation and have been modified in each version of the HCM as follows: In the 1985 HCM, an intersection delay model was presented (1985):

\[
d_s = 0.38 \frac{C(1-\lambda)^2}{(1-\lambda x)} + 173x^2 \left( (x-1) + \sqrt{(x-1)^2 + \frac{16x}{c}} \right)
\]

Where
\(d_s\): the average stop delay, sec/veh
\(C\), \(\lambda\), and \(x\) are defined as before
This is the first delay model for signalized intersections in the HCM; it estimates the stop delay at intersections for a 15-minute period. It was found that the 1985 HCM delay model can overestimate delay when the degree of saturation is high. The HCM in 1994 presented a new delay model:

\[
d_s = 0.38 \frac{C(1-\lambda)^2}{\{1-\lambda[\text{Min}(x,1.0)]\}} + 173x^2 \left( (x-1) + \sqrt{(x-1)^2 + \frac{mx}{c}} \right)
\]

Where
\(m\): an incremental calibration term representing the effect of arrival type and degree of platoon
\(d_s\), \(C\), \(\lambda\), and \(x\) are defined as before
The 1994 HCM model kept the form of the delay model from 1985; however, it changed the coefficient of arrival type and degree of platoon from a constant value to a variable, which needed to be calibrated for each specific case. Again, stop delay is selected as the performance measure for signalized intersection.

In 1997, the HCM presented a new delay model, which differs significantly from the previous ones. The 1997 HCM delay model (1997) is

\[
d = d_i \cdot PF + d_2 + d_s
\]

Where
\(d\): control delay, sec/veh
\[ d_1 : \text{incremental delay, sec/veh} \]
\[ d_2 : \text{initial delay, sec/veh} \]
\[ d_1 = \frac{0.50 \left[ 1 - \frac{g}{C} \right]^2}{\left[ 1 - \min \left( \frac{g}{C} \right) \right] X} \]  
\[ (2-7) \]

Where

\( g \): effective green for lane group, sec
\( C \): cycle length, sec
\( X \): degree of saturation

\[ d_2 = 900T \left[ (x-1) + \sqrt{(x-1)^2 + \frac{8klX}{cT}} \right] \]  
\[ (2-8) \]

\( T \): duration of analysis period, hr
\( X \): degree of saturation
\( k \): incremental delay factor
\( I \): upstream filtering/metering adjustment parameter
\( c \): lane group capacity, veh/hr

The 1997 HCM delay model presents an improvement in estimating signalized intersection delays. It defined delay as a summation of three forms of delay: uniform delay, incremental delay and initial delay, which is called control delay. Besides that, the coefficient changes from 0.38 to 0.50. The 2000 HCM presented the same delay model as that of the 1997 HCM.

The HCM delay models are used as the standard estimation of intersection delay. However, the model is not perfect in some aspects. For example, it is isolated intersection control delay which does not consider the upstream and the downstream effect. Thus, some studies tried to adjust or improve the HCM delay model.

Quiroga and Bullock (1999) used GPS data to measure intersection control delay and tried the measurement results as a new standard in evaluating the accuracy of new intersection delay models. With a GPS device, control delay can be calculated by

\[ D = \text{TravelTime} - \frac{\text{Link Length}}{S_f} \]  
\[ (2-9) \]

Where

\( D \): control delay, sec/veh
\( S_f \): free flow speed, ft/s
The paper compared the control delay measured in the field to the control delay calculated by the HCM method. The conclusion is that the deceleration and acceleration lengths were much longer than usually anticipated so that the part of the control delay that takes place in acceleration and deceleration is not negligible.

Benekohal and Kim (2005) indicated that for oversaturated conditions, the HCM 2000 model does not apply the progression adjustment factor (PF) when there is an initial queue. This causes the wrong estimation that delay with initial queue is sometimes less than delay without initial queue. Besides this, the model yields the same uniform delay values for all arrival types when there is an initial queue. This paper proposed a new model for estimating uniform delay for oversaturated conditions. The uniform delay is

\[
d_1 = \frac{1}{2s_g} \left[ Q_1 C + Q_2 (C - t_1) - q_0 C^2 - s g^2 \right]
\]

Where
- \( d_1 \): uniform delay, sec/veh
- \( s \): saturation flow rate, veh/hr/ln
- \( g \): effective green time, sec
- \( Q_1 \): the number of arrivals when queue increase rate changes for the first time
- \( Q_2 \): the number of arrivals at the end of cycle
- \( C \): cycle length, sec
- \( t_1 \): platoon duration time, sec
- \( q_0 \): overflow rate, veh/hr/ln

The authors concluded that the new model gives a better estimation of delay than the HCM 2000 delay model.

Ahmed and Abu-Lebdeh (2005) proposed a model for estimating delay at signalized intersections that is caused by downstream traffic disturbance such as queue spillback. The authors suggested adding one more delay term in the HCM 2000 delay model to represent the downstream induced delay, which is called \( d_4 \); the equation for \( d_4 \) is
\[ d_4 = \frac{n}{2} \left[ 2d_{4l} + (n-1)h_i \left( \frac{1}{v_i} - \frac{1}{\lambda_i} \right) \right] \]  
(2-11)

\[ d_{4l} = \frac{L_1 + L_2 + \text{off} - L_2}{V_a} - \frac{L_1}{\lambda_i} \]  
(2-12)

Where

- \( d_4 \): downstream induced delay, sec/veh
- \( n \): number of vehicles between upstream and downstream intersections
- \( d_{4l} \): portion of \( d_4 \) incurred by the first vehicle at the downstream intersection, %
- \( L_1 \): queue length at downstream intersection, ft
- \( L_2 \): remaining space on the link, ft
- \( h_i \): effective space headway, sec
- \( \lambda_i \): speed of mid-block stopping wave, ft/s
- \( v_i \): speed of mid-block starting wave, ft/s
- \( v_2 \): speed of stopping wave at downstream intersection, ft/s
- \( V_a \): average link speed, ft/s

The authors used CORSIM and the HCM 2000 delay model to get the intersection control delay for queue spillback conditions, and they concluded that there is additional delay compared to the HCM model and the new delay model can estimate delay well when compared to CORSIM.

A Shockwave is the boundary that marks the time-space domain of one flow state from another (May, 1990). It represents the discontinuity of speed and density. This method can be used for signalized intersections as shown in Figure 2-1.

In Figure 2-1, A, B, C, and D are different flow states. In state A, vehicles arrive at the intersection; in state B, vehicles stop at the intersection; in state C, vehicles discharge from the intersection; in state D, there is no flow. The lines separating different states are shockwaves. The triangle area between the shockwave \( w_{AB} \) and \( w_{BC} \) is considered as the total stop delay for vehicles stopped at an intersection.

Based on some previous research, Pueboobpaphan, and et al (2005) presented how to estimate travel speed along signalized arterials using shockwave analysis. The authors did a
modification of a conventional shockwave diagram so that it can present the traffic conditions more realistically. For example, the conventional shockwave analysis assumes that vehicles change from stopping to a certain speed instantly. Under this assumption, the authors suggested that delay at intersections would be estimated more accurately by modeling the speed changes over time; they modified the shockwave diagram and tested their results with simulated data. The results convinced the authors that the new model is more accurate. This research improved the estimation of speed changes around intersections, but in this paper it is assumed that a vehicle moves with constant speed on the link.

In queuing analysis, a queuing system can be defined by five characteristics (Gross and Harris, 1998). The definitions for the five aspects of a queuing system were extended for applications in signalized intersections.

- Inter-arrival time distribution.
  Inter-arrival time is the time interval between two consecutive arriving vehicles.

- Service time.
  Service time is the time interval for one vehicle to pass the intersection.

- Number of servers.
  It is the number of intersections that can “serve” the vehicles.

- System storage capacity.
  This is the maximum link length that one intersection can accommodate vehicles. It is the maximum number of vehicles that the upstream link of the intersection can accommodate.

- Queue discipline.
  It is assumed to be FIFO under the assumption that there is only one lane, or lane changing is limited on the arterial with multiple lanes.

In general, a queuing system can be written by using Kendall Notation (Gross and Harris, 1998):

\[
\text{Inter-arrival Time/Service Time/number of servers/System Capacity/ Queue Discipline}
\]
Queuing analysis can be applied to signalized intersection for estimating delay. There are two types of queuing analysis: deterministic queuing analysis and stochastic queuing analysis. Deterministic queuing analysis is applied in the intersection delay models, so it is not discussed in detail in this section.

In deterministic queuing analysis, the vehicles arrive at a deterministic rate; and the intersection serves at a deterministic rate when the signal is green and stops to serve when the signal is red.

As shown in Figure 2-2, the black bars represent red time and the white bars represent green time. There are two lines; one is the cumulative arrival line, which is the solid line. The other is the cumulative departure line, which is the dashed line. The area between the arrival line and the departure line is intersection delay for all the vehicles. Since the intersection delay models are based on deterministic queuing analysis, it is not considered as another method in estimating intersection delay in this research.

Stochastic queuing analysis is different from deterministic queuing analysis in that the arrival distribution and/or the service distribution are probabilistic.

For example, M/M/1/∞/FIFO presents a queuing system with exponential inter-arrival and service times, one server, infinite capacity and First-In-First-Out (FIFO) queue discipline.

Some studies were conducted in the past to find the performance measures for different queuing systems. Performance measures are important in that they are meaningful in evaluating how the system performs. Commonly used performance measures are: average service time and average waiting time in the system. Some widely used queuing systems such as M/M/1, M/G/1, and M/D/1 have been already studied and their equations for performance measures are ready to
Intersection stop delay can be estimated using the average waiting time provided by the stochastic queuing analysis.

A study performed by Geroliminis and Skabardonis (2005) used Markov Decision Process (MDP) to predict arrival profile and queue length along signalized arterials. This is an application of stochastic queuing analysis. MDP was used to model two consecutive traffic signals in this research; this approach can predict the arrival profile at downstream intersections, and the method can be applied to find travel time. They indicated that arterial travel time is greatly affected by platoon size and dispersion at intersections. Platoon size and dispersion can help to establish the relationship between two consecutive intersections, if the relationship can be found, and then arrival profile at downstream intersections can be predicted by the entering flow at the first upstream intersection.

By using MDP, that is: based on the arrival type on the previous intersection, what is the arrival type of the next intersection:

\[
P(A_{n+1} = i_{n+1} \mid A_n = i_n)
\]  

(2-13)

A nonlinear (concave) flow-density relationship was used to model platoon dispersion. A model was presented as the platoon dispersion as a function of distance. That is how platoons disperse after vehicles discharge from intersection. This way, the number of vehicles that can arrive at a downstream intersection can be known. Since signals “block” some vehicles from passing, a queue is created. Then with this method, queue length can be estimated. Therefore, it is a way to predict travel time.

### 2.1.1.2 Other arterial travel time models

Levinson (1996) performed a study about travel speed on arterials. He drew the conclusion that travel speed can be affected by two factors: same direction flow and cross direction flow. The impact of same direction flow is represented by the interactions between vehicles in the
same direction. The impact of cross direction flow is measured by intersection delay. These two are important in determining travel speed and should be considered in research about travel speed. This point of view identified that traffic volume and the status of the signal have great impacts on travel time variability.

Lum and Fan (1998) studied the speed-density relationship for arterials in Singapore and proposed a travel time-density model for interrupted flow. This method can be used for estimating travel time. They collected data on traffic volume and travel time to build the revised speed-density relationship. Since an arterial has interrupted flow, signals can affect the speed-density relationship. To adjust the impact, the number of intersections along the arterial is incorporated into the relationship. The journey time can be calculated as

\[ t_j = \frac{1}{u} = \alpha \exp(\beta k) + d \times f \]  

Where
\[ t_j \]: journey time, sec
\[ u \]: journey speed, ft/s
\[ k \]: density, veh/ft
\[ d \]: minimum delay per signalized intersection under free flow conditions, sec
\[ f \]: number of intersections
\[ \alpha \]: parameter related to journey speed
\[ \beta \]: parameter related to density

Olszewski (2000) conducted a comparison between this travel time estimation model from Singapore to the 1997 HCM arterial travel time model. Olszewski indicated that the first part of the model presents running time and incremental delay at signals which depends on traffic density, the second part of the model presents stopped delay which depends of density of signalized intersections. The compassion shows that the two models give similar results and HCM model gives lower speeds under un-congested conditions.

The above arterial travel time estimations do not consider the variability of the travel time. The following papers address the issue of variability in travel time estimation.
Fu and Hellinga (2000) studied the variability of delays at signalized intersections. They suggest delays that individual vehicles experienced at signalized intersections are subject to variations caused by random arrival. In this study, delay was defined as the sum of uniform delay and overflow delay. The variation of total delay is the sum of the variation of uniform delay and overflow delay.

As shown in several previous studies, arterial travel time is different from the travel time of other facilities in that an arterial has intersections along it and these cause vehicles to stop and make travel time longer. Because of this, intersection delay is also very important in finding arterial travel time. Because of the importance of intersection delay, some of the studies focus on intersection delay when studying arterial travel time so that they may simplify travel time estimation to intersection delay estimation.

Lin, Kulkarni, and Mirchandani (2004) performed a study on arterial travel time for the ATIS. In the study, they developed a model to estimate arterial travel time. In the model, they reduced travel time estimation to intersection delay estimation: in their paper, travel time is first defined as the sum of link travel time and intersection delay. They concluded that link travel time is not so sensitive to the traffic flow when the flow is medium or high. They suggested that link travel time should be estimated as a constant value. This method is similar to urban street travel time estimation in the HCM in the way that link travel time is not considered as a varying factor. While in this paper, they used the isolated intersection delay model. Therefore, they need to develop a transition matrix from conditional probability. The goal is to find what is the probability that a vehicle is in state $i$ based on that its previous state is $j$. So, states need to be defined.
There are two states defined in this study: 0 and 1: 0 represents the condition where the vehicle is not delayed at the intersection, while 1 represents the condition where the vehicle is delayed at the intersection. Therefore a 2×2 matrix is generated as follows:

\[
P = \begin{bmatrix}
p_{00}^i & 1 - p_{00}^i \\
1 - p_{11}^i & p_{11}^i
\end{bmatrix}
\]

(2-15)

Delay at intersections is calculated by using the conditional probability times Webster’s Delay Equation. The general model is

\[
d^{(i)} = \left[\delta(i), 1 - \delta(i)\right] \begin{bmatrix} p_{11}^i, 1 - p_{11}^i \end{bmatrix} E(D^{(i)})
\]

(2-16)

Where
- \(\delta(i)\): States that one intersection can have
- \(E(D^{(i)})\): Webster Delay Equation at the intersection, s/veh

This paper is considered as an improvement by the author in finding arterial travel time since it considered intersection delay with probability. Vehicles do not encounter intersection delay all the time, delay occurs with a certain probability. The transition matrix would be more realistic if the paper considered signal timing, offset, traffic flow (in consideration of queue), speed, and arrival type (in consideration of coordination) when it calculated conditional probabilities. The paper applied the model in four scenarios simulated in a simulation package. They concluded that the result is promising but needs to be verified with field data.

Besides the studies that considered the variability of travel time, there are other travel time studies which think travel time can be derived by estimating travel speed along the arterial. More recently, some studies are performed by deriving travel time which is the link length divided by travel speed.

Tarko (2006) et al. developed a method for estimating travel speed along urban arterial streets. The model does not need inputs such as signal timing and geometry information about intersections. The model is derived from the HCM intersection delay model; it captures the
arterial through movement travel speed. The inverse of the speed, called pace, equals the unit travel time. Pace can be calculated as

\[
p_i = \frac{3600}{V_0} + \frac{a_i}{l} \cdot \exp(a_z l) \left(1 - \frac{a_z F_i}{F_1 + F_2} \right) \left(1 + \frac{a_z F_i}{n_s} \right)
\]

(2-17)

Where

- \( V_i \): travel speed in direction \( i \), ft/s
- \( V_0 \): cruise speed, ft/s
- \( l \): average distance between adjacent signalized intersections, ft
- \( a_1, a_2, a_3 \): parameters of the model
- \( F_i \): average one-way flow in the analyzed direction \( i \), veh/hr/ln
- \( F_1, F_2 \): average one-way flows along the arterial street; veh/hr/ln
- \( F_s \): flow crossing the major road, veh/hr/ln. Select the stronger one-way volume on each side street and calculate the average
- \( n_s \): average number of through lanes in one direction on side streets
- \( n_i \): average number of through lanes in the considered direction on the major streets

The estimation results show that when compared to field data, the model underestimates travel time by 18%. The error is removed by adding one more adjustment parameter. This model is developed for planning purposes which usually does not have detailed traffic condition data. In this model, only the cruise speed, distance between intersections, number of through-lanes, and one-way volumes are required as inputs. The signal timing is not considered which may result in the biased estimation.

Based on their previous models in estimating arterial link travel speed, Xie and Cheu (2001) proposed a new model which they thought tends to improve the previous models.

The model is as follows:

\[
\text{Travel time} = \text{cruise time} + \text{signal delay}
\]

(2-24)

\[
\text{Signal delay} = 0.9 \Phi \left[ \frac{C(1 - \lambda)^2}{2(1 - \lambda x)} + \frac{x^2}{2q(1 - x)} \right]
\]

(2-25)
\[
\Phi = \begin{cases} 
\frac{(C - g)q - L_2}{(C - g)q}, & \text{if } (C - g)q \geq L_2 \\
0, & \text{if } (C - g)q < L_2 
\end{cases} 
\] 
\hspace{1cm} (2-26)

Cruise time = \frac{L_1}{u_{det}} 
\hspace{1cm} (2-27)

Where 
C: cycle length, sec 
\lambda: green ratio 
x: degree of saturation 
g: green time, sec 
q: flow rate, veh/hr/ln. 
\Phi: signal parameter 
L_1: Part of the link when vehicle can travel with normal speed, ft 
L_2: Part of the link when vehicle decelerates, it equals to link length minus L_1, ft 
u_{det}: Speed from detector. ft/s 

The model was tested with field data and the author concluded that the model gives better estimation than the previous models.

2.1.2 Arterial Travel Time Models Based on In-Field Detection Technologies

Besides the arterial travel time models reviewed above, there are other methods for estimating travel time by developing a model based on in-field data that are collected.

Since arterial travel time sometimes can not be estimated very accurately by some theoretical models such as the HCM model, some studies also try to estimate arterial travel time with the aid from advanced technologies. Usually, there are several technologies to help estimate arterial travel time: they are probe vehicle, observation through video technology, and loop detector.

Thus, some studies were completed with the aid of technologies as mentioned above to develop arterial travel time models, some of these studies are discussed here:

The National Institute of Statistical Science (NISS) performed two consecutive studies on arterial travel time; these studies were completed on year 1996 and 1998. In 1996, Sen, Soot, Ligas, and Tian (1996) did the first travel time study, which presents a comparison of some
arterial travel time models based on data collection methods, such as probe vehicles and detectors. The authors broadly reviewed the existing travel time estimation methods as mentioned above: Probe vehicles can measure travel time. With the Advanced Video Image technology, travel time can be measured directly. Detectors are used not only for arterials but also for freeways; they obtain volume and occupancy data and then being transferred to travel time data. Assignment-type models have a drawback that estimation is for a long time period so they are not very sensitive to external changes. The authors suggested that the estimated travel time is the sum of cruise time and stopped delay. Cruise time is the period of time when a vehicle is moving, and stopped delay is the period of time when a vehicle stops at a red signal or is slowly moving in a queue. Their conclusion was that variation in travel time mostly depends on variation in stopped delay.

The second study from NISS was performed by Graves, Karr, and Thakuriah (1998) as an extension of the previous one; in this study, the authors focused on travel time variability and they performed the study on a selected arterial in a Chicago suburban area. Travel time data were collected with a probe vehicle and loop detectors. The findings are that travel time variability relies on the signal status and the traffic volume when a particular vehicle enters the link. This paper presented a new variable in estimating arterial travel time: Relative Entry Time (RET). RET is related to the status of the downstream signal. In the paper, they developed a general travel time model and then adjusted the model for through, left turning, and right turning movements. Factors considered in travel time models are red time, green time, free flow travel time, time for each vehicle to clear the queue, and traffic volume. The general model is

$$\begin{align*}
TT(RET, n) &= FFTT + [TC(n) - (RET - R)] \\
\text{Where} & \\
RET &: \text{relative entry time, sec} \\
n &: \text{number of vehicles in the queue}
\end{align*}$$

(2-18)
\[ FFTT \]: free flow travel time, sec  
\[ TC(n) \]: time required for a queue of \( n \) vehicles to clear, sec  
\[ R \]: red time, sec  
Models for through, left, and right movements are different.

Zhang (1999) developed a model to find the journey speed along an arterial; the idea is if the speed can be estimated correctly, arterial travel time can be found by getting the link length divided by the speed. In his model, the loop detector provides occupancy measurement for developing the model.

The model is

\[
U_c = \gamma U_{v/c} + (1 - \gamma)U_{q/c}
\]  
\[(2-19)\]

Where  
\[ \gamma \]: a weighted factor which is between 0 and 1

\[
U_{v/c} = U_f - \alpha \exp \left( \beta \frac{V}{C} \right)
\]  
\[(2-20)\]

\[
U_{q/c} = 0.379 \sum \frac{q_i}{o_i}
\]  
\[(2-21)\]

Where  
\[ o_i \]: the occupancy measurement from the detector, %  
\[ \alpha \] and \[ \beta \]: parameters that need to be defined for each specific case

One available model to estimate the actual travel time is the “Illinois Model” (Zhang, 1999). The model presents a regression model for delay estimation, two of the regression parameters are based on data from detectors.

\[
T = UNDT + DELAY
\]  
\[(2-22)\]

Where  
\[ T \]: the link travel time, sec  
\[ UNDT \] is calculated as

\[
UNDT = \frac{L}{V_f}
\]  
\[(2-23)\]

Where  
\[ L \]: the link length, ft  
\[ V_f \]: the free flow speed, ft/s  
\[ DELAY = \beta_0 + \beta_1 deloc + \beta_2 o + \beta_3 grnrat \]

Where  
\[ deloc \]: ratio of detector setback to link length
o : detector occupancy, %
grnat : green ratio (green time over cycle length)
\( \beta_0, \beta_1, \beta_2, \beta_3 \) : regression parameters

Mark, Sadek, and Dickason (2005) applied Artificial Neural Networks (ANN) methods in predicting arterial travel time. The study tested the possibility for using ANN method based on detector data. The result shows that ANN is quite applicable in predicting arterial travel time in that it updates and recalculate the travel time when new travel time observation is obtained.

Travel time can also be predicted by analyzing historical data. To find the trend of travel time throughout a period of time and predict future travel time based on the trend discovered before, the method is called time series.

Liu and Shuldiner (2005) predicted travel time by the time series method. They collected field data on Massachusetts Route 9 for about 3.7 miles arterial with the aid of Automatic Vehicle Identification (AVI), which ensures the accuracy of field data for this research. With the AVI system, two cameras were used at the beginning and ending points of the road segment. Cameras can capture the license plate of vehicles using the optical character recognition, when the two plates match, one travel time observation can be generated.

There are three time series models used to develop a short-time travel time prediction model, they are: Auto Regressive Integrated Moving Average (ARIMA), ARIMA with traffic volumes as additional input variables, and Autoregressive Error Model. The first model is an ARIMA model that predicts travel time by the following equation:

\[
TT_t = a_1 TT_{t-1} + a_2 TT_{t-2} + C_1 Z_{t-1} \tag{2-28}
\]

Where
- \( TT_t \) : travel time of the \( t^{th} \) interval, sec
- \( TT_{t-1} \) : travel time of the \((t-1)^{th}\) interval, sec
- \( TT_{t-2} \) : travel time of the \((t-2)^{th}\) interval, sec
- \( a_1, a_2, C_1 \) : parameters of the model
- \( Z_{t-1} \) : white noise (0, \( \sigma \)) for the \((t-1)^{th}\) interval
The second model is also ARIMA, but this time with traffic volumes as additional input variables. This model assumes that current travel time can be affected by traffic volume as well as previous travel time, so the model is based on the first model with some additions:

\[ TT_t = a_1 TT_{t-1} + a_2 TT_{t-2} + b_1 X_{t-n} + b_2 X_{t-n-1} + \ldots + b_m X_{t-n-m-1} + C_1 Z_{t-1} \]  

Where

\[ X_{t-n} : \text{traffic volume of the (t-n)}^{th} \text{ interval, veh} \]
\[ X_{t-n-1} : \text{traffic volume of the (t-n-1)}^{th} \text{ interval, veh} \]
\[ X_{t-n-m-1} : \text{traffic volume of the (t-n-m-1)}^{th} \text{ interval, veh} \]
\[ n : \text{lag parameter between traffic flow and travel time} \]
\[ m : \text{number of intervals} \]
\[ a_1, a_2, C_1, b_1, b_2 \ldots b_m : \text{parameters of the model} \]
\[ Z_{t-1} : \text{white noise (0,} \sigma \text{) for the (t-1)}^{th} \text{ interval} \]

The third model is the Autoregressive Error Model:

\[ TT_t = a \times t + b + N_t \]
\[ N_t = N_{t-1} + Z_t \]

Where

\[ TT_t : \text{travel time of the t}^{th} \text{ interval, sec} \]
\[ t : \text{the number of the current interval} \]
\[ N_t : \text{noise in the t}^{th} \text{ interval} \]
\[ Z_t : \text{white noise (0,} \sigma \text{) for the t}^{th} \text{ interval} \]
\[ a, b, c : \text{parameters of the model} \]

The three models were tested and they are considered as reliable in predicting near future travel time, however, these models have a shortcoming in that they can only predict travel time under non-congested, no incident conditions. When the travel time increases greatly and rapidly in a short period of time, which means there is congestion, these models are not good predictors.

Du and Aultman-Hall (2006) developed a method for estimating road network travel time. In order to estimate the network travel time, the travel time of each link within the network has to be estimated. The research deals with the problem that some links have GPS data and some do not. The method classifies the links into groups. In each group, there are links which have GPS data and other links without GPS data. An expected travel time is estimated from the links which
have GPS data. For the links inside the group and do not have GPS data, a random travel time based on the expected value is generated and assigned to the link.

2.2 Simulation Methods in Arterial Analysis

Simulation can replicate the real traffic conditions and it can run for multiple times to get enough results for analysis. Thus, simulation methods are very helpful in many transportation analyses. Simulation packages are often used in arterial analysis in determining intersection delay and travel time.

2.2.1 Delay Models in Simulation Packages

Several simulation packages have their own intersection delay model. Some of the models are the same as the HCM model and others may have different ones. The delay models for PASSER, TRANSYT-7F, and Synchro are presented here.

PASSER (2002) has several packages to suit the need for different transportation facilities, among them, PASSER II focuses on arterials. PASSER II optimizes arterial progression and it is one of simulation software packages that has its own delay model. The model does not consider initial queue delay so it divided into two parts: uniform delay and random/saturation delay:

\[
d_u = 0.5 \times \frac{C(1 - g/C)^2}{1 - v/s} \times f
\]

\[
d_{rs} = 900T \left[ (X - 1) + \sqrt{(X - 1)^2 + \left(\frac{4X}{cT}\right)} \right]
\]

Where

- \(d_u\): uniform control delay, sec/veh
- \(d_{rs}\): random and saturation delay, sec/veh
- \(g\): effective green, sec
- \(C\): cycle length, sec
- \(v\): movement volume, veh/hr/ln
- \(s\): saturation flow rate, veh/hr/ln
- \(c\): capacity, or sxg/C, veh/hr/ln
- \(X\): degree of saturation
- \(f\): signal type (1.0 for pretimed and 0.85 for actuated),
- \(T\): time period adjustment factor.
This delay model is different from the HCM 2000 delay model. In the HCM 2000 model, there are three parts: uniform delay, incremental delay, and initial delay. In PASSER, there are only two parts, uniform delay and random/saturation delay, it does not include initial delay. PASSER’s uniform delay equation is slightly different from the HCM 2000 model, and in random delay which is called incremental delay in the HCM 2000, the upstream filtering adjustment factor \( I \) is assumed to be 0.5.

TRANSYT-7F (2002) has been developed and its delay model has been changed over time. Currently, the latest version, 10.1, uses the delay model from the HCM 2000 as its standard for delay estimation.

Synchro provides two options in delay calculation; users can choose the HCM signal delay or Synchro delay: the HCM signal delay uses the delay model in HCM 2000.

2.2.2 Estimating Delay and Travel Time Using Simulation

Simulation modeling replicates the reality and provides some useful evaluations on performance measures.

Signalized intersections along the arterial are essential for determining arterial operations; inappropriate signal timing and poor signal coordination between intersections can cause unnecessary delays on the arterial. Some research was completed in finding appropriate methods to optimize signals so that delay can be reduced. A selected review of such papers concerning optimizing signals is discussed here.

Travel time is greatly affected by intersection delay, which can be reduced by a certain amount if signals are optimized and coordinated with other signals. In the literature review, it is necessary to review papers that focus on signal optimization and coordination.

To find the optimal signal timing and an effective way to coordinate signals, several simulation software packages have been developed and applied to this research. Widely used
simulation packages include CORSIM, TRANSYT-7F, and PASSER. TRANSYT-7F and PASSER can provide optimal cycle length, green splits for all time periods or for different time period throughout a day, etc. Since these simulators don’t have the same algorithms underlying their optimization routines, several papers compare to find which software is suitable for certain circumstances, and provide recommendations for improving their simulation models.

Lin and Courage (1996) conducted research on how certain simulation software can determine phase times for actuated signals along the arterial. TRANSYT and TRANSYT-7F are tested in this paper. They stated that existing simulation models are not good enough and some of them are oversimplified since they are not considering some practical conditions on traffic operations, such as progression, and left turn movements. This paper focuses on how to determine average phase time in modeling actuated traffic signal control. In previous research, phase time calculations were aimed at reducing degree of saturation or queue service time.

Kamarajugadda and Park (2003) noticed that in some simulation packages intersection delay is calculated based on the HCM intersection delay model. In the model, delay is a function of multiple parameters; however, some of the variables such as volume, green time, and saturation flow rate are stochastic variables with their own distribution. This characteristic implies that the intersection delay would be better presented with a distribution rather than an average value. Therefore, they developed a delay variance equation for isolated intersections, and then expanded it to arterial intersections. This way, their model can optimize the arterial system better when compared with an existing simulation package such as Synchro.

2.3 Summary of the Literature Review

From the studies reviewed in this chapter, it is found that arterial travel time is estimation models fall into two ‘classes’. One is to develop an analytical model; either based on intersection control delay estimation or in-field technology. The other is to obtain the arterial travel time by
simulation. Some of the studies mentioned the variability of arterial travel time but none of them estimated it. The objective of this research is to develop an analytical model for estimating the expected arterial travel time as well as the variance of the travel time, and the travel time distribution. This methodology is discussed in more details in the next chapter.
Figure 2-1. Shockwave analysis

Figure 2-2. Deterministic queuing analysis
CHAPTER 3
METHODOLOGY

This chapter presents the study methodology. Section 3.1 studies a simple case to find the travel time for an arterial link between two intersections. Based on the method developed in the simple case, a generalized method is developed in Section 3.2 to model an arterial with multiple links. In Section 3.3, an example is presented to demonstrate the proposed methodology.

Travel time in this study is defined as the sum of “travel time in motion” and “time in queue”: travel time in motion is the period of time when the vehicle leaves the queue until it joins another queue. Time in the queue is the period of time when a vehicle joins the queue until it discharges from the queue. This part of travel time is essential in estimating arterial travel time accurately. In some previous papers, this amount of time is considered as delay at intersections. The simplest method from the HCM is to estimate the travel time as the sum of isolated intersection delay and the ideal travel time (link length divided by speed limit). However, that equation does not consider the effects from upstream and downstream intersections. In fact, besides the delay at the intersection, time in queue should also include moving time in queue when the queue is discharging. Thus, in this research, arterial travel time is defined as the sum of “travel time in motion”, “waiting time in queue” and “moving time in queue”. The three parts of travel time need to be estimated separately and then combined together to estimate the arterial travel time.

3.1 Simple Case: One Link between Two Intersections

In this study, to estimate travel time, a simple case is first studied and a general case is developed later based on the conclusions of the simple case. The simple case only includes one link between two intersections. Only the impact of the downstream intersection is considered. The simple case shows how to estimate the travel time for vehicles encountering different signal
status, regardless of what happened at the upstream intersection. Based on this case, the general case considers both the upstream and the downstream intersections for estimating travel time. A complete list of notation is provided in Appendix A for this research.

3.1.1 Definition of a Link

To estimate the arterial travel time in the simple case, the starting point and the ending point of the segment should be defined first. As shown in Figure 3-1, in this research, the starting point is defined as the stop line at the upstream intersection; the ending point is the stop line at the downstream intersection.

3.1.2 Decomposition of Travel Time

After defining the starting and ending points for the simple case, travel time can be decomposed into two parts for this basic segment as shown in Figure 3-2. The following equation defines the respective variables:

\[ ATT = T_M + T_Q \] (3-1)

Where
\( ATT \) (Arterial Travel Time): total travel time on the arterial;
\( T_M \) (Time in Motion): time period when the vehicles travel before they join the queue; and
\( T_Q \) (Time in Queue): time period from when the vehicles join the queue to when they leave the queue

Time in motion is a function of link length, queue length, initial speed, the acceleration rate, and the deceleration rate.

\( T_Q \) can be further divided into Waiting Time in the Queue and Moving Time in the Queue:

\[ T_Q = WT_Q + MT_Q \] (3-2)

Where
\( WT_Q \) (Waiting Time in the Queue): time that vehicles have to wait in the queue; and
\( MT_Q \) (Moving Time in the Queue): time that vehicles move when they are in the queue

Thus, the entire equation for \( ATT \) is

\[ ATT = T_M + WT_Q + MT_Q \] (3-3)
In this equation, the calculations for the three terms are different as the signal status changes. The $T_M$ variable is different since the trajectory of the vehicle changes as the signal status changes. For time in queue, there are two types of queues: moving queue and stopped queue; moving queue is the queue that is discharging when the signal status is green; stopped queue is the queue that is not discharging and vehicles are stopped when the current signal status is red. $WT_Q$ and $MT_Q$ are different for these two types of queues.

Thus, to estimate arterial travel time, the components of the arterial travel time need to be estimated. $T_M$ can be calculated if the vehicle motion is known. $WT_Q$ can be obtained from an intersection delay estimation since it is part of that. $MT_Q$ can be calculated by estimating the average queue length, the speed for the vehicle entering the queue and the speed leaving the queue. The equations for estimating $T_M$ and $MT_Q$ will be shown in the following sections, for various traffic conditions.

3.1.3 Estimation of $T_M$

Travel time can be accurately measured if the trajectory of the vehicle is recorded. $T_M$ can be different if the trajectory is different. Therefore, $T_M$ is estimated differently for different traffic conditions. Thus, this section presents the descriptions of the various conditions, and provides equations for $T_M$ for each condition. The three basic conditions are identified and shown in Figure 3-3.

Figure 3-3 shows that the vehicle trajectories arriving at an intersection. The dashed lines represent the vehicle trajectories; the black bar represents the effective red time and the white bar represents the effective green time of the intersection. According to the different vehicle trajectories, the cycle can be divided into three conditions as follows:
Condition 1: The vehicle arrives to decelerate until it stops, and then it has to wait. Some other vehicles may have arrived already, and the vehicle has to join the stopped queue. As shown in Figure 3-3, from \( t_1 \) to \( t_3 \).

Condition 2: The vehicle arrives when the current queue is moving and discharging: in this case the vehicle has to decelerate to join the moving queue and keep moving with the queue. As shown in Figure 3-3, from \( t_3 \) to \( t_4 \).

Condition 3: The vehicle arrives on green time and there is no queue at the intersection: in this case the vehicle can pass through the intersection with its desired speed. As shown in Figure 3-3, from \( t_4 \) to \( t_5 \). According to Figure 3-3, the interval length of the three conditions can be calculated as

\[
\text{Condition 1} = t_3 - t_1 = (t_3 - t_2) + (t_2 - t_1) = \frac{rW_{AB}}{W_{BC} - W_{AB}} + r \quad (3-4)
\]

\[
\text{Condition 2} = t_4 - t_3 = (t_4 - t_2) - (t_3 - t_2) = \frac{r(W_{AB}W_{BC})}{(W_{BC} - W_{AB})W_{AC}} \quad (3-5)
\]

\[
\text{Condition 3} = t_5 - t_4 = (t_5 - t_2) - (t_4 - t_2) = C - r - \frac{rW_{AB}}{W_{BC} - W_{AB}} \left( \frac{W_{BC}}{W_{AC}} + 1 \right) \quad (3-6)
\]

Where

\[
w_{AB} = -\frac{q_A}{k_B - k_A}, \quad w_{BC} = -\frac{q_C}{k_B - k_C}, \quad w_{AC} = -\frac{q_A - q_C}{k_A - k_C}
\]

\( w_{AB} \): shockwave speed between state A and B, mph;

\( w_{BC} \): shockwave speed between state B and C, mph;

\( w_{AC} \): shockwave speed between state A and C, mph;

\( q_A \): arrival rate (actual flow rate), veh/hr/ln;

\( q_C \): at capacity flow rate, veh/hr/ln;

\( q_A \): density at arrival rate, can be converted from flow divided by speed, veh/mi;

\( k_B \): jam density, veh/mi;

\( k_C \): density at capacity, veh/mi;

\( r \): effective red time, sec; and

\( C \): the cycle length, sec.

The sum of the three conditions is
Figure 3-3 is different from the one that appears in the literature review chapter since the vehicles trajectories after the green signal starts are not straight lines but curves. This is based on the theory of Kinematic Waves applied by Lighthill and Whitham (1955) to traffic flows at signalized intersections, where the increase of speeds for the discharged vehicles are achieved through a fan of waves of all possible velocities. These speeds are shown in Figure 3-3 as a fan when the green signal starts. The speeds change the trajectories of the vehicles from straight lines to curved lines. The equations that are used before (equation 3-4, 3-5, and 3-6) to calculate the time interval for three conditions will not be changed because the difference is negligible.

For the simple case, the total link length \( s \) is the distance from the starting point to the ending point of the link. As shown in Figure 3-4, it can be further divided into three parts: the acceleration distance \( (s_a) \), the constant speed distance \( (s_c) \), and the deceleration distance \( (s_d) \). \( s_d \) includes not only the distance for the vehicle to decelerate, but also includes the distance over which the queue extends \( (D_q) \) at the intersection. It is not necessary that all three distances are included in each travel time, as some of the travel times may only have one or two of them. \( T_M \) Equation for Condition 1 is

\[
T_M = \frac{V_m - V_{i\_link1}}{a_v} + \frac{s - \frac{V_m^2 - V_{i\_link1}^2}{2a_v} - \frac{D_q}{2a_d}}{V_m} + \frac{V_m}{a_d}
\]

Where
- \( V_{i\_link1} \): vehicles’ entering speed (initial speed) when they enter link 1, as shown in Figure 3-5. \( V_{i\_link1} \) can be determined by the vehicles trajectory in the previous link, ft/s
- \( V_m \): vehicles’ maximum operating speed when they travel on the link, ft/s
- \( a_v \): vehicles’ acceleration rate, ft/s²
Equation for Condition 2 is

\[ T_M = \frac{V_m - V_{i\_link1}}{a_d} + \frac{s - \frac{V_m^2 - V_{i\_link1}^2}{2a_d} - \frac{V_m^2 - V_{d\_link1}^2}{2a_d} - D_{q\_link1}}{V_m} + \frac{V_m - V_{d\_link1}}{a_d} \]  

(3-8)

Where
\[ V_{d\_link1} \]: discharging speed at link 1, ft/s, as shown in Figure 3-6
\[ V_{i\_link1}, V_m, a_d, a_d, D_{q\_link1} \] are same as defined for the \( T_M \) equation for Condition 1.

In this condition, the vehicle movement can be very complicated since the vehicle joins the discharging queue and may go back and forth between stopping and moving. Thus, the motion of the vehicle is not so easy to trace, in order to study these motions; they are simplified in this study by assuming constant speed when moving in the queue: for example, if vehicle’s entering speed is higher than the discharging speed, the motion is considered to be four parts. It accelerates with a constant acceleration rate until reaches the maximum speed, and then it keeps a constant speed until it decelerates from the maximum speed to the discharging speed, and then moves with the discharging speed in the queue. \( T_M \) Equation for Condition 3 is

\[ T_M = \frac{V_m - V_{i\_link1}}{a_d} + \frac{s - \frac{V_m^2 - V_{i\_link1}^2}{2a_d}}{V_m} \]  

(3-9)

Where
\[ V_{i\_link1}, V_m, a_d \] are same as defined for the \( T_M \) equation for Condition 1, \( V_{i\_link1} \) is shown in Figure 3-7.

For Condition 1 and 2, \( T_M \) equations contain the term \( D_{q\_link1} \), which is the distance over which the queue extends. Estimation of \( D_{q\_link1} \) is based on queue length. Although for each vehicle the \( D_{q\_link1} \) in front of it is different, in this analysis the average \( D_{q\_link1} \) for each condition will be used.
The average $D_{q_{\text{link1}}}$ can be estimated by graphically. In shockwave analysis, the average $D_{q_{\text{link1}}}$ for the stopped queue in Condition 1 and the moving queue in Condition 2 are approximately the same. The average $D_{q_{\text{link1}}}$ for the stopped queue and the moving queue is calculated as half of the maximum queue length as shown in the shockwave analysis. The equation is

$$\text{avg.}D_{q_{\text{link1}}} = \frac{Q_M}{2} = \frac{r}{7200} \left[ \frac{(w_{BC})(w_{AB})}{w_{BC} - w_{AB}} \right]$$

(3-10)

Where
- $\text{avg.}D_{q_{\text{link1}}}$: Average queue length at link 1, mi
- $Q_M$: Maximum queue length, mi
- $w_{AB}, w_{BC}$, and $r$ are same as previously defined.

### 3.1.4 Estimation for $QMT$

$QMT$ is the period of time that the vehicle moves in a queue, and it is not a motion that vehicles always experience. For the vehicles arriving in Condition 3, the queue is zero; therefore, moving time in the queue does not exist in this condition. However, under Condition 1 and Condition 2, $QMT$ does exist. For Condition 1, the vehicles join the stopped queue and when the traffic signal turns green, they begin to discharge. The time period for the vehicles to depart from the queue until they reach the stop line is $QMT$ for Condition 1. For Condition 2, the vehicles join the moving queue and move with other vehicles in the queue. The time period when the vehicles begin to move in the queue until they depart from the stop line is $QMT$ for Condition 2. Equations for these two conditions are developed below.

For each vehicle in Condition 1, $QMT$ depends on the position where the vehicle is in the queue. For each vehicle, the queue length equals to the distance for the vehicle to accelerate from zero speed. Thus, the time for this can be calculated by
\[ MT_Q = \sqrt{\frac{2D_{q\_link1}}{a_a}} \]  

(3-11)

Where \( D_{q\_link1} \) and \( a_a \) are as previously defined.

The average \( D_{q\_link1} \) for vehicles arrived in Condition 1 can be used in the above equation to estimate the average \( MT_Q \) (Equation 3-12). Thus:

\[ MT_Q = \sqrt{\frac{2\text{avg}.D_{q\_link1}}{a_a}} \]  

(3-12)

Where \( \text{avg}.D_{q\_link1} \) and \( a_a \) are same as previously defined.

For Condition 2, the vehicle joins the moving queue. Based on the assumption of constant speed within the queue, \( MT_Q \) is the time period for the vehicle to move with a constant speed, and the moving distance is the average \( D_{q\_link1} \), so for each vehicle, \( MT_Q \) is

\[ MT_Q = \frac{D_{q\_link1}}{V_{d\_link1}} \]  

(3-13)

Where \( D_{q\_link1} \) and \( V_{d\_link1} \) are same as previously defined.

The average \( D_{q\_link1} \) for vehicles arrived in Condition 2 can be used in the above equation to estimate the average \( MT_Q \) (Equation 3-14). Thus:

\[ MT_Q = \frac{\text{avg}.D_{q\_link1}}{V_{d\_link1}} \]  

(3-14)

Where \( \text{avg}.D_{q\_link1} \) and \( V_{d\_link1} \) are same as previously defined.

3.1.5 Estimation for \( W_T_Q \)

The vehicles arrive to join the stopped queue and have to wait until the signal turns green, thus, \( W_T_Q \) exists only for Condition 1. Different arriving times for different vehicles result in varying individual waiting times. The average \( W_T_Q \) can be calculated as half of the effective red time of the intersection.
3.1.6 Link Travel Time for All Three Conditions

As the equations for $T_M$, $MT_Q$, and $WT_Q$ of travel time are developed for each condition, the link travel time for the three conditions can be developed:

For Condition 1:

\[
ATT = T_M + WT_Q + MT_Q
\]

\[
= \frac{V_m - V_{l_{-link1}}}{a_u} + \frac{s - \frac{V_m^2 - V_{l_{-link1}}^2}{2a_u} - \frac{V_{l_{-link1}}^2}{2a_d}}{V_m} + \frac{V_m}{a_d} + \sqrt{\frac{2\text{avg}D_{d_{-link1}}}{a_u}} + WT_Q
\]

(3-15)

For Condition 2:

\[
ATT = T_M + MT_Q
\]

\[
= \frac{V_m - V_{l_{-link1}}}{a_u} + \frac{s - \frac{V_m^2 - V_{l_{-link1}}^2}{2a_u} - \frac{V_{l_{-link1}}^2}{2a_d}}{V_m} + \frac{V_m - V_d_{-link1}}{a_d} + \frac{\text{avg}D_{d_{-link1}}}{V_d}
\]

(3-16)

For Condition 3:

\[
ATT = T_M = \frac{V_m - V_{l_{-link1}}}{a_u} + \frac{s - \frac{V_m^2 - V_{l_{-link1}}^2}{2a_u}}{V_m}
\]

(3-17)

3.1.7 Probability that a Certain Condition Occurs

To estimate the expected value of travel time, the following probabilities should be considered: when one vehicle arrives at one intersection, the three conditions all have their own probabilities of occurrence. The three conditions described earlier present the three possibilities that one vehicle can meet when it arrives to an intersection and these probabilities are not equal, but should be estimated since they will be used to determine the percentage of vehicles in each condition. The arterial travel time variation is based on the probability that each condition can occur. Shockwave analysis can determine the proportion for each condition within one cycle.

Shockwave analysis (May 1990) defines the discontinuity of speed or density in the traffic stream. As shown in Figure 3-8, area A, B, and C indicate different speeds at one signalized intersection. Area A is the operating speed when the vehicles travel along the link, while area B
has zero speed when the vehicles are stopped at the intersection. Area C is the discharging speed when the vehicles begin to discharge at the beginning of the green. \( t_1, t_2, t_3, t_4, \) and \( t_5 \) are different time points that present the changes in queue length over the time period. \( t_1 \) is the time point when the signal turns red, \( t_2 \) is the time point when the signal turns green, \( t_3 \) is the time point when the queue clears, \( t_4 \) is the time point when the queue has dissipated at the intersection, \( t_5 \) is the time point when the signal turns red again. Therefore the time period between \( t_1 \) and \( t_3 \) is the time period when Condition 1 occurs. The time period between \( t_3 \) and \( t_4 \) is the time period when Condition 2 occurs. The time period between \( t_4 \) and \( t_5 \) is the time period when Condition 3 occurs.

The interval lengths of the three conditions are as defined before. Condition 1 occurs during the time period when vehicles have to join the stopped queue; it is from the beginning of red signal until the queue begins to move. Therefore the probability for Condition 1 is

\[
\Pr\{\text{Condition 1}\} = \frac{r + p \left( \frac{w_{AB}}{w_{BC} - w_{AB}} \right)}{C} = \frac{r w_{AB}}{w_{BC} - w_{AB}} + r \quad \text{(3-18)}
\]

Probability that Condition 2 occurs:

Condition 2 occurs during the time period when vehicles have to join the moving queue; it is from when the stopped queue begins to move until the queue dissipates. Therefore the probability for Condition 2 is

\[
\Pr\{\text{Condition 2}\} = \frac{r \left( w_{AB} w_{BC} \right)}{(w_{BC} - w_{AB}) w_{AC}} \quad \text{(3-19)}
\]

Condition 3 occurs during the time period when there is no queue present; it is from when the moving queue dissipates until the beginning of red signal for the next cycle. Therefore the probability for Condition 3 is
Travel time estimation is related to the probability that each condition occurs, as shown above, traffic flow rate, capacity, and density are involved in the equations for conditions. To consider arterials with multiple lanes, these three values will be used as per lane level.

With the probabilities for all three conditions, the expected link travel time for the simple case can be developed as

$$E[T_{total}] = E[T_{M} + W_{Q} + M_{T_{Q}} | \text{cond} = 1] P[\text{cond} = 1] + E[T_{M} + W_{Q} + M_{T_{Q}} | \text{cond} = 2] P[\text{cond} = 2] + E[T_{M} + W_{Q} + M_{T_{Q}} | \text{cond} = 3] P[\text{cond} = 3]$$  \hspace{1cm} (3-21)

3.2 General Case: An Arterial with Multiple Intersections

With the link travel time model developed, the arterial travel time model can be developed by generalizing the simple case, and considering the interaction between successive intersections.

The travel time on an arterial with multiple intersections would be very different if the vehicle meets with different traffic signal status: for example, travel times are different for two extreme cases: one is the vehicle travels on an arterial and arrive at all intersections in Condition 1 and has to stop, the other is the vehicle travels on an arterial and arrive at all intersections in Condition 3, thus, the vehicle does not need to reduce its speed. The travel time for these two vehicle trajectories can be very different from each other.

3.2.1 The Flow Profile

3.2.1.1 Travel time estimation under non-congested conditions

As shown in Figure 3-9, for an arterial with four intersections, each of the three conditions at each intersection has a certain probability for encountering another condition or encountering the same condition at the downstream intersection:
As shown in Figure 3-9, there are $3^4$ possible combinations. Each of them presents a group of vehicle’s travel trajectory; it is called a flow profile. To find the total arterial travel time, these possibilities need to be calculated and the travel time associated with each possibility also need to be estimated. To study the arterial with several intersections, it is necessary to find the 9 possibilities between two consecutive intersections, with the methodology for calculating the 9 possibilities; the methodology can be further used to find all possibilities for more than two intersections.

As shown in Figure 3-10, suppose there are two consecutive intersections, intersection 1 and intersection 2, and one link, link 1 between them. There is one additional intersection, intersection 0, and one additional link, link 0. Intersection 0 and Link 0 are used to determine the entering speed at intersection 1.

The 9 possibilities between two consecutive intersections are:

$P_{11}$ : It is the probability that the arrival vehicle stops at two consecutive intersections.

$P_{12}$ : It is the probability that the arrival vehicle stops at the first intersection and joins the discharging queue at the downstream intersection.

$P_{13}$ : It is the probability that the arrival vehicle stops at the first intersection and arrives without queue at the downstream intersection

$P_{21}$ : It is the probability that the vehicle joins the discharging queue at the first intersection and arrives to stop at the downstream intersection

$P_{22}$ : It is the probability that the arrival vehicle joins the discharging queue at two consecutive intersections

$P_{23}$ : It is the probability that the arrival vehicle joins the discharging queue at the first intersection and arrives without queue at the downstream intersection
\[ P_{31} : \text{It is the probability that the vehicle arrives without queue at the first intersection and arrives to stop at the downstream intersection} \]

\[ P_{32} : \text{It is the probability that the vehicle arrives without queue at the first intersection and arrives to join the discharging queue at the downstream intersection} \]

\[ P_{33} : \text{It is the probability that the vehicle arrives without queue at two consecutive intersections} \]

Link travel times for the 9 different flow profiles can be developed based on the travel time equations for the three conditions. In the equations, since the entering speed is not known, the travel time cannot be finalized. Now conditions at two consecutive intersections are known, thus, the specific entering speed can be estimated, and the link travel time equation can be developed for the 9 cases.

The entering speed refers to the speed that the vehicles enter link 1. The entering speed for the vehicles arrive at Condition 1 at the intersection 1 is calculated as below:

Since the vehicle encounters Condition 1 at the first intersection, it is assumed that the vehicle starts to travel with an initial speed of 0 and has to travel the distance over which the queue extends to enter the next link, thus:

\[
D_{\text{q, link}_0} = \frac{V_{i, \text{link}_1}^2 - V_{f, \text{link}_0}^2}{2a_a} = \frac{V_{i, \text{link}_1}^2}{2a_a} \quad (3-22)
\]

Where

- \( V_{f, \text{link}_0} \): Leaving speed (final speed) for link 0, ft/s
- \( D_{\text{q, link}_0} \): Distance over which the queue extends at link 0, mi
- \( V_{i, \text{link}_1} \) is same as previously defined

The entering speed for the link is

\[
V_{i, \text{link}_1} = \sqrt{2a_a D_{\text{q, link}_0}} \quad (3-23)
\]

Where \( V_{i, \text{link}_1}, a_a, \text{ and } D_{\text{q, link}_0} \) are same as previously defined
There are two queue lengths, one is \( D_{q_{\text{link}0}} \) and the other is \( D_{q_{\text{link}1}} \). Figure 3-11 shows the different location of \( D_{q_{\text{link}0}} \) and \( D_{q_{\text{link}1}} \).

Vehicles have a maximum operating speed as the limitation of their speeds, thus \( V_{i_{\text{link}1}} \) has to be compared with \( V_m \), the maximum operating speed on the arterial, if \( V_{i_{\text{link}1}} > V_m \), use \( V_m \) instead of \( V_{i_{\text{link}1}} \).

The entering speed for the vehicles arrive at Condition 2 at the first intersection is calculated as below:

Since the vehicle encounters Condition 2 at the first intersection, its entering speed for the link is the discharging speed at the upstream intersection:

\[
V_{i_{\text{link}1}} = V_{d_{\text{link}0}}
\]  
(3-24)

Where \( V_{d_{\text{link}0}} \): The discharging speed from link 0

Based on the assumption that the vehicles discharge with a constant speed, \( V_{d_{\text{link}0}} \) can be estimated by finding the speed at the time point when vehicles joining the queue at intersection 1.

The entering speed for the vehicles arrive at Condition 3 at the first intersection is calculated as below:

Since the vehicle encounters Condition 3 at the first intersection, its entering speed for the link is the maximum speed at the upstream intersection:

\[
V_{i_{\text{link}1}} = V_m
\]  
(3-25)

With the equations for \( V_0 \), the link travel time equation can be developed as follows:

Link travel time for \( P_{11} \) :
\begin{align*}
\tau_{t1} &= T_M + WT_Q + MT_Q \\
&= V_m - \frac{2a_d D_{q \_link0}}{a_e} s - \frac{V_m^2 - 2a_d D_{q \_link0}}{2a_d} - \frac{V_m^2 - V_d^2}{2a_d} - D_{q \_link1} + V_m + \sqrt{\frac{2D_{q \_link1}}{a}} + WT_Q \\
\text{Link travel time for } P_{t2} : \\
\tau_{t2} &= T_M + MT_Q \\
&= V_m - \frac{\sqrt{2a_d D_{q \_link0}}}{a_e} s - \frac{V_m^2 - 2a_d D_{q \_link0}}{2a_d} - \frac{V_m^2 - V_d^2}{2a_d} - D_{q \_link1} + V_m - V_d + \sqrt{\frac{2D_{q \_link1}}{a}} + WT_Q \\
\text{Link travel time for } P_{t3} : \\
\tau_{t3} &= T_M = \frac{V_m - \sqrt{2a_d D_{q \_link0}}}{a} s - \frac{V_m^2 - 2a_d D_{q \_link0}}{2a_d} V_m + \sqrt{\frac{2D_{q \_link1}}{a}} + WT_Q \\
\text{Link travel time for } P_{21} : \\
\tau_{t4} &= T_M + WT_Q + MT_Q \\
&= V_m - V_{d \_link0} + \frac{s}{2a_d} \left( \frac{V_m^2 - V_{d \_link0}^2}{2a_d} - \frac{V_m^2 - V_d^2}{2a_d} - D_{q \_link1} \right) + V_m + \sqrt{\frac{2D_{q \_link1}}{a}} + WT_Q \\
\text{Link travel time for } P_{22} : \\
\tau_{t5} &= T_M + MT_Q \\
&= V_m - V_{d \_link0} + \frac{s}{2a_d} \left( \frac{V_m^2 - V_{d \_link0}^2}{2a_d} - \frac{V_m^2 - V_d^2}{2a_d} - D_{q \_link1} \right) + V_m - V_d + \frac{D_{q \_link1}}{V_d} \\
\text{Link travel time for } P_{23} : \\
\tau_{t6} &= T_M = \frac{V_m - V_{d \_link0}}{a} s - \frac{V_m^2 - V_{d \_link0}^2}{2a_d} V_m + \sqrt{\frac{2D_{q \_link1}}{a}} + WT_Q \\
\text{Link travel time for } P_{31} : \\
\tau_{t7} &= T_M + WT_Q + MT_Q = \frac{s}{2a_d} \left( \frac{V_m^2 - D_{q \_link1}}{2a_d} \right) + V_m + \sqrt{\frac{2D_{q \_link1}}{a}} + WT_Q \\
\text{Link travel time for } P_{32} : \\
\tau_{t8} &= T_M + MT_Q \\
&= V_m - V_{d \_link0} + \frac{s}{2a_d} \left( \frac{V_m^2 - V_{d \_link0}^2}{2a_d} - \frac{V_m^2 - V_d^2}{2a_d} - D_{q \_link1} \right) + V_m - V_d + \frac{D_{q \_link1}}{V_d} \\
\text{Link travel time for } P_{33} : \\
\tau_{t9} &= T_M = \frac{V_m - V_{d \_link0}}{a} s - \frac{V_m^2 - V_{d \_link0}^2}{2a_d} V_m + \sqrt{\frac{2D_{q \_link1}}{a}} + WT_Q
\end{align*}
Link travel time for $P_{33}$:

\[ t_{32} = T_M + MT_Q = \frac{s - \frac{V_m^2 - V_{d_{-link1}}^2}{2a_d} - D_{a_{-link1}}}{V_m} + \frac{V_m - V_{d_{-link1}}}{a_d} + \frac{D_{a_{-link1}}}{V_d} \]  

(3-33)

The trajectories for the nine possibilities are shown in Figure 3-12. The next step is to develop the equations to estimate the probability that each flow profile occurs. All the vehicles arrive in Condition 1 at the first intersection departs when Condition 2 starts, these vehicles have possibility to arrive in Condition 1, 2 and 3 at the next intersection. As the same logic, the vehicles arrive in Condition 2 and 3 at the first intersection can also distributed into Condition 1, 2 and 3 at the next intersection. To find the probability of the 9 possibilities can be later used as the probability of flow profiles.

As shown in Figure 3-13, the probabilities of specific condition occurrence are determined using the offset, $T_M$, green time, red time, and cycle length.

The method to calculate the probabilities are as follows:

- **Step 1**: for vehicles arrive in Condition 1, find the travel time for the first vehicle and the last vehicle. And then locate the travel times for these two at the time bar on Figure 3-13, the time range between the two travel times are the possible travel times for the entire group of vehicles.

- **Step 2**: if the time range overlap with the conditions at next intersection, find the length of each interval, and then calculate the percentage for each interval, that is the probability that each flow profile occurs.

The same steps can be applied for vehicles arrive in Condition 2 and 3 at the first intersection also, therefore, the 9 probabilities for the 9 flow profiles can be estimated.

One problem in calculating the travel time is travel time depends on the conditions that the vehicles will meet in the next intersection; however, this cannot be determined beforehand. The solution to this problem is to calculate literately, the detailed procedure is as follows:
Suppose the vehicle arrive in Condition 1, define the trajectory of the vehicle and then calculate the travel time. And then locate the time in the time bar for the next interaction, if the time point falls into the range for Condition 1 at the next intersection, then the calculation is correct. Otherwise, continue with assumptions that the vehicle arrive in Condition 2 or 3, until the calculation is correct.

This method is applied to an example and it is shown at the end of the chapter to demonstrate the whole process.

3.2.1.2 Travel time estimation under congested conditions

From Figure 3-3, it is found that the three conditions are all completed during one cycle. If the sum of the three conditions is greater than the cycle length, not all the three conditions can be completed within one cycle. Thus, some vehicles cannot pass the intersection and they form a residual queue. Residual queue is defined as the group of vehicles that cannot pass the intersection during the first green interval after they arrive. In the field, there are two possibilities: a) Condition 3 is terminated; this does not create a residual queue because Condition 3 is the time difference between the discharge of the current queue and beginning of the next cycle. b) Condition 2 is terminated, as shown in Figure 3-14 which creates a residual queue. If the flow and density do not change from cycle to cycle, the residual queue keeps increasing as shown in Figure 3-15. For example, if the residual queue at the first cycle is \( n \), the new residual queue at the next cycle will be \( 2n \); the next one will be \( 3n \), and so on. The residual queue cannot be reduced unless demand is reduced.

As shown above, congested conditions are defined to occur when at least one vehicle cannot pass the intersection during the first green interval after they arrive. According to this definition the approach is congested when condition 2 is terminated earlier.
If Condition 2 is terminated, the residual queue can be calculated based on shockwave analysis.

As shown in Figure 3-16, the difference between when Condition 2 is completed and when it is terminated can be calculated as:

Complete Condition 2 time interval: \[
\frac{rW_{AB}}{W_{BC} - W_{AB}} \left( \frac{W_{BC}}{W_{AC}} + 1 \right) - \frac{rW_{AB}}{W_{BC} - W_{AB}}
\]

Terminated Condition 2 time interval: \[
C - \frac{rW_{AB}}{W_{BC} - W_{AB}} + r
\]

The difference:
\[
\frac{rW_{AB}}{W_{BC} - W_{AB}} \left( \frac{W_{BC}}{W_{AC}} + 1 \right) - \frac{rW_{AB}}{W_{BC} - W_{AB}} - C + \frac{rW_{AB}}{W_{BC} - W_{AB}} - r
\]

The residual queue can be calculated as:
\[
\frac{Q_R}{\text{Diff}_{\text{cond}2}} = \frac{Q_M}{\text{Cond}2}
\]

Thus,
\[
Q_R = \frac{Q_M}{\text{Cond}2} \times \text{Diff}_{\text{cond}2}
\]

(3-35)

Where
- \(Q_R\): The residual queue, ft
- \(Q_M\): The maximum queue, ft
- \(\text{Diff}_{\text{cond}2}\): The difference between the completed Condition 2 and the terminated Condition 2, sec

After the residual queue is calculated, the total queue can be estimated by adding the residual queue length to the initial queue length. The travel times can be estimated cycle by cycle and compared to the cycle–based simulated travel times.

Travel time varies as a function of the queue length. For the first cycle, there is no residual queue so the expected travel time can be calculated as before. Starting from the second cycle, the residual queue keeps increasing from \(Q_R\) to \((n-1)Q_R\). The residual queue has to be added to the current queue length when calculating travel time. It can affect travel time in two ways:
$T_M$: this part of travel time happens on the segment which is not occupied by queue. If the residual queue exists and keeps increasing, the segment length is reduced and $T_M$ is affected.

$MT_Q$: this part of travel time occurs when the queue is discharging. Since the queue length is increased, $MT_Q$ is also affected.

The queue length used in calculating $T_M$ and $MT_Q$, it is defined as the average queue length. It has to be adjusted according to the residual queue:

As shown in Figure 3-17 using the second cycle as an example, the new average queue is the average queue length of the queue formed in Condition 1 and Condition 2.

The general calculation steps are as follows:

For the $n$th cycle length, the residual queue at the beginning of the cycle is $(n-1)Q_R$, and the residual queue at the end of the cycle is $nQ_R$.

The average queue length for Condition 1 is \[ \frac{Q_M}{2} + (n-1)Q_R \]

The average queue length for Condition 2 is \[ \frac{Q_M - Q_R}{2} + nQ_R \]

Thus, the new average queue length is \[ \frac{Q_M}{2} + (n-\frac{3}{4})Q_R \]

There are two groups of vehicles. The first one is the vehicles joining the residual queue and waiting for one more cycle to pass the intersection. The second one is the vehicles passing the intersection during the current cycle. Their travel times are different since the travel time for the queued vehicle is the travel time for the un-queued vehicle plus one cycle length. The percentage of each group has to be calculated in order to calculate an expected travel time. Use the second cycle as an example: As shown in Figure 3-18, the maximum queue length for the cycle is $Q_M + Q_R$, the residual queue when the cycle is ended is $2Q_R$, the discharged queue length
is $Q_M - Q_R$, if the value is greater than $Q_R$, then the percentage for the first group is $\frac{Q_R}{Q_M - Q_R} \times 100\%$.

If the value is smaller than $Q_R$, then the percentage for the first group is 100%. Under this situation, there are vehicles that have to wait more than one cycle. To generalize these for the $n$th cycle, the discharged queue is $Q_M + nQ_R - (n-1)Q_R = Q_M - Q_R$. Thus, no matter which cycle it is, the discharged queue remains the same.

With consideration of the above problems, to calculate the expected travel time for congested conditions, the steps are shown below:

- Starting from the first cycle, calculate the expected travel time as before.
- Starting from the second cycle, calculate the residual queue for each cycle and add the queue length when calculating TM and MTQ.
- Calculate the expected travel time.
- Determine the percentage of vehicles that belongs to the queued group and the un-queued group.
- Calculate the expected travel time as

$$E(TT) = TT \times \%\text{ queued} + TT \times \%\text{ non-queued}$$

It is found that if $Q_M - Q_R$ is smaller than $Q_R$, some vehicles in the residual queue cannot pass the intersection even after waiting for one cycle. When calculating the expected travel time, this situation has to be considered.

To generalize the method, the discharged queue is always $Q_M - Q_R$. In this queue, the percentage of vehicles arriving during a given cycle and discharged during current cycle would be determined. Then the expected travel time can be calculated based on the travel times and percentages associated with them.
3.2.2 Inputs and Outputs in the Model

This section describes the inputs and outputs for the travel time estimation model for a small system of two consecutive intersections and a link between them.

3.2.2.1 Inputs

There are several inputs in the model to estimate travel time. They are: flow rate at the downstream intersection, entering speed at the downstream intersection, the link length, and the queue length. Among the variables, flow rate needs to be estimated and the discharging speed needs to be assumed.

- **Flow rate at the downstream intersection based on upstream flow.**

  The flow rate for the internal arterial approach can be calculated based on the initial flow rate: flow rate of the incoming approaches is an input to the methodology:

  \[
  f_{i\_link1} = f_{f\_link0} - f_{int1\_WR} - f_{int1\_WL} + f_{int1\_NR} + f_{int1\_SL}
  \]

  Where
  - \( f_{i\_link1} \): Entering (initial) flow rate at link 1 upstream intersection, veh/hr/ln
  - \( f_{f\_link0} \): Final flow rate at link 0 downstream intersection, veh/hr/ln
  - \( f_{int1\_WR} \): Flow rate at intersection 1 from westbound right turning movement, veh/hr/ln
  - \( f_{int1\_WL} \): Flow rate at intersection 1 from westbound left turning movement, veh/hr/ln
  - \( f_{int1\_NR} \): Flow rate at intersection 1 from southbound right turning movement, veh/hr/ln
  - \( f_{int1\_SL} \): Flow rate at intersection 1 from northbound left turning movement, veh/hr/ln

- **Discharging speed at each intersection.**

  For the arterial travel time, the entering speed for the next intersection is a very important value in estimating travel time for multiple intersections, as shown before; discharging speed from the upstream link is the entering speed for the next link. Based on the previous entering speed calculation, entering speeds for different conditions at downstream intersections are:

  Entering speed for Condition 1:

  \[
  V_{i\_link1} = \sqrt{2a_g D_{q\_link0}} \text{ or } V_m
  \]

  (3-23)
Entering speed for Condition 2:

\[ V_{i\_link1} = V_{d\_link0} \]  

(3-24)

Entering speed for Condition 3:

\[ V_{i\_link1} = V_{m} \]  

(3-25)

The entering speed for Condition 1 and 3 are known. However, the entering speed for Condition 2 needs to be assumed. Vehicles are discharging so that the flow is the maximum flow, according to the flow-density-speed relationship, assuming a linear speed-density relationship, the speed is approximately \( \frac{V_0}{2} \). It is assumed that this will be the speed for Condition 2.

- **List of assumptions.**

Besides the discharging speed assumption, there are several other assumptions that are to be used; these values are obtained from four resources to get the appropriate values: Traffic Engineering Handbook (for acceleration rate), A Policy on Geometric Design of Highways and Streets (for deceleration rate), the HCM 2000 (for Single Vehicle Spacing), and field data collection (for Maximum Operating Speed). They are as follows:

- Acceleration rate: 10.76 \( \text{ft/s}^2 \)
- Deceleration rate: -11.2 \( \text{ft/s}^2 \)
- Maximum Operating Speed: as the speed limit of the arterial

**3.3.2.2 Outputs**

The outputs of the model are the expected travel time, the travel time distribution, and the variance of the travel time for the whole arterial. A detailed summary of inputs and outputs for the model are included in table 3-1.
3.3 An Example for Applying the Model

The following example illustrates an application of the model. Suppose there is a signalized arterial with two intersections (Figure 3-20).

The question is what is the expected travel time on link 1 (between intersection 1 and intersection 2)?

3.3.1 Inputs for the Model

- The link length between intersection 1 and 2 is 1320 ft, or 0.25 mile.
- The arterial has two-way traffic, with one lane per direction.
- Maximum operating speed: The maximum operating speed is assumed to be equal to the speed limit of the arterial, which is 30 mph.
- Traffic signal control: There are pre-timed signals at each intersection, the cycle length is 80 seconds, the main arterial effective green time is 60 seconds, and effective red time is 20 seconds. The offset between intersections is 30 seconds.
- Traffic volume: \( f_{\text{link0}} \) is 900 veh/h.
- At each intersection, traffic volumes are as follows: NB and SB LT is 50 veh/hr/ln, NB and SB RT is 100 veh/hr/ln, EB LT is 5% of the \( f_{\text{link0}} \), EB RT is 10% of the \( f_{\text{link0}} \), EB TH is 85% of the \( f_{\text{link0}} \).
- Acceleration and deceleration rates are 10 ft/s².

3.3.2 Model Development

The goal is to find the expected travel time between intersection 1 and intersection 2, as well as the variance of the travel time. To estimate travel time, the following steps are to be completed:

3.3.2.1 Step 1: Calculate the entering flows at two intersections

**Intersection 1:**
\[
\frac{f_{\text{link1}}}{\text{veh/h}} = f_{\text{link0}} - f_{\text{int1 WR}} - f_{\text{int1 WL}} + f_{\text{int1 NR}} + f_{\text{int1 SL}}
\]
\[
= 900 - 900 \times 10\% - 900 \times 5\% + 50 + 100 = 915 \text{ veh/h}
\]

**Intersection 2:**
\[
\frac{f_{\text{link2}}}{\text{veh/h}} = f_{\text{link0}} - f_{\text{int2 WR}} - f_{\text{int2 WL}} + f_{\text{int2 NR}} + f_{\text{int2 SL}}
\]
\[
= (915) \times 85\% + 50 + 100 = 928 \text{ veh/h}
\]
Where

\( f_{i_{\text{link2}}} \): Entering (initial) flow rate at link 2
\( f_{f_{\text{link1}}} \): Final flow rate at link 1
\( f_{\text{int2,WR}} \): Flow rate at intersection 2 from westbound right turning movement
\( f_{\text{int2,WL}} \): Flow rate at intersection 2 from westbound left turning movement
\( f_{\text{int2,NR}} \): Flow rate at intersection 2 from southbound right turning movement
\( f_{\text{int2,SL}} \): Flow rate at intersection 2 from northbound left turning movement

### 3.3.2.2 Step 2: Calculate the probabilities for Conditions 1, 2, and 3

Using intersection 1 as an example, the probability for Conditions 1, 2, and 3 can be calculated as

\[
W_{AB} = -\frac{q_A}{k_B - k_A}, \quad W_{BC} = -\frac{q_C}{k_B - k_C}, \quad W_{AC} = -\frac{q_A - q_C}{k_A - k_C}
\]

In this example, based on the assumption of a linear function for the speed-density relationship (Figure 3-21), the calculations are as follows:

\[
q_{A_{\text{int1}}} = 915 \text{ veh/hr}, \quad q_{C_{\text{int1}}} = 1800 \text{ veh/hr},
\]

\[
k_{C_{\text{int1}}} = \frac{q_{C_{\text{int1}}}}{V_m / 2} = \frac{1800}{30 / 2} = 120 \text{ veh/mile}
\]

\[
k_{B_{\text{int1}}} = \frac{q_{C_{\text{int1}}}}{V_m / 2} \times 2 = \frac{1800}{30 / 2} \times 2 = 240 \text{ veh/mile}
\]

Based on the four known values, the mathematical expression of the flow-density curve can be calculated, it is \( Y = -15X^2 + 3600X \)

\[
q_{A_{\text{int1}}} \text{ is } 915 \text{ veh/hr, thus, } k_{A_{\text{int1}}} = -15k_{A_{\text{int1}}}^2 + 3600k_{A_{\text{int1}}}, \quad k_{A_{\text{int1}}} = 0.254 \text{ veh/mi}
\]

Thus,

\[
w_{AB} = -\frac{q_A}{k_B - k_A} = -\frac{915}{180 - 0.254} = -3.817, \quad w_{BC} = -\frac{q_C}{k_B - k_C} = -\frac{1800}{240 - 120} = -15.00, \quad w_{AC} = -\frac{q_A - q_C}{k_A - k_C} = -\frac{900 - 1800}{0.254 - 120} = -7.39
\]

\[
\Pr\{\text{Condition 1}\} = \frac{R + R\left(\frac{w_{AB}}{w_{BC} - w_{AB}}\right)}{C} = \frac{R\left(\frac{w_{BC}}{w_{BC} - w_{AB}}\right) - \frac{15.00}{-15.00 + 3.817}}{80} = 0.335
\]

\[
\Pr\{\text{Condition 2}\} = \frac{R\left(\frac{w_{AB}}{w_{BC} - w_{AB}}\right)}{C} = \frac{20(-3.817 \times (-15.00))}{(-15.00 + 3.817) \times (-7.39)} = 0.173
\]
Thus, the same method of calculation is performed for intersection 2. The calculation results for intersection 1 and intersection 2 are shown in Table 3-2.

3.3.2.3 Step 3: Calculate the queue lengths and determine $V_{i_{-link1}}$ for three conditions

$D_{q_{-link1}}$ for condition 1 and Condition 2

$D_{q_{-link1}}$ is one factor in calculating $T_M$ for vehicles arriving at the intersection in Condition 1 and Condition 2. The average $D_{q_{-link1}}$ for the stopped queue in Condition 1 and the moving queue in Condition 2 are approximately the same.

$$\text{avg.} D_{q_{-link1}} = \frac{Q_M}{2} = \frac{r}{7200} \left[ \frac{w_{BC}}{w_{BC} - w_{AB}} \right] \times 5280$$

$$= \frac{20 \times 5280}{7200} \left[ -15 \times (-3.817) \right] = 76.49 \text{ ft}$$

$$\text{avg.} D_{q_{-link0}} = \frac{Q_M}{2} \times \text{veh} \_ \text{spacing}$$

$$= \frac{r}{7200} \left[ \frac{w_{BC}}{w_{BC} - w_{AB}} \right] \times 5280 = \frac{20 \times 5280}{7200} \left[ -15 \times (-3.870) \right] = 75.08 \text{ ft}$$

Calculate $V_{i_{-link1}}$ for Condition 1, Condition 2, and Condition 3:

Entering speed for Condition 1:

$$V_{i_{-link1}} = \sqrt{2a \_d \_d_{q_{-link0}}} = \sqrt{2 \times 10 \times 75.08} = 38.75 \text{ ft/s} = 26.42 \text{ mph} < V_m$$

Thus, $V_{i_{-link1}} = 26.42 \text{ mph}$

Entering speed for Condition 2:

$$V_{i_{-link1}} = V_{d_{-link0}} = \frac{V_m}{2} = \frac{30}{2} = 15.00 \text{ mph}$$

Entering speed for Condition 3:

$$V_{i_{-link1}} = V_m = 30 \text{ mph}$$
3.3.2.4 Step 4: Calculate the minimum and maximum travel time for each possible flow profile.

For vehicle starts from Condition 1 at the first intersection, calculate the travel time for the first vehicle in the group, and then locate the time point on the travel time bar for the next intersection:

\[ t_{t_{11}} = 27.38 \, \text{sec} \]

It is found that the time point for this travel time on the time bar for the next intersection is 54.21 sec, which is smaller than the end time point of Condition 1 on the next intersection (56.95 sec). Thus, the minimum travel time point is 54.21 seconds.

Find the time intervals for each possible flow profile and then calculate the probability.

For vehicles that arrive in Condition 1 at the first intersection, there is only one time interval, the probability is

\[ P_{t_{11}} = \frac{56.95 - 54.21}{56.95 - 54.21} = 1.0 \]

3.3.2.5 Step 5: Calculate travel time components \( T_M, MT_Q \), and \( WT_Q \), the total travel time for 9 flow profiles, the expected travel time, the standard deviation of travel time, and the travel time distribution

\( T_M \) are calculated based on the trajectory of the vehicles \( WT_Q \): is approximately calculated as half of the red time according to shockwave analysis. \( MT_Q \): is based on the queue spacing and the acceleration rate. Travel times can be calculated for 9 different flow profiles. The travel times are shown in Table 3-2.

The expected travel time can be calculated for the three different groups of vehicles that arrive in different conditions at intersection 1. The travel times, the associated probabilities, and the expected travel times are shown in Table 3-3.

\[
E(TT_{\text{Cond1}}) = TT_{11} \times \text{Scenario}_{11} \% + TT_{12} \times \text{Scenario}_{12} \% + TT_{13} \times \text{Scenario}_{13} \%
\]
\[
E(TT_{\text{Cond2}}) = TT_{21} \times \text{Scenario}_{21} \% + TT_{22} \times \text{Scenario}_{22} \% + TT_{23} \times \text{Scenario}_{23} \%
\]
\[
E(TT_{\text{Cond3}}) = TT_{31} \times \text{Scenario}_{31} \% + TT_{32} \times \text{Scenario}_{32} \% + TT_{33} \times \text{Scenario}_{33} \%
\]
The total expected travel time is calculated as follows:

\[
E(TT) = TT_{Cond1} \times Cond1\% + TT_{Cond2} \times Cond2\% + TT_{Cond3} \times Cond3\%
\]
\[
E(TT) = 41.29 \times 0.41 + 32.29 \times 0.59 = 35.94 \text{ sec}
\]

The standard deviation of travel time is

\[
\sigma = \sqrt{E(x^2) - [E(x)]^2} = \sqrt{\sum p_i x_i^2 - [\sum p_i x_i]^2} = 4.74 \text{ sec}
\]

The travel time distribution is shown in Figure 3-22.

Figure 3-1. Definition of a link

Figure 3-2. Decomposition of travel time
Figure 3-3. The three conditions

\[ s \] : Total length of the link (exclude the length of intersection), ft

\[ s_a \] : Accelerating distance, ft

\[ s_c \] : Constant speed distance, ft

\[ s_p \] : The distance from the end point of S2 till the end of the link, ft

\[ D_q \] : Distance over which the queue extends, ft

Figure 3-4. Link length
Figure 3-5. $T_M$ Equation for Condition 1

Figure 3-6. $T_M$ Equation for Condition 2

Figure 3-7. $T_M$ Equation for Condition 3
Figure 3-8. Shockwave analysis to find the interval lengths for conditions

Figure 3-9. Probabilities of changing from one condition to another condition

Figure 3-10. Study area
Figure 3-11. $D_{q_{\_link0}}$ and $D_{q_{\_link1}}$
Figure 3-12. The trajectories for the nine flow profiles

\( P_{11} \): From Condition 1 to Condition 1
Entering speed for the downstream intersection:
\[ \sqrt{2a_d D_{q\_link0}} \text{ or } V_m \]

\( P_{12} \): From Condition 1 to Condition 2
Entering speed for the downstream intersection:
\[ \sqrt{2a_d D_{q\_link0}} \text{ or } V_m \]

\( P_{13} \): From Condition 1 to Condition 3
Entering speed for the downstream intersection:
\[ \sqrt{2a_d D_{q\_link0}} \text{ or } V_m \]

\( P_{21} \): From Condition 2 to Condition 1
Entering speed for the downstream intersection:
\[ V_{d\_link0} \]

\( P_{22} \): From Condition 2 to Condition 2
Entering speed for the downstream intersection:
\[ V_{d\_link0} \]

\( P_{23} \): From Condition 2 to Condition 3
Entering speed for the downstream intersection:
\[ V_{d\_link0} \]

\( P_{31} \): From Condition 3 to Condition 1
Entering speed for the downstream intersection:
\[ V_m \]

\( P_{32} \): From Condition 3 to Condition 2
Entering speed for the downstream intersection:
\[ V_m \]

\( P_{33} \): From Condition 3 to Condition 3
Entering speed for the downstream intersection:
\[ V_m \]
Figure 3-13. Time and space diagram

Figure 3-14. Residual queue due to Condition 2 is terminated
Figure 3-15. Increasing residual queue due to terminated Condition 2

Figure 3-16. Residual queue calculation when Condition 2 is terminated

Figure 3-17. Average queue length under congested conditions
Figure 3-18. The discharged queue

Figure 3-19. Flow at downstream intersection

Figure 3-20. Sketch for signalized arterial

Figure 3-21. Flow-density-speed relationship
Figure 3-22. Travel time distribution for the example

Table 3-1. Inputs and outputs for travel time estimation model

<table>
<thead>
<tr>
<th>Input</th>
<th>Output</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_{i,\text{link}1} ), capacity, density</td>
<td>( P_{11}, P_{12}, P_{13}, P_{21}, P_{22}, P_{23}, P_{31}, P_{32}, P_{33} )</td>
</tr>
<tr>
<td>( r ), Cycle _length</td>
<td>( tt_{11}, tt_{12}, tt_{13}, tt_{21}, tt_{22}, tt_{23}, tt_{31}, tt_{32}, tt_{33} )</td>
</tr>
<tr>
<td>( s )</td>
<td>Expected travel time, variance of travel time, distribution of travel time</td>
</tr>
<tr>
<td>( V_{i,\text{link}1}, V_m )</td>
<td></td>
</tr>
<tr>
<td>( a_v, a_d )</td>
<td></td>
</tr>
<tr>
<td>( D_{q,\text{link}0}, D_{q,\text{link}1} )</td>
<td></td>
</tr>
<tr>
<td>( WT_Q )</td>
<td></td>
</tr>
</tbody>
</table>

Table 3-2. Probabilities for three conditions at intersection 1 and 2

<table>
<thead>
<tr>
<th>Intersection</th>
<th>( \text{Pr}{1} )</th>
<th>( \text{Pr}{2} )</th>
<th>( \text{Pr}{3} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intersection 1</td>
<td>0.335</td>
<td>0.173</td>
<td>0.492</td>
</tr>
<tr>
<td>Intersection 2</td>
<td>0.337</td>
<td>0.179</td>
<td>0.484</td>
</tr>
<tr>
<td></td>
<td>Probability</td>
<td>TT</td>
<td>Expected TT</td>
</tr>
<tr>
<td>----------</td>
<td>-------------</td>
<td>-----</td>
<td>-------------</td>
</tr>
<tr>
<td>TT11</td>
<td>0.41</td>
<td>41.29</td>
<td></td>
</tr>
<tr>
<td>TT12</td>
<td>0.00</td>
<td>32.32</td>
<td>41.29</td>
</tr>
<tr>
<td>TT13</td>
<td>0.00</td>
<td>30.03</td>
<td></td>
</tr>
<tr>
<td>TT21</td>
<td>0.00</td>
<td>46.66</td>
<td></td>
</tr>
<tr>
<td>TT22</td>
<td>0.00</td>
<td>32.84</td>
<td>0</td>
</tr>
<tr>
<td>TT23</td>
<td>0.00</td>
<td>30.55</td>
<td></td>
</tr>
<tr>
<td>TT31</td>
<td>0.00</td>
<td>46.46</td>
<td></td>
</tr>
<tr>
<td>TT32</td>
<td>0.59</td>
<td>32.29</td>
<td>32.29</td>
</tr>
<tr>
<td>TT33</td>
<td>0.00</td>
<td>30.00</td>
<td></td>
</tr>
</tbody>
</table>
Chapter 4 describes the data collection for this research. The objective of data collection is to collect field travel times as well as other traffic data to validate the analytical model.

4.1 Travel Time Data Collection

The data collection helps validate the analytical model. Field travel time can be obtained and compared to the estimated travel time from the analytical model.

There are 4 sites where field travel time data were collected.

As shown in Figure 4-1, one existing data set from Gainesville, FL. (Newberry Road) is used. This data was provided by Dr. Washburn et al. (2006). Data were collected on the arterial segment which includes five intersections. The total length of the segment is 1,883 ft. Four cameras were placed along the segment; they are facing to the east. Travel time and volume data used in this research were collected on 4/30/2005 and 5/1/2005.

Travel time data were collected based on recorded videos. The method used was to record the time when one particular vehicle enters the first intersection and when it leaves the last intersection. Table 4-1 shows the number of travel time runs that were conducted.

Another 3 sites for data collection are Beaver_Pugh (Figure 4-2), Beaver_Sparks (Figure 4-3), and Park (Figure 4-4) in State College, Pennsylvania. Please note that the three sketches are not to scale.

On Beaver_Pugh (Figure 4-2) and Beaver_Sparks (Figure 4-3), data were collected for a link between two signalized intersections. The link length for Beaver_Pugh is 1263 ft, and that of Beaver_Sparks is 1429 ft. The two arterial segments all have two lanes, one-way traffic. Travel time data were collected during morning peak hour (7:30 am – 8:30 am), midday (12:30 pm-1:30 pm), and the pm peak hour (5:00pm-6:00pm) in May 2004.
On Park Ave (Figure 4-4), data were collected for two consecutive links between three consecutive intersections. The link length between North Atherton Street and Allen Street is 1420 ft, the link length between Allen Street and Grove Alley Street is 1471 ft. The arterial has two-way traffic. There is one lane for each direction. Travel time data were collected during the midday (12:30 pm-1:30 pm) and the pm peak hour (5:00pm-6:00pm) in May 2004.

Travel time data were collected through probe vehicles. The method used was to record the time when the probe vehicle enters the first intersection and when it leaves the last intersection. In each run, the probe vehicle traveled along the arterial with the speed limit. When speed was lower than 5 mph below the speed limit, the vehicle was considered to be delayed. The total travel time and the delay time were measured. The travel time estimated by the model will be compared to the field data and the model will be refined as appropriate. Table 4-2 shows the number of travel time runs that were conducted.

4.2 Other Traffic Data Collection

Other data were collected and will be used as inputs for the model. They are:

- Link length;
- Number of Lanes (full lanes, turning bays);
- Traffic control (signal timing plan, offset);
- Traffic volume; and
- Maximum operating speed (the speed limit).

The data were collected the same time while the travel times were collected at each site.
Figure 4-1. Sketch of site at Newberry Rd.

Figure 4-2. Sketch for site Beaver_Pugh

Figure 4-3. Sketch for site Beaver_Sparks

Figure 4-4. Sketch for site Park

Table 4-1. Number of samples collected for travel time data from Newberry Rd.

<table>
<thead>
<tr>
<th>Number of samples</th>
<th>4/30/2005</th>
<th>4/31/2005</th>
</tr>
</thead>
<tbody>
<tr>
<td>EB</td>
<td>36</td>
<td>48</td>
</tr>
<tr>
<td>WB</td>
<td>38</td>
<td>31</td>
</tr>
</tbody>
</table>
Table 4-2. Number of samples for travel time data for Beaver_Pugh, Beaver_Sparks, and Park

<table>
<thead>
<tr>
<th>Number of samples</th>
<th>Beaver_Pugh</th>
<th>Beaver_Sparks</th>
<th>Park</th>
</tr>
</thead>
<tbody>
<tr>
<td>EB_AM</td>
<td>8</td>
<td>8</td>
<td>N/A</td>
</tr>
<tr>
<td>EB_MID</td>
<td>7</td>
<td>7</td>
<td>7</td>
</tr>
<tr>
<td>EB_PM</td>
<td>7</td>
<td>7</td>
<td>5</td>
</tr>
<tr>
<td>WB_AM</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>WB_MID</td>
<td>N/A</td>
<td>N/A</td>
<td>7</td>
</tr>
<tr>
<td>WB_PM</td>
<td>N/A</td>
<td>N/A</td>
<td>5</td>
</tr>
</tbody>
</table>

Note: N/A means the data are not available for the case
The analytical model is based on pre-timed signal timing. However, the signal timings from the field are all semi-actuated and these signal timings cannot be directly used in the model. Thus, they need to be approximated to pre-timed signal timing for the model validation.

Therefore, there are two steps in the model validation process. The first is to simulate the real traffic conditions in AIMSUN and compare the simulated travel times to the field travel times. This step is to make sure that AIMSUN can replicate field conditions accurately. The second step is to adjust AIMSUN to approximate the semi-actuated to pre-timed signal timings and compare the travel times from model estimation to simulation.

5.1 Simulate the Real Traffic Condition and Compare to the Field Data

AIMSUN is used as the simulation tool in this research. There are two steps in this part: first, replicate the real traffic conditions with the semi-actuated signal timing; second, compare the simulated travel times to the field travel times to make sure AIMSUN can simulate accurately.

5.1.1 Replicate the Real Traffic Conditions with Semi-Actuated Signal Timing

There are 4 sites that field travel time data were collected. Among the four sites, data were collected during different time periods for EB and/or WB traffic. Thus, each time period for each direction in each site is counted as one case. There are total fourteen cases and their signal timings are all semi-actuated.

5.1.1.1 Site 1: Beaver_Pugh

Site 1 is the Beaver Ave. between Pugh St. and Garner St. in State College, PA. Travel times were collected during three time periods AM, Mid and PM. Since the segment is one-way arterial, the travel times collected are only for EB. Thus, the 3 cases for Site 1 are:
Case 1: Beaver_Pugh_AM_EB.
Case 2: Beaver_Pugh_Mid_EB.
Case 3: Beaver_Pugh_PM_EB.

The signal timings for the 3 cases of the site are shown in Figure 5-1, Figure 5-2, and Figure 5-3.

5.1.1.2 Site 2: Beaver_Sparks

Site 2 is the Beaver Ave. between Sparks St. and Atherton St. in State College, PA. Travel times were collected during three time periods AM, Mid and PM. Since the segment is one-way arterial, the travel times collected are only for EB. Thus, the 3 cases for Site 2 are:

- Case 4: Beaver_Sparks_AM_EB.
- Case 5: Beaver_Sparks_Mid_EB.
- Case 6: Beaver_Sparks_PM_EB.

The signal timings for the 3 cases of the site are shown in Figure 5-4, Figure 5-5, and Figure 5-6.

5.1.1.3 Site 3: Park

Site 3 is the Park Ave. between Atherton St. and Grove Alley St. in State College, PA. Travel times were collected during two time periods Mid and PM for both EB and WB. Thus, the 4 cases for Site 3 are:

- Case 7: Park_Mid_EB.
- Case 8: Park_Mid_WB.
- Case 9: Park_PM_EB.
- Case 10: Park_PM_WB.

The signal timings for the Case 7 and 8 is are shown in Figure 5-7 and that for Case 9 and 10 is shown in Figure 5-8.
5.1.1.4 Site 4: Newberry rd.

Site 4 is the Newberry Road between I-75 exit ramp and the NW 66th St. in Gainesville, FL. Travel times were collected during two time periods April 30th, 2005 and May 1st, 2005 for both EB and WB. Thus, the 4 cases for Site 4 are:

- Case 11: Newberry_4_30_EB.
- Case 12: Newberry_4_30_WB.
- Case 13: Newberry_5_1_EB.
- Case 14: Newberry_5_1_WB.

The signal timings for the two cases from May 1st of the site are the same as the one for April 30th. The signal timings for the 4 cases are shown in Figure 5-9.

Besides signal timings, other traffic characteristics were also collected for each site. They are listed in Chapter 4.

5.1.2 Simulate the Travel Times and Compare with Filed Travel Times

AIMSUN NG 5.0 is used to replicate the real traffic condition. The simulated travel times cannot be the exact same value as the field travel times. However, they have to be statistically the same.

To validate the simulated travel times equal to the field travel times, error tolerances are calculated and the acceptable travel time intervals are estimated based on the tolerances:

\[ e = \sqrt{\frac{z^2 s^2}{n}} = \frac{zs}{\sqrt{n}} \]  (5-1)

Where
- \( e \): tolerance
- \( s \): the standard deviation
- \( n \): the number of samples

The \( z \) value is determined by the confidence interval (CI). If the CI is 90\%, the \( z \) value is 1.645, if the CI is 95\%, the \( z \) value is 1.96. For this research, the CI is set to 95\%. 

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Based on the samples that were collected, the tolerances and the acceptable travel times for each scenario are calculated. They are presented in Table 5-1.

From Table 5-1, if the travel time from the simulator is within the acceptable travel time range, it is considered an acceptable simulated travel time. For some of the cases, the tolerance is high compared to other cases. This is because the travel time varies significantly during the field travel time collection. Fourteen cases are simulated in AIMSUN. Each of them is simulated for 1 hour, and travel times for the stream (between the left-most intersection and the right-most intersection) are reported for every 10 minutes for un-congested conditions and every cycle for congested conditions. Thus, if the site is not congested, in the 1 hour simulation period, 6 travel times are reported. 10 replications are made for each of the cases and travel time is averaged based on all replications. If it is congested, travel times were collected for each cycle and for 10 replications before spillback occurs.

The simulated travel times for each of the cases are listed in Table 5-2, compared to the field data. Among all simulations, only the simulated Beaver_Pugh_Mid_EB does not statistically equal the field travel time. Because this case cannot be accurately replicated in simulation, this case is removed from model validation.

5.2 Compare the Model Estimation to the Simulation

There are two steps in this part. First, approximate the semi-actuated to pre-timed signal timing and replace them in simulation. Second, compare the model estimated travel times to the simulated travel times to make sure AIMSUN can simulate accurately.

5.2.1 Replicate with Pre-timed Signal Timing

In this research, the travel time estimation model is based on pre-timed signal timing. However, travel times are collected on arterials with semi-actuated signal timing. Thus, the semi-
actuated signal timings have to be approximated to pre-timed signal timing in order to test the
capability of the analytical model.

In order to obtain appropriate signal timings, there are several criteria in transforming the
signal plans:

- Cycle length of all intersections have to be same in order to use offset in model estimation:
  the method is to get the sum of the green times for all intersections, use the longest cycle as
  the standard cycle length, and rescale all green times from smaller cycles so that the new
  cycle length after the rescale is the same as the longest cycle length.

- Use the minimum green interval in actuated signal timing as the green interval for phases
  that have minimum recall, and use the maximum green interval as the green interval for
  phases that have do not have minimum recall.

- Dual ring signal timings have to be changed to single ring signal timing since it is beyond
  the ability of the analytical model.

### 5.2.1.1 Site 1: Beaver_Pugh

The approximate signal timings for Site 1 are shown in Figure 5-10 (for Case 1) and Figure
5-11 (for Case 3). According to the criteria, the green times for one intersection in Case
1(Beaver_Pugh_AM_EB) are changed because the cycle lengths of the two intersections are
different in the field.

### 5.2.1.2 Site 2: Beaver_Sparks

The approximate signal timings for Site 2 are shown in Figure 5-12 (for Case 4), Figure
5-13 (for Case 5), and Figure 5-14 (for Case 6). According to the criteria, the green times for one
intersection in Case 5 (Beaver_Sparks_Mid_EB) are changed because the cycle lengths of the
two intersections are different.

### 5.2.1.3 Site 3: Park

The approximate signal timings for Site 3 are shown in Figure 5-15 (for Case 7 and
Case 8) and Figure 5-16 (for Case 9 and Case 10).
5.2.1.4 Site 4: Newberry Rd.

The approximate signal timings for Site 4 are shown in Figure 5-17 (for Case 11 and Case 13) and Figure 5-18 (for Case 12 and Case 14). Case 11 and 13 are for EB, Case 12 and 14 are for WB. In Figure 5-17, for the second intersection, the EB through movement has green time all the time. Thus, intersection 2 does not exist for EB, it works as same as an uncontrolled intersection. The third and the fifth intersection have dual ring phasing. As shown in Figure 5-17, the two phases inside one dual ring are combined into one single phase. The green time for the new phase is determined by averaging the green times from the two phases. In Figure 5-18, the second intersection exists since the WB through movement does not have green time all the time. The rest of the intersections are approximated the same way as in Figure 5-17.

This step is to create a set of inputs that are applicable for both the simulation and the analytical model. The heavy vehicle volume is not considered in the analytical model. Thus, the volume of heavy vehicles is replaced on equal volume of passenger cars.

5.2.2 Comparisons and Conclusions

In this section, the pre-timed signal timings are used to obtain simulated travel times from AIMSUN. The simulated travel time is then compared to the model estimates. Travel times are obtained both from AIMSUN and the analytical model for different offsets. The offset starts from 0 seconds and is increased by 10 seconds until it reaches the cycle length of the case. The results are shown in Tables 5-3 to Table 5-15 for each case. The tables provide the average travel time from the simulation and the model, the standard deviation as estimated in the model estimated travel time, the percent difference between the average travel time from the simulation and the model, and the travel time estimated by the HCM 2000 travel time model. The travel time distributions of Case 1(Beaver_Pugh_AM_EB) are shown in Figure 5-19 to Figure 5-21 as a demonstration of how travel times are distributed as the offset changes.
The comparisons between the average simulated travel time and the average model estimated travel time are shown in figures 5-22 to Figure 5-34. Among all the cases, only Case 10 (Park_PM_WB) is a congested case. Unlike the other cases which the travel time is calculated for each offset, its travel time is calculated cycle by cycle before spillback occurs. The queue length changes every cycle and travel time changes with it. The detailed queue length calculation is shown in Figure 5-35.

Observations based on these comparisons are summarized below:

**Generally the trends in travel times from the analytical model are comparable with the simulation.**

The percent difference between the simulated and the model-estimated travel times are shown in tables Table 5-3 to 5-15. As shown in the tables, the percent difference for Beaver_Pugh site, Beaver_Sparks site, and Park site are smaller than that of the Newberry cases. The analytical model does not give good results when multiple driveways are present.

The percent difference for Case 11, 12, 13, and 14 are shown in Table 5-12, Table 5-13, Table 5-14, and Table 5-15. As shown in Figure 5-36, the likely reason for that is the driveway densities at these sites compared to other sites are much higher. Vehicles often have to slow down or even stop prior to the driveway to avoid blocking the entrance/exit at the driveways and this affects the travel time. Travel time estimation in the analytical model does not consider the deceleration and acceleration at driveways. However, the microscopic simulation records the travel time of each vehicle that goes through the corridor and the delays at the driveways are considered. Thus, the travel times are quite different.

**The travel times from the model do not change smoothly like the ones from the simulation.**
In the analytical model estimation, the travel time changes as the offset changes. Travel time is calculated as the expected travel time of all selected travel times. If the offset increases by 10 seconds, in some of the circumstances it does not change the set of selected travel times. Thus, travel times sometimes remain the same for several consecutive offsets.

| Intersection of Beaver Ave. and S. Pugh St. | Max green: 100 sec  
Min green: 18 sec  
Y: 3 sec  
AR: 1 sec |
|--------------------------------------------|--------------------------------------------------|
| Yield Point: 20 sec  
Cycle Length: 45 sec | Max green: 18 sec  
Min green: 7 sec  
Y: 3 sec  
AR: 2 sec |

| Intersection of Beaver Ave. and Garner St. | Max green: 100 sec  
Min green: 54 sec  
Y: 3.5 sec  
AR: 1.5 sec |
|------------------------------------------|--------------------------------------------------|
| Yield Point: 56 sec  
Cycle Length: 90 sec | Max green: 26 sec  
Min green: 7 sec  
Y: 3.4 sec  
AR: 1.6 sec |

Figure 5-1. Semi-actuated signal timing for Case 1 (Beaver_Pugh_AM_EB)

| Intersection of Beaver Ave. and S. Pugh St. | Max green: 100 sec  
Min green: 37 sec  
Y: 3 sec  
AR: 1 sec |
|--------------------------------------------|--------------------------------------------------|
| Yield Point: 3 sec  
Cycle Length: 80 sec | Max green: 34 sec  
Min green: 7 sec  
Y: 3 sec  
AR: 2 sec |

| Intersection of Beaver Ave. and Garner St. | Max green: 100 sec  
Min green: 47 sec  
Y: 3.5 sec  
AR: 1.5 sec |
|------------------------------------------|--------------------------------------------------|
| Yield Point: 33 sec  
Cycle Length: 80 sec | Max green: 23 sec  
Min green: 7 sec  
Y: 3.4 sec  
AR: 1.6 sec |

Figure 5-2. Semi-actuated signal timing for Case 2 (Beaver_Pugh_Mid_EB)
<table>
<thead>
<tr>
<th>Intersection</th>
<th>Max green: 100 sec</th>
<th>Min green: 54 sec</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Y: 3 sec</td>
</tr>
<tr>
<td></td>
<td></td>
<td>AR: 1 sec</td>
</tr>
<tr>
<td>Max green: 37 sec</td>
<td></td>
<td>Min green: 7 sec</td>
</tr>
<tr>
<td>Min green: 7 sec</td>
<td></td>
<td>Y: 3 sec</td>
</tr>
<tr>
<td>Y: 3 sec</td>
<td></td>
<td>AR: 2 sec</td>
</tr>
<tr>
<td>AR: 1 sec</td>
<td></td>
<td>Max green: 32 sec</td>
</tr>
<tr>
<td>Min green: 7 sec</td>
<td></td>
<td>Y: 3.4 sec</td>
</tr>
<tr>
<td>Y: 3.4 sec</td>
<td></td>
<td>AR: 1.6 sec</td>
</tr>
</tbody>
</table>

Figure 5-3. Semi-actuated signal timing for Case 3 (Beaver_Pugh_PM_EB)

<table>
<thead>
<tr>
<th>Intersection</th>
<th>Max green: 100 sec</th>
<th>Min green: 56 sec</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Y: 3 sec</td>
</tr>
<tr>
<td></td>
<td></td>
<td>AR: 1 sec</td>
</tr>
<tr>
<td>Max green: 26 sec</td>
<td></td>
<td>Min green: 3 sec</td>
</tr>
<tr>
<td>Min green: 3 sec</td>
<td></td>
<td>Y: 3 sec</td>
</tr>
<tr>
<td>Y: 3 sec</td>
<td></td>
<td>AR: 1 sec</td>
</tr>
<tr>
<td>AR: 1 sec</td>
<td></td>
<td>Max green: 36 sec</td>
</tr>
<tr>
<td>Min green: 7 sec</td>
<td></td>
<td>Y: 3 sec</td>
</tr>
<tr>
<td>Y: 3 sec</td>
<td></td>
<td>AR: 1 sec</td>
</tr>
</tbody>
</table>

Figure 5-4. Semi-actuated signal timing for Case 4 (Beaver_Sparks_AM_EB)
<table>
<thead>
<tr>
<th>Intersection of Beaver Ave. and S. Sparks St.</th>
<th>Max green: 100 sec</th>
<th>Min green: 32 sec</th>
<th>Y: 3 sec</th>
<th>AR: 1 sec</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield Point: 1 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cycle Length: 65 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Intersections of Beaver Ave. and S. Sparks St.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Yield Point: 25 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cycle Length: 90 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max green: 100 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Min green: 25 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Y: 3 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>AR: 1 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max green: 25 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Min green: 3 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Y: 3 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>AR: 1 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 5-5. Semi-actuated signal timing for Case 5 (Beaver_Sparks_Mid_EB)

<table>
<thead>
<tr>
<th>Intersection of Beaver Ave. and S. Sparks St.</th>
<th>Max green: 100 sec</th>
<th>Min green: 32 sec</th>
<th>Y: 3 sec</th>
<th>AR: 1 sec</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield Point: 71 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cycle Length: 100 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Intersections of Beaver Ave. and S. Sparks St.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Yield Point: 32 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cycle Length: 90 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max green: 100 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Min green: 17 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Y: 3 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>AR: 0 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max green: 100 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Min green: 38 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Y: 3 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>AR: 1 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max green: 37 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Min green: 7 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Y: 3 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>AR: 1 sec</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 5-6. Semi-actuated signal timing for Case 6 (Beaver_Sparks_PM_EB)
| Intersection of Park Ave. and Atherton St. | Max green: 22 sec  
Min green: 2 sec  
Y: 3 sec  
AR: 0 sec |
|------------------------------------------|---------------------------------------------------|
| Yield Point: 86 sec  
Cycle Length: 100 sec | ![Signal Phases] |
| Max green: 100 sec  
Min green: 38 sec  
Y: 3 sec  
AR: 2 sec |
| Max green: 7 sec  
Min green: 2 sec  
Y: 3 sec  
AR: 0 sec |
| Max green: 4 sec  
Min green: 4 sec  
Y: 3 sec  
AR: 0 sec |
| Max green: 43 sec  
Min green: 23 sec  
Y: 3 sec  
AR: 1 sec |
| Max green: 24 sec  
Min green: 2 sec  
Y: 3 sec  
AR: 1 sec |
| Max green: 7 sec  
Min green: 2 sec  
Y: 3 sec  
AR: 0 sec |
| Max green: 45 sec  
Min green: 16 sec  
Y: 3 sec  
AR: 2 sec |
| Max green: 20 sec  
Min green: 2 sec  
Y: 3 sec  
AR: 2 sec |

Figure 5-7. Semi-actuated signal timing for Case 7 (Park_Mid_EB) and Case 8 (Park_Mid_WB)
| Intersection of Park Ave. and Atherton St. | Max green: 19 sec  
Min green: 2 sec  
Y: 3 sec  
AR: 0 sec |
|-------------------------------------------|--------------------------------------------------|
| Yield Point: 68 sec  
Cycle Length: 100 sec | Max green: 100 sec  
Min green: 44 sec  
Y: 3 sec  
AR: 2 sec |
| Max green: 4 sec  
Min green: 4 sec  
Y: 3 sec  
AR: 0 sec | Max green: 17 sec  
Min green: 2 sec  
Y: 3 sec  
AR: 0 sec |
| Max green: 11 sec  
Min green: 2 sec  
Y: 3 sec  
AR: 2 sec | Max green: 43 sec  
Min green: 23 sec  
Y: 3 sec  
AR: 1 sec |
| Max green: 20 sec  
Min green: 2 sec  
Y: 3 sec  
AR: 0 sec | Max green: 7 sec  
Min green: 2 sec  
Y: 3 sec  
AR: 0 sec |
| Max green: 39 sec  
Min green: 16 sec  
Y: 3 sec  
AR: 2 sec | Max green: 26 sec  
Min green: 2 sec  
Y: 3 sec  
AR: 2 sec |

**Figure 5-8.** Semi-actuated signal timing for Case 9 (Park_PM_EB) and Case 10 (Park_PM_WB)
| Intersection of Newberry Rd. and I-75 NB off ramp (exit) |  | Max green: 22 sec  
Min green: 4 sec  
Y: 3 sec  
AR: 2 sec |
|---|---|---|
| Yield Point: 122 sec  
Cycle Length: 150 sec |  | Max green: 200 sec  
Min green: 118 sec  
Y: 3 sec  
AR: 2 sec |
| Intersection of Newberry Rd. and I-75 NB off ramp (entrance) |  | Max green: 22 sec  
Min green: 4 sec  
Y: 3 sec  
AR: 2 sec |
| Yield Point: 122 sec  
Cycle Length: 150 sec |  | Max green: 37 sec  
Min green: 6 sec  
Y: 3 sec  
AR: 2 sec |
| Intersection of Newberry Rd. and NW 69th Street |  | Max green: 10 sec  
Min green: 4 sec  
Y: 3 sec  
AR: 2 sec  
Max green: 28 sec  
Min green: 4 sec  
Y: 3 sec  
AR: 2 sec |
| Yield Point: 121 sec  
Cycle Length: 150 sec |  | Max green: 150 sec  
Min green: 90 sec  
Y: 3 sec  
AR: 2 sec  
Max green: 100 sec  
Min green: 72 sec  
Y: 3 sec  
AR: 2 sec |
| Intersection of Newberry Rd. and Oaks Mall West |  | Max green: 16 sec  
Min green: 4 sec  
Y: 3 sec  
AR: 2 sec  
Max green: 150 sec  
Min green: 72 sec  
Y: 3 sec  
AR: 2 sec |
| Yield Point: 106 sec  
Cycle Length: 150 sec |  | Max green: 14 sec  
Min green: 4 sec  
Y: 3 sec  
AR: 2 sec  
Max green: 28 sec  
Min green: 4 sec  
Y: 3 sec  
AR: 2 sec |

Figure 5-9. Semi-actuated signal timing for Case 11 (Newberry_4_30_EB) Case 12 (Newberry_4_30 WB) Case 13 (Newberry_5_1_EB) Cases 14(Newberry_5_1 WB)
<table>
<thead>
<tr>
<th>Intersection of Newberry Rd. and NW 66th Street</th>
<th>Max green: 16 sec</th>
<th>Max green: 29 sec</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield Point: 106 sec</td>
<td>Min green: 4 sec</td>
<td>Min green: 4 sec</td>
</tr>
<tr>
<td>Cycle Length: 150 sec</td>
<td>Y: 3 sec</td>
<td>Y: 3 sec</td>
</tr>
<tr>
<td></td>
<td>AR: 2 sec</td>
<td>AR: 2 sec</td>
</tr>
<tr>
<td>Max green: 31 sec</td>
<td>Max green: 150 sec</td>
<td></td>
</tr>
<tr>
<td>Min green: 4 sec</td>
<td>Min green: 72 sec</td>
<td></td>
</tr>
<tr>
<td>Y: 3 sec</td>
<td>Y: 3 sec</td>
<td></td>
</tr>
<tr>
<td>AR: 2 sec</td>
<td>AR: 2 sec</td>
<td></td>
</tr>
<tr>
<td>Max green: 11 sec</td>
<td>Max green: 19 sec</td>
<td></td>
</tr>
<tr>
<td>Min green: 4 sec</td>
<td>Min green: 4 sec</td>
<td></td>
</tr>
<tr>
<td>Y: 3 sec</td>
<td>Y: 3 sec</td>
<td></td>
</tr>
<tr>
<td>AR: 2 sec</td>
<td>AR: 2 sec</td>
<td></td>
</tr>
</tbody>
</table>

**Figure 5-9. Continued**

<table>
<thead>
<tr>
<th>Intersection of Beaver Ave. and S. Pugh St.</th>
<th>Green: 40 sec</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cycle Length: 90 sec</td>
<td>Yellow: 3 sec</td>
</tr>
<tr>
<td></td>
<td>All Red: 2 sec</td>
</tr>
<tr>
<td>Green: 40 sec</td>
<td></td>
</tr>
<tr>
<td>Yellow: 3 sec</td>
<td></td>
</tr>
<tr>
<td>All Red: 2 sec</td>
<td></td>
</tr>
</tbody>
</table>

**Figure 5-10. Approximate A.M. signal timing for Case 1 (Beaver_Pugh_AM_EB)**
| Intersection of Beaver Ave. and S. Pugh St. | Cycle Length: 100 sec | Green: 54 sec  
Yellow: 3 sec  
All Red: 1 sec  
Green: 37 sec  
Yellow: 3 sec  
All Red: 2 sec |
| --- | --- | --- |
| Intersection of Beaver Ave. and Garner St. | Cycle Length: 100 sec | Green: 58 sec  
Yellow: 4 sec  
All Red: 1 sec  
Green: 32 sec  
Yellow: 4 sec  
All Red: 1 sec |
| Intersection of Beaver Ave. and S. Sparks St. | Cycle Length: 90 sec | Green: 56 sec  
Yellow: 3 sec  
All Red: 2 sec  
Green: 26 sec  
Yellow: 3 sec  
All Red: 2 sec |
| Intersection of Beaver Ave. and Atherton St. | Cycle Length: 90 sec | Green: 12 sec  
Yellow: 3 sec  
All Red: 0 sec  
Green: 31 sec  
Yellow: 3 sec  
All Red: 1 sec  
Green: 36 sec  
Yellow: 3 sec  
All Red: 1 sec |

Figure 5-11. Approximate P.M. signal timing for Case 3 (Beaver_Pugh_PM_EB)

Figure 5-12. Approximate A.M. signal timing for Case 4 (Beaver_Sparks_AM_EB)
| Intersection of Beaver Ave. and S. Sparks St. | Green: 52sec  
Yellow: 3 sec  
All Red: 1 sec |
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cycle Length: 90 sec</td>
<td></td>
</tr>
<tr>
<td></td>
<td>![Signal Timing Diagram]</td>
</tr>
</tbody>
</table>
|                                             | Green: 40sec  
Yellow: 3 sec  
All Red: 1 sec |
|                                             | ![Signal Timing Diagram]                        |
|                                             | ![Signal Timing Diagram]                        |
|                                             | ![Signal Timing Diagram]                        |
| Intersection of Beaver Ave. and Atherton St. | Green:17 sec  
Yellow: 3 sec  
All Red: 0 sec  |
| Cycle Length: 90 sec                        |                                                 |
|                                             | ![Signal Timing Diagram]                        |
|                                             | Green:38 sec  
Yellow: 3 sec  
All Red: 1 sec |
|                                             | ![Signal Timing Diagram]                        |
|                                             | ![Signal Timing Diagram]                        |
|                                             | ![Signal Timing Diagram]                        |
| Intersection of Beaver Ave. and S. Sparks St. | Green: 60sec  
Yellow: 3 sec  
All Red: 1 sec |
| Cycle Length: 100 sec                       |                                                 |
|                                             | ![Signal Timing Diagram]                        |
|                                             | Green: 32sec  
Yellow: 3 sec  
All Red: 1 sec |
|                                             | ![Signal Timing Diagram]                        |
|                                             | ![Signal Timing Diagram]                        |
|                                             | ![Signal Timing Diagram]                        |
| Intersection of Beaver Ave. and Atherton St. | Green:16 sec  
Yellow: 3 sec  
All Red: 0 sec |
| Cycle Length: 100 sec                       |                                                 |
|                                             | ![Signal Timing Diagram]                        |
|                                             | Green:36 sec  
Yellow: 3 sec  
All Red: 1 sec |
|                                             | ![Signal Timing Diagram]                        |
|                                             | ![Signal Timing Diagram]                        |
|                                             | ![Signal Timing Diagram]                        |
| Intersection of Beaver Ave. and S. Sparks St. | Green: 37 sec  
Yellow: 3 sec  
All Red: 1 sec |
| Cycle Length: 100 sec                       |                                                 |
|                                             | ![Signal Timing Diagram]                        |

Figure 5-13. Approximate Mid signal timing for Case 5 (Beaver_Sparks_Mid_EB)

Figure 5-14. Approximate P.M. signal timing for Case 6 (Beaver_Sparks_PM_EB)
| Intersection of Park Ave. and Atherton St. | Cycle Length: 100 sec | Green: 22 sec  
Yellow: 3 sec  
All Red: 0 sec |
|----------------------------------------|------------------------|-------------------|
|                                        |                        | Green: 38 sec  
Yellow: 3 sec  
All Red: 2 sec |
|                                        |                        | Green: 27sec  
Yellow: 3 sec  
All Red: 2 sec |
| Intersection of Park Ave. and Allen St. | Cycle Length: 100 sec | Green: 8 sec  
Yellow: 3 sec  
All Red: 0 sec |
|                                        |                        | Green: 52 sec  
Yellow: 3 sec  
All Red: 1 sec |
|                                        |                        | Green: 29sec  
Yellow: 3 sec  
All Red: 1 sec |
| Intersection of Park Ave. and Grove Alley St. | Cycle Length: 100 sec | Green: 9 sec  
Yellow: 3 sec  
All Red: 0 sec |
|                                        |                        | Green: 54 sec  
Yellow: 3 sec  
All Red: 2 sec |
|                                        |                        | Green: 24 sec  
Yellow: 3 sec  
All Red: 2 sec |

Figure 5-15. Approximate signal timing for Case 7 (Park_Mid_EB) Case 8 (Park_Mid_WB)
<table>
<thead>
<tr>
<th>Intersection of Park Ave. and Atherton St.</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cycle Length: 100 sec</td>
<td>Green: 19 sec</td>
<td>Yellow: 3 sec</td>
</tr>
<tr>
<td></td>
<td>All Red: 0 sec</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Green: 44 sec</td>
<td>Yellow: 3 sec</td>
</tr>
<tr>
<td></td>
<td>All Red: 2 sec</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Green: 24 sec</td>
<td>Yellow: 3 sec</td>
</tr>
<tr>
<td></td>
<td>All Red: 2 sec</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Intersection of Park Ave. and Allen St.</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cycle Length: 100 sec</td>
<td>Green: 13 sec</td>
<td>Yellow: 3 sec</td>
</tr>
<tr>
<td></td>
<td>All Red: 0 sec</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Green: 52 sec</td>
<td>Yellow: 3 sec</td>
</tr>
<tr>
<td></td>
<td>All Red: 1 sec</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Green: 24 sec</td>
<td>Yellow: 3 sec</td>
</tr>
<tr>
<td></td>
<td>All Red: 1 sec</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Intersection of Park Ave. and Grove Alley St.</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cycle Length: 100 sec</td>
<td>Green: 9 sec</td>
<td>Yellow: 3 sec</td>
</tr>
<tr>
<td></td>
<td>All Red: 0 sec</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Green: 47 sec</td>
<td>Yellow: 3 sec</td>
</tr>
<tr>
<td></td>
<td>All Red: 2 sec</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Green: 31 sec</td>
<td>Yellow: 3 sec</td>
</tr>
<tr>
<td></td>
<td>All Red: 2 sec</td>
<td></td>
</tr>
</tbody>
</table>

Figure 5-16. Approximate signal timing for Case 9 (Park_PM_EB) and Case 10 (Park_PM WB)
| Intersection of Newberry Rd. and I-75 NB off ramp (exit) | Cycle Length: 150 sec | Green: 22 sec  
Y: 3 sec  
AR: 2 sec |
|----------------------------------------------------------|----------------------|--------------------------------------------------|
|                                                          |                      | Green: 118 sec  
Y: 3 sec  
AR: 2 sec |
|                                                          |                      | Green: 105 sec  
Y: 3 sec  
AR: 2 sec |
|                                                          |                      | Green: 35 sec  
Y: 3 sec  
AR: 2 sec |
|                                                          |                      | Green: 93 sec  
Y: 3 sec  
AR: 2 sec |
|                                                          |                      | Green: 14 sec  
Y: 3 sec  
AR: 2 sec |
|                                                          |                      | Max green: 28 sec  
Y: 3 sec  
AR: 2 sec |
|                                                          |                      | Green: 25 sec  
Y: 3 sec  
AR: 2 sec |
|                                                          |                      | Green: 17 sec  
Y: 3 sec  
AR: 2 sec |

Figure 5-17. Approximate signal timing for Case 11 (Newberry_4_30_EB) and Case 13 (Newberry_5_1_EB)
| Intersection of Newberry Rd. and I-75 NB off ramp (exit) | Green: 22 sec Y: 3 sec AR: 2 sec |
| Cycle Length: 150 sec | Green: 118 sec Y: 3 sec AR: 2 sec |
| Intersection of Newberry Rd. and I-75 NB off ramp (entrance) | Green: 22 sec Y: 3 sec AR: 2 sec |
| Cycle Length: 150 sec | Green: 76 sec Y: 3 sec AR: 2 sec |
| Intersection of Newberry Rd. and NW 69th Street | Green: 105 sec Y: 3 sec AR: 2 sec |
| Cycle Length: 150 sec | Green: 35 sec Y: 3 sec AR: 2 sec |
| Intersection of Newberry Rd. and Oaks Mall West | Green: 93 sec Y: 3 sec AR: 2 sec |
| Cycle Length: 150 sec | Green: 14 sec Y: 3 sec AR: 2 sec |
| Intersection of Newberry Rd. and NW 66th Street | Max green: 28 sec Y: 3 sec AR: 2 sec |
| Cycle Length: 150 sec | Green: 93 sec Y: 3 sec AR: 2 sec |

Figure 5-18. Approximate signal timing for Case 12 (Newberry_4_30_WB) and Case 14 (Newberry_5_1_WB)
Figure 5-18. Continued

![Travel Time Distribution for Beaver_Pugh_AM (offset = 0, 10, 20, 30, 80, 90)](image)

Figure 5-19. Travel Time Distribution for Beaver_Pugh_AM (offset = 0, 10, 20, 30, 80, 90)

![Travel Time Distribution for Beaver_Pugh_AM (offset = 40)](image)

Figure 5-20. Travel Time Distribution for Beaver_Pugh_AM (offset = 40)
Figure 5-21. Travel Time Distribution for Beaver_Pugh_AM (offset = 50, 60, 70)

Figure 5-22. Travel time comparisons for Beaver_Pugh_AM EB (case 1)

Figure 5-23. Travel time comparisons for Beaver_Pugh_PM EB (case 3)
Figure 5-24. Travel time comparisons for Beaver_Sparks_AM_EB (case 4)

Figure 5-25. Travel time comparisons for Beaver_Sparks_Mid_EB (case 5)

Figure 5-26. Travel time comparisons for Beaver_Sparks_PM_EB (case 6)
Figure 5-27. Travel time comparisons for Park_Mid_EB (case 7)

Figure 5-28. Travel time comparisons for Park_Mid_WB (case 8)

Figure 5-29. Travel time comparisons for Park_PM_EB (case 9)
Figure 5-30. Travel time comparisons for Park_PM_WB (case 10)

Figure 5-31. Travel time comparisons for Newberry_4_30_EB (case 11)

Figure 5-32. Travel time comparisons for Newberry_4_30_WB (case 12)
Figure 5-33. Travel time comparisons for Newberry_5_1_EB (case 13)

Figure 5-34. Travel time comparisons for Newberry_5_1_WB (case 14)
<table>
<thead>
<tr>
<th>Cycle</th>
<th>Beginning</th>
<th>$Q_M$</th>
<th>$Q_R$</th>
<th>$Q_M + Q_R$</th>
<th>$Q_M + 2Q_R$</th>
<th>$Q_M + 3Q_R$</th>
<th>$Q_M + 4Q_R$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cycle 1</td>
<td>Beginning</td>
<td>$Q_M$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>End</td>
<td>$Q_M - Q_R$</td>
<td>$Q_R$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cycle 2</td>
<td>Beginning</td>
<td>$Q_R$</td>
<td>$Q_M$</td>
<td></td>
<td>$Q_M + Q_R$</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>End</td>
<td>$Q_M - Q_R$</td>
<td>$2Q_R - Q_M$</td>
<td>$Q_M$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cycle 3</td>
<td>Beginning</td>
<td>$2Q_R - Q_M$</td>
<td>$Q_M$</td>
<td>$Q_M$</td>
<td></td>
<td>$Q_M + 2Q_R$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>End</td>
<td>$2Q_R - Q_M$</td>
<td>$2Q_M - 3Q_R$</td>
<td>$3Q_R - Q_M$</td>
<td>$Q_M$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cycle 4</td>
<td>Beginning</td>
<td>$3Q_R - Q_M$</td>
<td>$Q_M$</td>
<td>$Q_M$</td>
<td>$Q_M$</td>
<td>$Q_M + 3Q_R$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>End</td>
<td>$Q_M - Q_R$</td>
<td>$4Q_R - 2Q_M$</td>
<td>$Q_M$</td>
<td>$Q_M$</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 5-35. Queue length calculation for every cycle for Case 10 (Park_PM_WB)
Figure 5-35. Continued
<table>
<thead>
<tr>
<th>Location</th>
<th>Total length:</th>
<th>Total driveway:</th>
<th>Driveway Density:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beaver_Pugh</td>
<td>1263 ft</td>
<td>4</td>
<td>0.00317/mile</td>
</tr>
<tr>
<td>Beaver_Sparks</td>
<td>1429 ft</td>
<td>4</td>
<td>0.00280/mile</td>
</tr>
<tr>
<td>Park</td>
<td>2891 ft</td>
<td>4</td>
<td>0.00138/mile</td>
</tr>
<tr>
<td>Newberry</td>
<td>1883 ft</td>
<td>12</td>
<td>0.00637/mile</td>
</tr>
</tbody>
</table>

Figure 5-36. Link length and driveways
Table 5-1. Tolerances and the acceptable travel times

<table>
<thead>
<tr>
<th>Case</th>
<th>Scenario</th>
<th>Tolerance at 95% CI(sec)</th>
<th>Acceptable travel time at 95% CI(sec)</th>
<th>Tolerance at 90% CI(sec)</th>
<th>Acceptable travel time at 90% CI(sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>Beaver_Pugh_AM_EB</td>
<td>6.14</td>
<td>[30.99,43.26]</td>
<td>5.15</td>
<td>[31.97,42.28]</td>
</tr>
<tr>
<td>Case 2</td>
<td>Beaver_Pugh_Mid_EB</td>
<td>3.13</td>
<td>[32.15,38.42]</td>
<td>2.63</td>
<td>[32.65,37.92]</td>
</tr>
<tr>
<td>Case 3</td>
<td>Beaver_Pugh_PM_EB</td>
<td>4.14</td>
<td>[31.28,39.57]</td>
<td>3.48</td>
<td>[31.95,38.91]</td>
</tr>
<tr>
<td>Case 4</td>
<td>Beaver_Sparks_AM_EB</td>
<td>17.45</td>
<td>[54.18,89.07]</td>
<td>14.64</td>
<td>[56.98,86.27]</td>
</tr>
<tr>
<td>Case 5</td>
<td>Beaver_Sparks_Mid_EB</td>
<td>31.9</td>
<td>[43.53,107.33]</td>
<td>26.77</td>
<td>[48.66,102.2]</td>
</tr>
<tr>
<td>Case 6</td>
<td>Beaver_Sparks_PM_EB</td>
<td>13.9</td>
<td>[84.96,112.76]</td>
<td>11.67</td>
<td>[87.19,110.52]</td>
</tr>
<tr>
<td>Case 7</td>
<td>Park_Mid_EB</td>
<td>10.32</td>
<td>[68.25,88.89]</td>
<td>8.66</td>
<td>[69.91,87.23]</td>
</tr>
<tr>
<td>Case 8</td>
<td>Park_Mid_WB</td>
<td>20.25</td>
<td>[92.04,132.54]</td>
<td>17</td>
<td>[95.29,129.28]</td>
</tr>
<tr>
<td>Case 9</td>
<td>Park_PM_EB</td>
<td>14.15</td>
<td>[74.45,102.75]</td>
<td>11.88</td>
<td>[76.72,100.48]</td>
</tr>
<tr>
<td>Case 10</td>
<td>Park_PM_WB</td>
<td>76.47</td>
<td>[248.73,401.67]</td>
<td>64.18</td>
<td>[261.02,389.38]</td>
</tr>
<tr>
<td>Case 11</td>
<td>Newberry 4_30_EB</td>
<td>6.37</td>
<td>[56.27,69.01]</td>
<td>5.34</td>
<td>[57.3,67.98]</td>
</tr>
<tr>
<td>Case 12</td>
<td>Newberry 4_30_WB</td>
<td>5.42</td>
<td>[48.32,59.16]</td>
<td>4.55</td>
<td>49.19,58.29</td>
</tr>
<tr>
<td>Case 13</td>
<td>Newberry 5_1_EB</td>
<td>7.62</td>
<td>[58.9,74.14]</td>
<td>6.39</td>
<td>[60.13,72.92]</td>
</tr>
<tr>
<td>Case 14</td>
<td>Newberry 5_1_WB</td>
<td>6.18</td>
<td>[52.43,64.79]</td>
<td>5.19</td>
<td>[53.43,63.80]</td>
</tr>
</tbody>
</table>

Note: the averaged simulated travel times are based on 10 replications

Table 5-2. Simulated travel time compared to field travel time

<table>
<thead>
<tr>
<th>Case</th>
<th>Scenario</th>
<th>Average TT from AIMSUN (sec)</th>
<th>Filed average TT (sec)</th>
<th>Acceptable or not</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>Beaver_Pugh_AM_EB</td>
<td>36.67</td>
<td>37.13</td>
<td>Y</td>
</tr>
<tr>
<td><strong>Case 2</strong></td>
<td><strong>Beaver_Pugh_Mid_EB</strong></td>
<td><strong>40.83</strong></td>
<td><strong>35.29</strong></td>
<td>N</td>
</tr>
<tr>
<td>Case 3</td>
<td>Beaver_Pugh_PM_EB</td>
<td>37.83</td>
<td>35.43</td>
<td>Y</td>
</tr>
<tr>
<td>Case 4</td>
<td>Beaver_Sparks_AM_EB</td>
<td>68.00</td>
<td>71.63</td>
<td>Y</td>
</tr>
<tr>
<td>Case 5</td>
<td>Beaver_Sparks_Mid_EB</td>
<td>70.83</td>
<td>75.43</td>
<td>Y</td>
</tr>
<tr>
<td>Case 6</td>
<td>Beaver_Sparks_PM_EB</td>
<td>91.33</td>
<td>98.86</td>
<td>Y</td>
</tr>
<tr>
<td>Case 7</td>
<td>Park_Mid_EB</td>
<td>72.00</td>
<td>78.57</td>
<td>Y</td>
</tr>
<tr>
<td>Case 8</td>
<td>Park_Mid_WB</td>
<td>99.00</td>
<td>112.29</td>
<td>Y</td>
</tr>
<tr>
<td>Case 9</td>
<td>Park_PM_EB</td>
<td>83.5</td>
<td>88.6</td>
<td>Y</td>
</tr>
<tr>
<td>Case 10</td>
<td>Park_PM_WB</td>
<td>330.33</td>
<td>325.2</td>
<td>Y</td>
</tr>
<tr>
<td>Case 11</td>
<td>Newberry 4_30_EB</td>
<td>68.83</td>
<td>62.64</td>
<td>Y</td>
</tr>
<tr>
<td>Case 12</td>
<td>Newberry 4_30_WB</td>
<td>58.83</td>
<td>53.74</td>
<td>Y</td>
</tr>
<tr>
<td>Case 13</td>
<td>Newberry 5_1_EB</td>
<td>72.67</td>
<td>66.52</td>
<td>Y</td>
</tr>
<tr>
<td>Case 14</td>
<td>Newberry 5_1_WB</td>
<td>57.17</td>
<td>58.61</td>
<td>Y</td>
</tr>
</tbody>
</table>

Note: the averaged simulated travel times are based on 10 replications
<table>
<thead>
<tr>
<th>Offset (sec)</th>
<th>Average TT from AIMSUN using the offsets (sec)</th>
<th>Expected TT from model using the offsets (sec)</th>
<th>Standard deviation for model estimated TT (sec)</th>
<th>Percent difference when compared to AIMSUN results (%)</th>
<th>TT estimated by HCM arterial travel time model</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>43.50</td>
<td>35.12</td>
<td>0.25</td>
<td>-19.26%</td>
<td>43.45</td>
</tr>
<tr>
<td>10</td>
<td>42.67</td>
<td>35.12</td>
<td>0.25</td>
<td>-17.69%</td>
<td>43.45</td>
</tr>
<tr>
<td>20</td>
<td>40.50</td>
<td>35.12</td>
<td>0.25</td>
<td>-13.28%</td>
<td>43.45</td>
</tr>
<tr>
<td>30</td>
<td>38.67</td>
<td>35.12</td>
<td>0.25</td>
<td>-9.18%</td>
<td>43.45</td>
</tr>
<tr>
<td>40</td>
<td>39.50</td>
<td>54.31</td>
<td>0.29</td>
<td>+37.49%</td>
<td>43.45</td>
</tr>
<tr>
<td>50</td>
<td>46.33</td>
<td>53.9</td>
<td>0.86</td>
<td>+16.34%</td>
<td>43.45</td>
</tr>
<tr>
<td>60</td>
<td>53.00</td>
<td>53.9</td>
<td>0.86</td>
<td>+1.70%</td>
<td>43.45</td>
</tr>
<tr>
<td>70</td>
<td>58.17</td>
<td>53.9</td>
<td>0.86</td>
<td>-7.34%</td>
<td>43.45</td>
</tr>
<tr>
<td>80</td>
<td>48.67</td>
<td>35.12</td>
<td>0.25</td>
<td>-27.84%</td>
<td>43.45</td>
</tr>
<tr>
<td>90</td>
<td>43.50</td>
<td>35.12</td>
<td>0.25</td>
<td>-19.26%</td>
<td>43.45</td>
</tr>
</tbody>
</table>
Table 5-4. Travel time comparisons for Case 3 (Beaver_Pugh_PM_EB)

<table>
<thead>
<tr>
<th>Offset (sec)</th>
<th>Average TT from AIMSUN using the offsets (sec)</th>
<th>TT estimation from model using the offsets (sec)</th>
<th>Standard deviation for model estimated TT (sec)</th>
<th>Percent difference when compared to AIMSUN results (%)</th>
<th>TT estimated by HCM arterial travel time model</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>44.83</td>
<td>35.09</td>
<td>0.51</td>
<td>-21.73%</td>
<td>49.85</td>
</tr>
<tr>
<td>10</td>
<td>44.83</td>
<td>35.09</td>
<td>0.51</td>
<td>-21.73%</td>
<td>49.85</td>
</tr>
<tr>
<td>20</td>
<td>43.33</td>
<td>36.72</td>
<td>0.51</td>
<td>-15.26%</td>
<td>49.85</td>
</tr>
<tr>
<td>30</td>
<td>41.33</td>
<td>48.77</td>
<td>9.89</td>
<td>18.00%</td>
<td>49.85</td>
</tr>
<tr>
<td>40</td>
<td>40.17</td>
<td>56.57</td>
<td>0.89</td>
<td>40.83%</td>
<td>49.85</td>
</tr>
<tr>
<td>50</td>
<td>45.17</td>
<td>56.57</td>
<td>0.89</td>
<td>25.24%</td>
<td>49.85</td>
</tr>
<tr>
<td>60</td>
<td>49.67</td>
<td>56.57</td>
<td>0.89</td>
<td>13.89%</td>
<td>49.85</td>
</tr>
<tr>
<td>70</td>
<td>56.50</td>
<td>56.57</td>
<td>0.89</td>
<td>0.12%</td>
<td>49.85</td>
</tr>
<tr>
<td>80</td>
<td>55.00</td>
<td>42.39</td>
<td>9.63</td>
<td>-22.93%</td>
<td>49.85</td>
</tr>
<tr>
<td>90</td>
<td>45.00</td>
<td>35.09</td>
<td>0.51</td>
<td>-22.02%</td>
<td>49.85</td>
</tr>
<tr>
<td>100</td>
<td>44.83</td>
<td>35.09</td>
<td>0.51</td>
<td>-27.76%</td>
<td>49.85</td>
</tr>
<tr>
<td>Offset (sec)</td>
<td>Offset in AIMSUN (sec)</td>
<td>Average TT from AIMSUN using the offsets (sec)</td>
<td>TT estimation from model using the offsets (sec)</td>
<td>Standard deviation for model estimated TT (sec)</td>
<td>Percent difference when compared to AIMSUN results (%)</td>
</tr>
<tr>
<td>------------</td>
<td>------------------------</td>
<td>-----------------------------------------------</td>
<td>-----------------------------------------------</td>
<td>-----------------------------------------------</td>
<td>-------------------------------------------------</td>
</tr>
<tr>
<td>0</td>
<td>-50</td>
<td>71</td>
<td>67.24</td>
<td>0.22</td>
<td>-5.30%</td>
</tr>
<tr>
<td>10</td>
<td>-40</td>
<td>64.67</td>
<td>40.01</td>
<td>0.63</td>
<td>-38.13%</td>
</tr>
<tr>
<td>20</td>
<td>-30</td>
<td>57</td>
<td>39.47</td>
<td>0.33</td>
<td>-30.75%</td>
</tr>
<tr>
<td>30</td>
<td>-20</td>
<td>56.33</td>
<td>42.18</td>
<td>0.75</td>
<td>-25.12%</td>
</tr>
<tr>
<td>40</td>
<td>-10</td>
<td>56.67</td>
<td>67.24</td>
<td>0.22</td>
<td>18.65%</td>
</tr>
<tr>
<td>50</td>
<td>0</td>
<td>59</td>
<td>67.24</td>
<td>0.22</td>
<td>13.97%</td>
</tr>
<tr>
<td>60</td>
<td>10</td>
<td>61.5</td>
<td>67.24</td>
<td>0.22</td>
<td>9.33%</td>
</tr>
<tr>
<td>70</td>
<td>20</td>
<td>61.67</td>
<td>67.24</td>
<td>0.22</td>
<td>9.03%</td>
</tr>
<tr>
<td>80</td>
<td>30</td>
<td>64.83</td>
<td>67.24</td>
<td>0.22</td>
<td>3.72%</td>
</tr>
<tr>
<td>90</td>
<td>40</td>
<td>71</td>
<td>67.24</td>
<td>0.22</td>
<td>-5.30%</td>
</tr>
</tbody>
</table>
Table 5-6. Travel time comparisons for Case 5 (Beaver_Sparks_Mid_EB)

<table>
<thead>
<tr>
<th>Offset (sec)</th>
<th>Offset in AIMSUN (sec)</th>
<th>Average TT from AIMSUN using the offsets (sec)</th>
<th>TT estimation from model using the offsets (sec)</th>
<th>Standard deviation for model estimated TT (sec)</th>
<th>Percent difference when compared to AIMSUN results (%)</th>
<th>TT estimated by HCM arterial travel time model</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>-62</td>
<td>88.5</td>
<td>76.93</td>
<td>0.52</td>
<td>-13.07%</td>
<td>73.87</td>
</tr>
<tr>
<td>10</td>
<td>-52</td>
<td>85</td>
<td>57.61</td>
<td>15.99</td>
<td>-32.22%</td>
<td>73.87</td>
</tr>
<tr>
<td>20</td>
<td>-42</td>
<td>64</td>
<td>45.66</td>
<td>2.20</td>
<td>-28.66%</td>
<td>73.87</td>
</tr>
<tr>
<td>30</td>
<td>-32</td>
<td>58.83</td>
<td>46.94</td>
<td>0.66</td>
<td>-20.21%</td>
<td>73.87</td>
</tr>
<tr>
<td>40</td>
<td>-22</td>
<td>57.17</td>
<td>76.93</td>
<td>0.52</td>
<td>34.56%</td>
<td>73.87</td>
</tr>
<tr>
<td>50</td>
<td>-12</td>
<td>57.67</td>
<td>76.93</td>
<td>0.52</td>
<td>33.40%</td>
<td>73.87</td>
</tr>
<tr>
<td>60</td>
<td>-2</td>
<td>60</td>
<td>76.93</td>
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Table 5-7. Travel time comparisons for Case 6 (Beaver_Sparks_PM EB)

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<th>Standard deviation for model estimated TT (sec)</th>
<th>Percent difference when compared to AIMSUN results (%)</th>
<th>TT estimated by HCM arterial travel time model</th>
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<td>69.97</td>
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<td>TT estimation from model using the offsets (sec)</td>
<td>Standard deviation for model estimated TT(sec)</td>
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<th>Standard deviation for model estimated TT(sec)</th>
<th>Percent difference when compared to AIMSUN results (%)</th>
<th>TT estimated by HCM arterial travel time model</th>
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Table 5-10. Travel time comparisons for Case 9 (Park PM EB)

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<th>Offset in AIMSUN( sec) Int. 3</th>
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<th>TT estimation from model using the offsets (sec)</th>
<th>Standard deviation for model estimated TT (sec)</th>
<th>Percent difference when compared to AIMSUN results (%)</th>
<th>TT estimated by HCM arterial travel time model</th>
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<td>TT from model using the offsets (sec) (with consideration of residual queue)</td>
<td>Standard deviation for model estimated TT(sec)</td>
<td>Percent difference when compared to AIMSUN results (%)</td>
<td>TT estimated by HCM arterial travel time model</td>
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<td>TT estimation from model using the offsets (sec)</td>
<td>Standard deviation for model estimated TT (sec)</td>
<td>Percent difference when compared to AIMSUN results (%)</td>
</tr>
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Table 5-13. Travel time comparisons for Case 12 (Newberry_4_30_WB)

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Table 5-15. Travel time comparisons for Case 14 (Newberry_5_1_WB)

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CHAPTER 6
SENSITIVITY ANALYSIS AND APPLICATION

In this chapter, sensitivity analysis is performed to test how sensitive the analytical model is to selected inputs. The following variables are tested:

- G/C ratio
- Link length
- Maximum operating speed
- Acceleration/ deceleration rate
- The entering flow rate at each intersection

These variables are tested for each of the 13 cases used in chapter 5. Changing some of the variables (g/C ratio and flow rate) can lead to congested conditions. Thus, the travel times are shown for both non-congested and congested conditions. Travel times when spillback occurs are not calculated since they are beyond the capability of the model. Varying values of offset are used in the previous chapter for model validation so it will not be used in this chapter. Because of the similarity of the sensitivity analysis for each site, only one case is selected from each of the four sites to show in this chapter. The rest of the analysis is included in Appendix B.

6.1 G/C Ratio

6.1.1 Site 1: Beaver_Pugh_AM /Case 1

The cycle length for the two intersections is 90 seconds. The green time is changed to create different g/C ratios. Figure 6-1 shows that, as expected, travel time decreases as the g/C ratio increases. When g/C ratio is 0.11, the arterial is congested and spillback occurs.
6.1.2 Site 2: Beaver_Sparks_AM/Cae 4

The cycle length for the two intersections is 90 seconds. The green time is changed to create different g/C ratios. Figure 6-2 shows that, as expected, travel time decreases as the g/C ratio increases. When g/C ratio is 0.11 and 0.22, the arterial is congested and spillback occurs.

6.1.3 Site 3: Park_Mid_EB/Case 7

The cycle length for the two intersections is 100 seconds. The green times are changed to create different g/C ratios. Figure 6-3 shows that, as expected, travel time decreases as the g/C ratio increases. When g/C ratio is 0.1 and 0.2, the arterial is congested and spillback occurs.

6.1.4 Site 4: Newberry_4_30_EB/Case 11

The cycle length for the two intersections is 100 seconds. The green time is changed to create different g/C ratios. Figure 6-4 shows that, as expected, travel time decreases as the g/C ratio increases. When g/C ratio is 0.07, 0.13, and 0.20, the arterial is congested and spillback occurs.

6.2 Link Length

6.2.1 Site 1: Beaver_Pugh_AM/Case 1

Starting from 100 ft, Link length is increased by 100 ft for every run. The maximum link length is set to 2000 ft. As shown in the previous chapter, offset is a very important variable in estimating travel time. For the one link cases, there are two possible ways to do the sensitivity analysis for link length. One is to adjust the ideal offset every time when the link length is changing; the other is to keep it unadjusted. The result from the first method is shown in Figure 6-5; the result for the second method is shown in Figure 6-6.

As shown in Figure 6-5, if the offset is adjusted as the link length changes, travel time increases as the link length increases. If the offset is not changed, as shown in Figure 6-6, travel time at first increases and then drops to a low value at 1100 ft, and then increases at 1300 ft. This
is because the travel time is calculated by averaging all possible travel times. For this case, the travel time drops at 1100 ft, the selected travel times are \( t_{12}, t_{22}, \) and \( t_{33} \). In 1000 ft, the selected travel times are \( t_{11}, t_{21}, \) and \( t_{32} \). Selected travel time sets are different and the probabilities associated with each selected travel time are changing at every run. Thus, travel time does not keep increasing as the link length increases if the offset is unchanged.

6.2.2 Site 2: Beaver_Sparks_AM/Case 4

Starting from 100 ft, Link length is increased by 100 ft for every run. The maximum link length is set to 2000 ft. The link length is also tested in two ways as mentioned in case 1. The result from the first method is shown in Figure 6-7; the result for the second method is shown in Figure 6-8. As shown in Figure 6-7, if the offset is adjusted as the link length changes, travel time increases as the link length increases. If the offset is not changed, as shown in Figure 6-8, travel time at first increases and then drops to a low value at 1500 ft, and then increases at 1800 ft. The reason for this change is the same as the one mentioned in Case 1.

6.2.3 Site 3: Park_Mid_EB/Case 7

In this case, there are two links. To see how travel time is affected in this two-link case. One link length is kept unchanged when the other one is increased by 100 ft starting from 100 ft. The maximum link length used is 2000 ft. The offset is kept unchanged. The result from the first link is shown in Figure 6-9; the result for the second link is shown in Figure 6-10. As shown in Figure 6-9, if the offset is unchanged as the link length changes, travel time at first increases and then drops to a low value at 1200 ft, and then increases at 1500 ft. As shown in Figure 6-10, if the offset is not changed, travel time at first increases and then drops to a low value at 200 ft, then increases at 400 ft. The reason for these changes are as explained in Case 1.
6.2.4 Site 4: Newberry_4_30_EB/Case 11

In this case, there are three links. To see how travel time is affected in this three-link case. Two link lengths are kept unchanged when the other one is increased by 100 ft starting from 100 ft. The maximum link length used is 1000 ft. The offset is kept unchanged. The result from the first link is shown in Figure 6-11; the result for the second link is shown in Figure 6-12; the result for the third link is shown in Figure 6-13; As shown in Figure 6-11, if the offset is unchanged as the link length changes, travel time at first increases and then drops to a low value at 600 ft, and then increases at 700 ft. As shown in Figure 6-12, if the offset is not changed, travel time at first increases and then drops to a low value at 900 ft, and then increases at 1000 ft. As shown in Figure 6-13, if the offset is not changed, travel time increases all the time. It is possible since the selected travel time set does not change for this case, as link length increases, travel time increases.

6.3 Maximum Operating Speed

6.3.1 Site 1: Beaver_Pugh_AM/Case 1

Similar to the link length sensitivity analysis, the changes in maximum operating speed can cause changes in the ideal offset and then the travel time. Thus, as for the link length, there are two ways to test the sensitivity; with adjusted or unadjusted offset. For this case, the maximum operating speed is 25 mph. Thus, the lower bound of the maximum operating speed is set to 5 mph and increase by 5 mph every time.

Figure 6-14 shows that travel time decreases as the maximum operating speed increases when the offset is changed every run. Figure 6-15 shows how travel time changes if the offset is not changed: travel time at first decreases as the maximum operating speed increases and when the speed reaches 30 mph, the travel time increases and then decreases again there after. The
reason is same as the one mentioned for Figure 6-6. Travel time does not keep decreasing as the speed increases if the offset is unchanged.

6.3.2 Site 2: Beaver_Sparks_AM/Case 4

Similarly to Case 1, the sensitivity analysis is performed in two ways and the lower bound of the maximum operating speed is set to 15mph and increase by 5mph every time until it reaches 45 mph.

Figure 6-16 shows that travel time deceases as the maximum operating speed increases when the offset is changed every run. Figure 6-17 shows how travel time changes if the offset is not changed: travel time at first decreases as the maximum operating speed increases and when the speed reaches 25 mph, the travel time increases and then decreases again there after. The reason is same as the one mentioned for Figure 6-6. Travel time does not keep decreasing as the speed increases if the offset is unchanged.

6.3.3 Site 3: Park_Mid_EB/Case 7

As mentioned before, there are two links in this case. To see how travel time is affected in this two-link case, the maximum operating speed for one link is kept unchanged when the other one is increased by 5mph starting from 15 mph until it reaches 45mph. The offset is kept unchanged.

The result from the first link is shown in Figure 6-18; the result for the second link is shown in Figure 6-19. As shown in Figure 6-18, if the offset is unchanged as the maximum operating speed changes, travel time at first decreases, increases to a higher value at 40 mph, and then decreases at 45mph. As shown in Figure 6-19, if the offset is not changed, travel time decreases when the maximum operating speed increases.
6.3.4 Site 4: Newberry_4_30_EB/Case 11

As mentioned before, there are three links in this case. To see how travel time is affected in this three-link case, the maximum operating speed for two links are kept unchanged when the other one is increased by 5mph starting from 25 mph until it reaches 55mph. The offset is kept unchanged.

The result from the first link is shown in Figure 6-20; the result for the second link is shown in Figure 6-21; the result for the third link is shown in Figure 6-22. As shown in Figure 6-20, if the offset is unchanged as the maximum operating speed changes, travel time at first decreases and then increases to a higher value at 40 mph, and then decreases at 45mph. As shown in Figure 6-21, if the offset is unchanged as the maximum operating speed changes, travel time at first decreases and then increases to a higher value at 35 mph, and then decreases at 40mph. As shown in Figure 6-22, if the offset is not changed, travel time decreases when the maximum operating speed increases.

6.4 Acceleration Rate / Deceleration Rate

6.4.1 Site 1: Beaver_Pugh_AM /Case 1

For this case, the acceleration rate is 15.78 ft/s². The lower bound for the analysis is set to 5 ft/s² and it is increased by 2.5 ft/s² until it reaches 20 ft/s².

Figure 6-23 shows that travel time slightly decreases as the acceleration rate increases.

For this case, the deceleration rate is 16.83 ft/s². The lower bound for the analysis is set to 5 ft/s² and it is increased by 2.5 ft/s² until it reaches 20 ft/s².

Figure 6-24 shows that travel time slightly decreases as the deceleration rate increases.
6.4.2 Site 2: Beaver_Sparks_AM/Case 4

For both the acceleration and the deceleration rate, similarly to the previous case, the lower bound for this sensitivity analysis is set to 5 ft/s² and it is added by 2.5 ft/s² until it researches 20 ft/s².

Figure 6-25 shows that travel time slightly decreases as the acceleration rate increases.

Figure 6-26 shows that travel time slightly decreases as the deceleration rate increases.

6.4.3 Site 3: Park_Mid_EB/Case 7

For both the acceleration and the deceleration rate, similarly to the previous case, the lower bound for this sensitivity analysis is set to 5 ft/s² and it is added by 2.5 ft/s² until it reaches 20 ft/s².

Figure 6-27 shows that travel time slightly decreases as the acceleration rate increases.

Figure 6-28 shows that travel time slightly decreases as the deceleration rate increases.

6.4.4 Site 4: Newberry_4_30_EB/Case 11

For both the acceleration and the deceleration rate, similarly the previous case, the lower bound for this sensitivity analysis is set to 5 ft/s² and it is added by 2.5 ft/s² until it researches 20 ft/s².

Figure 6-29 shows that travel time slightly decreases as the acceleration rate increases.

Figure 6-30 shows that travel time slightly decreases as the deceleration rate increases.

6.5 The Entering Flow Rate at Each Intersection

6.5.1 Site 1: Beaver_Pugh_AM /Case 1

Depending on the flow levels and the respective green ratios and offsets, the flow rates of the side streets have a different impact on travel time. All the entering flows can affect travel time at each intersection. The sensitivity analysis for flow rate examines how the travel time varies as the entering flows at each intersection change. For this case, which has two
intersections, the method is to keep the entering flow of one intersection unchanged and test how the travel time varies as the entering flow of the other intersection changes. For the entering flow at intersection 1, Figure 6-31 shows that travel time increases as the flow rate increases under both non-congested and congested conditions. For the entering flow at intersection 2, Figure 6-32 shows that travel times remain almost the same as flow increases under non-congested conditions; when congestion occurs, the travel time increases.

6.5.2 Site 2: Beaver_Sparks_AM/Case 4

For this case, there are two intersections. The method is the same as in Case 1. For the entering flow at intersection 1, Figure 6-33 shows that travel time decreases as the flow rate increases, which is counter intuitive. This happens because when the flow increases, it results in an increase of the interval length of Conditions 1 and 2, and a decrease of Condition 3 (Figure 6-34). Since the three conditions for the other intersection remain the same, the resulting travel time changes randomly. As shown in Figure 6-34, when the entering flow rate at intersection 1 increases, the interval lengths of Conditions 1 and 2 increase; the possible travel times that are obtained for calculating the expected travel time are different. In this case, the obtained travel times change from t11 and t21 to t31 and t32, which result in overall lower travel times. Since the offset remains the same, the increase in flows results in more optimal travel times.

For the entering flow at intersection 2, Figure 6-34 shows travel time increases as flow increases under both congested and un-congested condition.

6.5.3 Site 3: Park_Mid_EB/Case 7

For this case, there are three intersections. The method is the same as in Case 1. For the entering flow at intersection 1, Figure 6-35 shows that travel time decreases as the flow rate increases. Similar to Figure 6-33, Figure 6-35 shows a decreasing trend in travel time as entering flow increases. The explanation for this trend is also similar to the one for Figure 6-33: when the
flow increases, it results in an increase of the interval length of Conditions 1 and 2 and a decrease of Condition 3. Since the three conditions for the other intersection remain the same, the resulting travel time changes arbitrarily. For intersection 2, Figure 6-36 shows travel time remains almost the same as flow increases under congested and increases under un-congested condition. For intersection 3, Figure 6-37 shows that travel time increases as flow increases under both congested and non-congested condition.

6.5.4 Site 4: Newberry_4_30_EB/Case 11

For this case, there are four intersections. The method is the same as in Case 1. For the entering flow at intersection 1, Figure 6-38 shows travel time decreases as the flow rate increases. Similar to Figure 6-33, Figure 6-38 shows a decreasing trend in travel time as entering flow increases. The explanation for this trend is also similar to the one for Figure 6-36: when the flow increases, it results in an increase of the interval length of Conditions 1 and 2 and a decrease of Condition 3. Since the three conditions for the other intersection remain the same, the resulting travel time changes randomly. For the entering flow at intersection 2, Figure 6-39 shows travel time increases as flow increases under both congested and non-congested condition. For the entering flow at intersection 3, Figure 6-40 shows travel time decreases as flow increases under congested and increases non-congested condition. For the entering flow at intersection 4, Figure 6-41 shows travel time remains almost the same as flow increases under congested and increases non-congested condition. Similar to Figure 6-33 and Figure 6-38, Figure 6-41 (for the un-congested part) shows travel time decreases as the flow rate increases. The explanations are the same as that for Figure 6-38.

6.6 Conclusions

Based on the sensitivity analysis for all the cases, the following conclusions can be reached:
6.6.1 g/C Ratio

When the g/C ratio increases, the travel time decreases for all the cases. The longer the green time given to the major street, the shorter the travel time is for the major street. The travel time continues to decrease while the g/C ratio can satisfy the demand; then it decreases only slightly. When the g/C ratio is extremely low, residual queues begin to form, congestion occurs and travel time becomes very high. There are two types of congested travel times. In the first type the vehicles arriving in the previous cycle can be discharged in the current cycle. This travel time can be calculated. In the other type the residual queue keeps increasing until spill back occurs. For this situation travel time cannot be calculated using their analytical model.

6.6.2 Link Length

When link length increases, the travel time does not necessarily increase. The travel time depends on the offset for each case. For one-link cases, if the offset is set to the ideal offset every time as the link length changes, the travel times keep increasing as the link length increases. If the offset is unchanged, the travel times do not keep increasing as the link length increases, they increase at first, and then at a certain point they drop to a lower value. Starting from that value, the travel times increase again. Travel time is significantly affected by signal coordination. If the coordination is poor, even a shorter distance can cause a longer travel time. For the cases that contain more than one links, the travel times are tested as the link length of one link changes and while the others remain unchanged. The offsets are kept unchanged for these cases. The travel times show similar trends as the one-link cases with unadjusted offset.

6.6.3 Maximum Operating Speed

The maximum operating speed has the opposite effect on travel times to the link length. As expected increasing, maximum operating speed results in lower travel times.
6.6.4 Acceleration/ Deceleration Rate

Increasing the acceleration and deceleration rate reduces travel time. However, compared to other factors, travel time changes are relatively small. Thus, travel time is not highly affected by the acceleration and deceleration rates.

6.6.5 Entering Flow Rate at Each Intersection

Before congestion occurs, there is no trend in travel time. In the methodology, flow rate is used to calculate the interval lengths for the three conditions. Condition 1 is when vehicles have to wait in the stopped queue; Condition 2 is when vehicles move with the discharging queue, Condition 3 is when vehicles move without a queue present. If the flow increases, the Condition 1 and Condition 2 increase, and Condition 3 decreases. For one-link cases, the upstream intersection flow can affect the travel time this way. For the downstream intersection, the travel time is not only affected by the interval lengths for the conditions, it is also affected by the residual queue. When the flow rate increases to a certain point, a residual queue occurs and it causes the travel time to increase. After the residual queue occurs, the segment is considered as congested. Thus, the congested travel time is higher than the non-congested travel time.

![Figure 6-1. Sensitivity analysis: g/C ratio versus travel time(Beaver_Pugh_AM)](image)

Figure 6-1. Sensitivity analysis: g/C ratio versus travel time(Beaver_Pugh_AM)
Figure 6-2. Sensitivity analysis: g/C ratio versus travel time (Beaver_Sparks_AM)

Figure 6-3. Sensitivity analysis: g/C ratio versus travel time (Park_Mid_EB)

Figure 6-4. Sensitivity analysis g/C ratio versus travel time (Newberry_4_30_EB)
Figure 6-5. Sensitivity analysis: link length versus travel time with adjusted offset (Beaver_Pugh_AM)

Figure 6-6. Sensitivity analysis: link length versus travel time with unadjusted offset (Beaver_Pugh_AM)

Figure 6-7. Sensitivity analysis: link length versus travel time with adjusted offset (Beaver_Sparks_AM)
Figure 6-8. Sensitivity analysis: link length versus travel time with unadjusted offset (Beaver_Sparks_AM)

Figure 6-9. Sensitivity analysis: link length versus travel time with unadjusted offset for link 0 (Park_Mid_EB)

Figure 6-10. Sensitivity analysis: link length versus travel time with unadjusted offset for link 1 (Park_Mid_EB)
Figure 6-11. Sensitivity analysis: link length ratio versus travel time with unadjusted offset for link 0 (Newberry_4_30_EB)

Figure 6-12. Sensitivity analysis: link length versus travel time with unadjusted offset for link 1 (Newberry_4_30_EB)

Figure 6-13. Sensitivity analysis: link length versus travel time with unadjusted offset for link 2 (Newberry_4_30_EB)
Figure 6-14. Sensitivity analysis: maximum operating speed versus travel time with adjusted offset (Beaver_Pugh_AM)

Figure 6-15. Sensitivity analysis: maximum operating speed versus travel time with unadjusted offset (Beaver_Pugh_AM)

Figure 6-16. Sensitivity analysis: maximum operating speed versus travel time with adjusted offset (Beaver_Sparks_AM)
Figure 6-17. Sensitivity analysis: maximum operating speed versus travel time with unadjusted offset (Beaver_Sparks_AM)

Figure 6-18. Sensitivity analysis: maximum operating speed versus travel time with unadjusted offset for link 0 (Park_Mid_EB)
Figure 6-19. Sensitivity analysis: maximum operating speed versus travel time with unadjusted offset (Park_Mid_EB)

![Sensitivity Analysis (TT vs MOS) for Newberry_4_30_EB_link0](image)

Figure 6-20. Sensitivity analysis: maximum operating speed versus travel time with unadjusted offset for link 0 (Newberry_4_30_EB)

![Sensitivity Analysis (TT vs MOS) for Newberry_4_30_EB_link1](image)

Figure 6-21. Sensitivity analysis: maximum operating speed versus travel time with unadjusted offset for link 1 (Newberry_4_30_EB)

![Sensitivity Analysis (TT vs MOS) for Newberry_4_30_EB_link2](image)
Figure 6-22. Sensitivity analysis: maximum operating speed versus travel time with unadjusted offset for link 2 (Newberry_4_30 EB)

Figure 6-23. Sensitivity analysis: acceleration rate versus travel time (Beaver_Pugh_AM)

Figure 6-24. Sensitivity analysis: deceleration rate versus travel time (Beaver_Pugh_AM)

Figure 6-25. Sensitivity analysis: acceleration rate versus travel time (Beaver_Sparks_AM)
Figure 6-26. Sensitivity analysis: deceleration rate versus travel time (Beaver_Sparks_AM)

Figure 6-27. Sensitivity analysis: acceleration rate versus travel time (Park_Mid_EB)

Figure 6-28. Sensitivity analysis: deceleration rate versus travel time (Park_Mid_EB)
Figure 6-29. Sensitivity analysis: acceleration rate versus travel time (Newberry_4_30_EB)

Figure 6-30. Sensitivity analysis: deceleration rate versus travel time (Newberry_4_30_EB)

Figure 6-31. Sensitivity analysis: flow rate versus travel time for intersection 1(Beaver_Pugh_AM)
Figure 6-32. Sensitivity analysis: flow rate versus travel time for intersection 2 (Beaver_Pugh_AM)

Figure 6-33. Sensitivity analysis: flow rate versus travel time for intersection 1 (Beaver_Sparks_AM)

Figure 6-34. Travel time changes due to flow changes
Figure 6-35. Sensitivity analysis: flow rate versus travel time for intersection 2(Beaver_Sparks_AM)

Figure 6-36. Sensitivity analysis: flow rate versus travel time for intersection 1(Park_Mid_EB)

Figure 6-37. Sensitivity analysis: flow rate versus travel time for intersection 2(Park_Mid_EB)
Sensitivity Analysis (TT vs Intersection 3 Entering Flow) for Park_Mid_EB

![Graph showing Sensitivity Analysis for Park_Mid_EB](image1)

Figure 6-38. Sensitivity analysis: flow rate versus travel time for intersection 3(Park_Mid_EB)

Sensitivity Analysis (TT vs Intersection 1 Entering Flow) for Newberry_4_30_EB

![Graph showing Sensitivity Analysis for Newberry_4_30_EB](image2)

Figure 6-39. Sensitivity analysis: flow rate versus travel time for intersection 1(Newberry_4_30_EB)

Sensitivity Analysis (TT vs Intersection 3 Entering Flow) for Newberry_4_30_EB

![Graph showing Sensitivity Analysis for Newberry_4_30_EB](image3)

Figure 6-40. Sensitivity analysis: flow rate versus travel time for intersection 3(Newberry_4_30_EB)
Figure 6-41. Sensitivity analysis: flow rate versus travel time for intersection 4 (Newberry_4_30_EB)

Figure 6-42. Sensitivity analysis: flow rate versus travel time for intersection 5 (Newberry_4_30_EB)
CHAPTER 7
CONCLUSIONS AND FUTURE RESEARCH

In this chapter, the method developed for arterial travel time estimation is summarized and the results obtained from the model are reviewed. Some conclusions are reached based on the estimation results. Some suggestions for improving the model and some possible future research directions are presented at the end of the chapter.

7.1 Overview of the Analytical Model

This research proposed an analytical model for estimating the expected travel time for signalized arterials. In the method, the travel time is defined as the sum of $T_M$ (travel time in motion), $W_{QW}$ (Waiting Time in the Queue), and $M_{QW}$ (Moving Time in the Queue). Using the vehicle trajectories when traveling between two intersections, the travel times are calculated for 9 different flow profiles. If there are n intersections, then there are $3^n$ flow profiles. The expected travel time is calculated based on the probability of occurrence of each travel time for each flow profile. In this analytical model, the green time, cycle length, link length, maximum operating speed, offset, acceleration/deceleration rate, and the entering flow rate at each intersection are used as inputs.

7.2 Results from the Analytical Model

Data were collected at State College, PA and Gainesville, FL with 4 different data collection sites for various time periods for a total of 14 cases. Since the field data were collected on arterials with semi-actuated signal timing, field traffic conditions were firstly replicated in the simulator. After changing the signal timings to pre-timed, the travel times from the simulation were compared to the travel times estimated by the analytical model. For the three sites from State College, PA, the results show that travel times from both the simulation and the analytical model have similar trends. The average difference between the simulated and the model
estimated travel times shows that the travel time difference for Beaver_Pugh, Beaver_Sparks, and Park cases are smaller than those of Newberry cases. In the Park Ave. site, traffic conditions are congested and for some cases spillback occurs. Travel time cannot be estimated with the analytical model for the spillback cases. For Newberry Rd Site, there are big differences in travel times between the simulation and the model. The differences are caused by the driveway delay which is not considered in the analytical model. The sensitivity analysis shows that 1) when the g/C ratio increases, the travel time decreases for all the cases; 2) when link length increases, the travel time does not necessarily increase; the travel time depends on the offset for each case; 3) the maximum operating speed has the opposite effect on travel times to the link length; higher maximum operating speed results in lower travel times; 4) increasing the acceleration and deceleration rate reduces travel time; but the magnitude of the changes is relatively small; and 5) before congestion occurs, there is no trend in travel time as a function of flow; the congested travel time is always higher than the non-congested travel time and increases with flow.

7.3 Suggestions and Future Research

There are several aspects that are not considered in this analytical model. These aspects would enhance the capability of the analytical model

7.3.1 Delay Caused by Driveways

The first aspect is the delay caused by driveways. It is known that turning movements in driveways can cause additional delay. Thus, to make the estimation more accurate, it has to take the delay as an extra term and add it into the total estimated travel time.

Previous studies estimate turning delays based on different factors selected. Some common factors are flow rate on the major street, turning flow rate, operating speed, number of lanes on the major street, and driveway density.
7.3.2 Actuated or Semi-Actuated Signal Timing

The signal timing in the model is pre-timed. However, semi-actuated signals are widely used in the field and the field data were collected from the arterials with semi-actuated signal timing. The major difference between the pre-timed and the semi-actuated signal timing is that the green interval for the minor street changes every cycle due to the fluctuating traffic arrival rate. Cycle by cycle analysis would be required in the cases for actuated or semi-actuated signal timing. Thus, simulation might be more appropriate.

7.3.3 Travel Time Estimations for Other O/D Pairs

The travel time estimated in this analytical model is for the through movement, which is only one pair of OD. For an arterial with several intersections and driveways, there are several pairs of ODs. The travel time estimations for these ODs are also very helpful for travelers and operators. These travel time estimations are related to travel time associated with turning maneuver which is sometimes interrupted by other conflicting movements. Thus, the travel time estimations depend on the specific turning movement, the conflicting movement, and the flow rate for both. Then according to these a specific travel time is calculated.

7.3.4 Travel Time Estimation for Spill Back Cases

Travel time estimation for congested conditions when spill back occurs is also a possible future research topic. Such an analytical procedure may result in an iterative process when the queues of the two successive intersections interact. Additional research should be undertaken to investigate this issue further.
APPENDIX A
SENSITIVITY ANALYSIS

g/C ratio

Figure A-1. Sensitivity analysis: g/C ratio versus travel time (Beaver_Pugh_PM)

Figure A-2. Sensitivity analysis: g/C ratio versus travel time (Beaver_Sparks_Mid)

Figure A-3. Sensitivity Analysis: g/C ratio versus travel time (Beaver_Sparks_PM)
Figure A-4. Sensitivity analysis: G/C ratio versus travel time (Park_Mid_WB)

Figure A-5. Sensitivity analysis: G/C ratio versus travel time (Park_PM_EB)

Figure A-6. Sensitivity analysis: G/C ratio versus travel time (Park_PM_WB)
Figure A-7. Sensitivity analysis: g/C ratio versus travel time (Newberry_4_30_WB)

Figure A-8. Sensitivity analysis: g/C ratio versus travel time (Newberry_5_1_EB)

Figure A-9. Sensitivity analysis: g/C ratio versus travel time (Newberry_5_1_WB)
Figure A-10. Sensitivity analysis: link length versus travel time with adjusted offset (Beaver_Pugh_PM)

Figure A-11. Sensitivity analysis: link length versus travel time with unadjusted offset (Beaver_Pugh_PM)

Figure A-12. Sensitivity analysis: link length versus travel time with adjusted offset (Beaver_Sparks_Mid)
Figure A-13. Sensitivity analysis: link length versus travel time with unadjusted offset (Beaver_Sparks_Mid)

Figure A-14. Sensitivity analysis: link length versus travel time with adjusted offset (Beaver_Sparks_PM)

Figure A-15. Sensitivity analysis: link length versus travel time with unadjusted offset (Beaver_Sparks_PM)
Figure A-16. Sensitivity analysis: link length versus travel time with unadjusted offset (Park_Mid_WB)

Figure A-17. Sensitivity analysis: link length versus travel time with unadjusted offset (Park_Mid_WB)

Figure A-18. Sensitivity analysis: link length ratio versus travel time with unadjusted offset (Park_PM_EB)
Figure A-19. Sensitivity analysis: link length versus travel time with unadjusted offset (Park_PM_EB)

Figure A-20. Sensitivity analysis: link length ratio versus travel time with unadjusted offset (Newberry_4_30_WB)

Figure A-21. Sensitivity analysis: link length ratio versus travel time with unadjusted offset (Newberry_4_30_WB)
Sensitivity Analysis (TT vs Link Length) for Newberry_4_30_WB_link 2

Figure A-22. Sensitivity analysis: link length ratio versus travel time with unadjusted offset (Newberry_4_30 WB)

Sensitivity Analysis (TT vs Link Length) for Newberry_4_30_WB_link 3

Figure A-23. Sensitivity analysis: link length ratio versus travel time with unadjusted offset (Newberry_4_30 WB)

Sensitivity Analysis (TT vs Link Length) for Newberry_5_1_EB_link 0

Figure A-24. Sensitivity analysis: link length versus travel time with unadjusted offset (Newberry_5_1 EB)
Figure A-25. Sensitivity analysis: link length versus travel time with unadjusted offset (Newberry_5_1_EB)

Figure A-26. Sensitivity analysis: link length versus travel time with unadjusted offset (Newberry_5_1_EB)

Figure A-27. Sensitivity analysis: link length ratio versus travel time with unadjusted offset (Newberry_5_1_WB)
Figure A-28. Sensitivity analysis: link length ratio versus travel time with unadjusted offset (Newberry_5_1_WB)

Figure A-29. Sensitivity analysis: link length ratio versus travel time with unadjusted offset (Newberry_5_1_WB)

Figure A-30. Sensitivity analysis: link length ratio versus travel time with unadjusted offset (Newberry_5_1_WB)
Maximum Operating Speed

Figure A-31. Sensitivity analysis: MOS versus travel time with adjusted offset (Beaver_Pugh_PM)

Figure A-32. Sensitivity analysis: MOS versus travel time with unadjusted offset (Beaver_Pugh_PM)

Figure A-33. Sensitivity analysis: MOS versus travel time with adjusted offset (Beaver_Sparks_Mid)
Figure A-34. Sensitivity analysis: MOS versus travel time with unadjusted offset (Beaver_Sparks_Mid)

Figure A-35. Sensitivity analysis: MOS versus travel time with adjusted offset (Beaver_Sparks_PM)

Figure A-36. Sensitivity Analysis: MOS versus travel time with unadjusted offset (Beaver_Sparks_PM)
Figure A-37. Sensitivity analysis: MOS versus travel time with unadjusted offset (Park_Mid_WB)

Figure A-38. Sensitivity analysis: MOS versus travel time with unadjusted (Park_Mid_WB)

Figure A-39. Sensitivity analysis: MOS versus travel time with unadjusted offset (Park_PM_EB)
Figure A-40. Sensitivity analysis: MOS versus travel time with unadjusted offset (Park_PM_EB)

Figure A-41. Sensitivity analysis: MOS versus travel time with unadjusted offset (Newberry_4_30_WB)

Figure A-42. Sensitivity analysis: MOS versus travel time with unadjusted offset (Newberry_4_30_WB)
Figure A-43. Sensitivity analysis: MOS versus travel time with unadjusted offset (Newberry_4_30_WB)

Figure A-44. Sensitivity analysis: MOS versus travel time with unadjusted offset (Newberry_4_30_WB)

Figure A-45. Sensitivity analysis: MOS versus travel time with unadjusted offset (Newberry_5_1_EB)
Figure A-46. Sensitivity analysis: MOS versus travel time with unadjusted offset (Newberry_5_1_EB)

Figure A-47. Sensitivity analysis: MOS versus travel time with unadjusted offset (Newberry_5_1_EB)

Figure A-48. Sensitivity analysis: MOS versus travel time with unadjusted offset (Newberry_5_1_WB)
Figure A-49. Sensitivity analysis: MOS versus travel time with unadjusted offset (Newberry_5_1_WB)

Figure A-50. Sensitivity analysis: MOS versus travel time with unadjusted offset (Newberry_5_1_WB)

Figure A-51. Sensitivity analysis: MOS versus travel time with unadjusted offset (Newberry_5_1_WB)
Figure A-52. Sensitivity analysis: acceleration rate versus travel time (Beaver_Pugh_PM)

Figure A-53. Sensitivity analysis: deceleration rate versus travel time (Beaver_Pugh_PM)

Figure A-54. Sensitivity analysis: acceleration rate versus travel time (Beaver_Sparks_Mid)
Figure A-55. Sensitivity analysis: deceleration rate versus travel time (Beaver_Sparks_Mid)

Figure A-56. Sensitivity analysis: acceleration rate versus travel time (Beaver_Sparks_PM)

Figure A-57. Sensitivity analysis: deceleration rate versus travel time (Beaver_Sparks_PM)
Figure A-58. Sensitivity analysis: acceleration rate versus travel time (Park_Mid_WB)

Figure A-59. Sensitivity analysis: deceleration rate versus travel time (Park_Mid_WB)

Figure A-60. Sensitivity Analysis: acceleration rate versus travel time (Park_PM EB)
Figure A-61. Sensitivity Analysis: deceleration rate versus Travel time (Park_PM_EB)

Figure A-62. Sensitivity analysis: acceleration rate versus travel time (Newberry_4_30_WB)

Figure A-63. Sensitivity analysis: deceleration rate versus travel time (Newberry_4_30_WB)
Figure A-64. Sensitivity analysis: acceleration rate versus travel time (Newberry_5_1_EB)

Figure A-65. Sensitivity analysis: deceleration rate versus travel time (Newberry_5_1_EB)

Figure A-66. Sensitivity analysis: acceleration rate versus travel time (Newberry_5_1_WB)
Figure A-67. Sensitivity analysis: deceleration rate versus travel time (Newberry_5_1_WB)

Figure A-68. Sensitivity analysis: flow rate versus travel time for intersection 1(Beaver_Pugh_PM)

Figure A-69. Sensitivity Analysis: flow rate versus travel time for intersection 2(Beaver_Pugh_PM)
Figure A-70. Sensitivity analysis: flow rate versus travel time for intersection 1(Beaver_Sparks_Mid)

Figure A-71. Sensitivity analysis: flow rate versus travel time for intersection 2(Beaver_Sparks_Mid)

Figure A-72. Sensitivity analysis: flow rate versus travel time for intersection 1(Beaver_Sparks_PM)
Figure A-73. Sensitivity analysis: flow rate versus travel time for intersection 2 (Beaver_Sparks_PM)

Figure A-74. Sensitivity analysis: flow rate versus travel time for intersection 3 (Park_Mid_WB)

Figure A-75. Sensitivity analysis: flow rate versus travel time for intersection 2 (Park_Mid_WB)
Figure A-76. Sensitivity analysis: flow rate versus travel Time for intersection 1 (Park_Mid_WB)

Figure A-77. Sensitivity analysis: flow rate versus travel time for intersection 1 (Park_PM_EB)

Figure A-78. Sensitivity analysis: flow rate versus travel time for intersection 2 (Park_PM_EB)
Figure A-79. Sensitivity analysis: flow rate versus travel time for intersection 3 (Park_PM_EB)

Figure A-80. Sensitivity analysis: flow rate versus travel time for intersection 3 (Park_PM_WB)

Figure A-81. Sensitivity analysis: flow rate versus travel time for intersection 5 (Newberry_4_30_WB)
Figure A-82. Sensitivity analysis: flow rate versus travel time for intersection 4 (Newberry_4_30_WB)

Figure A-83. Sensitivity analysis: flow rate versus travel time for intersection 3 (Newberry_4_30_WB)

Figure A-84. Sensitivity analysis: flow rate versus travel time for intersection 2 (Newberry_4_30_WB)
Figure A-85. Sensitivity analysis: flow rate versus travel time for intersection 1 (Newberry_4_30_WB)

Entering Flow rate at Each Intersection: same as Newberry_5_1_EB
Entering Flow rate at Each Intersection: same as Newberry_4_30_WB


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BIOGRAPHICAL SKETCH

Xiao Cui was born on July 10, 1979, in Beijing, China. She earned her Bachelor of Science degree in urban and regional planning from Zhongshan University in 2001 in Guangzhou, China. After that, she came to the United States for her further study. She got her Master of Science degree in the same major from the University of Iowa in 2003. Upon graduating in May 2003, she entered the Ph.D. program in transportation engineering at Penn State University in the Department of Civil and Environmental Engineering. In August 2004, she transferred to the Department of Civil and Coastal Engineering at the University of Florida.