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Travel Time in Macroscopic Traffic Models for Origin-Destination Estimation

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TRAVEL TIME IN MACROSCOPIC TRAFFIC MODELS FOR ORIGIN-DESTINATION ESTIMATION

by

Eric Youngblom

A Thesis Submitted in

Partial Fulfillment of the

Requirements for the Degree of

Master of Science

in Engineering

at

The University of Wisconsin-Milwaukee

May 2013

ABSTRACT
TRAVEL TIME IN MACROSCOPIC TRAFFIC
MODELS FOR ORIGIN-DESTINATION ESTIMATION

by

Eric Youngblom

The University of Wisconsin-Milwaukee, 2013
Under the Supervision of Professor Alan J. Horowitz

Transportation macroscopic modeling is a tool for analyzing and prioritizing future transportation improvements. Transportation modeling techniques continue to evolve with improvements to computer processing speeds and traffic data collection. These improvements allow transportation models to be calibrated to real life traffic conditions. The transportation models rely on an origin-destination (OD) matrix, which describes the quantity and distribution of trips in a transportation network. The trips defined by the OD matrix are assigned to the network through the process of traffic assignment. Traffic assignment relies on the travel time (cost) of roadways to replicate route choice of trips between OD trip pairs. Travel time is calculated both along the roadway and from delay at the intersections. Actuated traffic signals, one form of signalized intersections, have not been explicitly modeled in macroscopic transportation models. One of the objectives of this thesis is to implement actuated signals in the macroscopic modeling framework, in order to improve traffic assignment by more accurately representing delay at intersections. An actuated traffic signal module was implemented into QRS II, a transportation macroscopic model, using a framework from

the 2010 Highway Capacity Manual. Results from actuated intersections analyzed with QRS II indicate the green time for each phase was reasonably distributed and sensitive to lane group volume and input parameters.

Private vendor travel time data from companies such as Navteq and INRIX, have extensive travel time coverage on freeways and arterials. Their extensive travel time coverage has the potential to be useful in estimating OD matrices. The second objective of this thesis is to use travel time in the OD estimation framework. The presented OD estimation method uses travel time to determine directional split factors for bi-directional traffic counts. These directional split factors update target volumes during the OD estimation procedure. The OD estimation technique using travel time from floating car runs was tested using a mid-sized network in Milwaukee, WI. The analysis indicates applicability of using travel time in OD estimation.

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CHAPTER 1: INTRODUCTION

1.1: Background

Transportation macroscopic modeling is a tool for analyzing and prioritizing future transportation improvements. Transportation forecasting and modeling techniques continue to evolve with improvements to computer processing speeds and traffic data collection. These improvements allow transportation models to be calibrated to real life traffic conditions. The better the model is calibrated the more capable it is to analyze changes to transportation infrastructure. Traffic models are calibrated for road intersections with information such as approach lane geometry and traffic control plans; calibrated on the road segments with data such as capacity and travel speed; and calibrated for volumes in the transportation network by creating an OD matrix.

For transportation planning models to evaluate the performance of transportation infrastructure, an OD matrix is required. An OD matrix describes the quantity and distribution of trips in a transportation network. The OD trip matrix is assigned to the traffic network with vehicles using routes that minimize their travel costs between the given OD pair. The OD matrix is often derived using traffic counts and a pattern OD matrix, but new techniques have been developed, which utilize alternative traffic data such as path flows. Utilizing additional traffic data beyond traffic counts and pattern OD matrices provides more information about the traffic volumes and patterns in the network, which then has potential for deriving more accurate OD matrices.

1.2: Purpose

The purpose of this thesis is to investigate the use of actuated traffic signals in macroscopic traffic models and the use of travel time in OD matrix estimation.

1.2.1: Actuated Signals in Macroscopic Models

Previously, actuated traffic signals have not been implemented in macroscopic models and instead fixed time signals or adaptive signal algorithms have represented these intersections. The type of traffic control devices affects the amount of delay for vehicles at intersections. By explicitly modeling actuated signals in the macroscopic model there is potential for improving intersection delay results by more accurately setting the intersection cycle length and green split times. Adjusting the cycle lengths and green splits of an intersection has direct impact on route choice due to change in link travel time.

1.2.2: Travel Time in OD Estimation

Travel time from private sector vendors, such as Navteq, provide large scale travel time coverage for many arterials and freeways. The availability and extensive coverage of the private sector travel time data has the potential to be effective in OD estimation and model calibration. Previously, travel time has not been used to estimate OD matrices. This is most likely due to the expense and the amount of time needed to collect detailed travel time data. The second purpose of this thesis is to develop and test a technique for using travel time data in the OD estimation procedure as well as analyzing the effectiveness of the Navteq database in the proposed OD estimation framework.

1.3: Objectives and Scope

Both of the topics presented in this thesis were developed in collaboration with Dr. Alan Horowitz. Dr. Horowitz is the developer of the QRS II macroscopic software; he programmed both the actuation module and the OD estimation technique using travel time into the QRS II software. My personal roles in both topics of this thesis are detailed in the next two sections. Both of the thesis topics were analyzed using a mid-sized network in Milwaukee, WI.

1.3.1: Actuated Signals in Macroscopic Models

This thesis will present the development of the actuation procedure for the QRS II macroscopic model. The QRS II actuation module adopts the HCM 2010 Chapter 31 actuation procedure for calculating average cycle length and green splits times. The elements covered for the actuation procedure in the macroscopic model include:

- Assumptions for implementing HCM actuation into QRS II
- Analyzing two intersections in QRS II with actuation and testing sensitivity to adjusting passage time, min/max green time, and detector length.
- Analyzing the convergence of the actuation procedure for the individual intersections.
- Analyzing the convergence of the equilibrium traffic assignment with actuated signals.

My personal contribution to the actuation module includes:

- Finding the actuation method in HCM 2010.
- Working with Dr. Horowitz on assumptions for implementation.
- Prototyping the concept of the actuation procedure outside of QRS II.
- Analysis the performance of the actuation procedure.

1.3.2: Travel Time in OD Estimation

This thesis will analyze the effectiveness of using travel time in conjunction with traffic counts and a pattern OD matrix to estimate an OD matrix. The topics discussed in the thesis include:

- The manipulations of the BPR volume-delay function for calculating volume as a function of travel time.
- The development of an algorithm for using travel time in OD estimation to set directional split factors for bi-directional traffic counts.
- Comparison of modeled travel time to Navteq and floating car travel time data.
- The effectiveness of Navteq and floating car travel time in the OD estimation framework.
- Performance of the travel time algorithm to set directional split factors during the OD estimation.

My personal contribution to the OD estimation algorithm using travel time includes:

- The review of volume-delay function
- Choosing a volume-delay function for solving the inverse – volume as a function of travel time
- Finding a method to statically estimate the inverse volume-delay function
- Analyzing the travel time data sources
- Testing the OD algorithm once Dr. Horowitz programmed it into QRS II.

CHAPTER 2: LITERATURE REVIEW

2.1: Four Step Transportation Planning Model

2.1.1: Overview

One of the most common ways for creating an OD matrix for use in a macroscopic transportation model is through the four-step transportation planning method. The four-step method uses socio-demographic census information (among other inputs) along with a detailed transportation network to forecast the quantity, location, travel route, and mode choice of travel for trips in a geographic area. This is done through four steps: trip generation, trip distribution, mode split, and trip assignment.

2.1.2: Trip Generation

Trip generation determines the quantity of person-trips that are produced and attracted in a geographic zone for given trip purpose. The trip purposes include: home based work (HBW), home based non-work (HBNW), and non-home based (NHB). Generated trips are a function of land use and socio-demographic information. The socio-demographic information is provided by the Census department and is aggregated to varying geographic areas with the most useful for transportation purposes being the TAZ (Transportation Analysis Zones).

2.1.3: Trip Distribution

Using person trips generated in the first step, trip distribution matches the trip origins with destinations. The Gravity model is the most popular model for this step:

$$T_{ij} = K_i K_j T_i T_j f(C_{ij})$$

$$\sum_j T_{ij} = T_i, \sum_j T_{ij} = T_j$$

$$K_i = \frac{1}{\sum_j K_j T_j f(C_{ij})}, K_j = \frac{1}{\sum_j K_i T_i f(C_{ij})}$$

where:
T_{ij} = Trips between origin i, and destination j
T_i = Trips originating at i
T_j = Trips destined at j
C_{ij} = Travel Cost between i and j
K_i, K_j = balancing factor solved iteratively
f = distance decay factor

The Gravity model considers the distance, travel cost and the attractiveness of a destination to determine how trips are distributed in a network.

2.1.4: Mode Split

The mode split step determines the proportion of person trips that uses each available mode of transportation (e.g. personal vehicle, transit, bicycle, walk, etc.) to travel from an origin to a destination. One popular formulation for calculating mode split is the logit model. The logit model compares the utility (advantage and disadvantage of using a mode) and then calculates the proportion of trips from an OD pair using each of the available modes.

2.1.5: Traffic Assignment

Traffic assignment is the process of replicating the driver's choice path when traveling from an origin to a destination. The acceptable methods vary by traffic conditions and area type. A critical output of traffic assignment procedures is the proportion of trips from an origin (i) to a destination (j) using path (k). Three popular assignment techniques are all- or-nothing, user equilibrium, and stochastic traffic assignment.

All or nothing traffic assignment:

Also known as shortest path assignment; all-or-nothing assignment routes all vehicles on the shortest path (lowest travel costs) between an origin/destination pair without considering congestion. This approach is feasible in rural areas with minimal routes choices and little to no effects from congestion, but in urban areas with multiple path choices and congestion; user equilibrium and stochastic assignment are more appropriate.

User equilibrium assignment:

User equilibrium assignment is based on Wardrop's first principle (Wardrop, 1952) that states that network user equilibrium is reached when no driver can reduce their travel costs by changing their path. Wardrop's first principle assumes that there is perfect real time information available to the driver. This assignment takes into consideration congestion and produces realistic assignments.

Stochastic assignment:

Stochastic assignment, also based on Wardrop's first principle, assumes that drivers choose paths that to the best of their knowledge will minimize travel costs. This technique is based on the fact that there is imperfect route choice information as well as a number of drivers who are unfamiliar with the area. This assignment technique, if calibrated correctly, more closely resembles the actual behaviors of drivers than the equilibrium assignment.

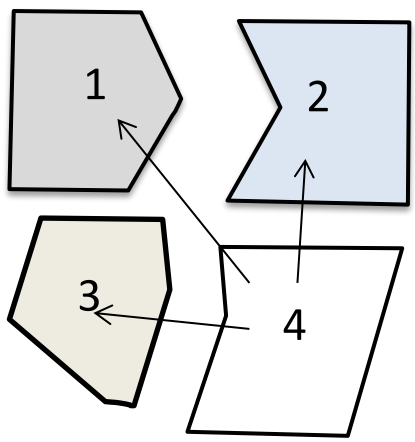
Area-spread traffic assignment:

QRS II implements a technique that creates many paths between an origin and a destination, where each path starts and ends at intersections within traffic analysis zones. This technique overcomes the imprecision in path building that can occur because of large zones. Area-spread traffic assignment can also satisfy Wardrop's first principle.

2.2: Synthetic Origin Destination Estimation

2.2.1: Overview

The OD matrix describes the quantity and distribution of trips in a transportation network. It is formed from travel surveys, 4-step transportation method, or synthetically using origin-destination estimation methods. To understand the structure of the OD matrix, a short example showing the OD matrix associated with geographic zones is represented in Figure 1. The left portion of the figure shows four zones of a transportation network with the arrows representing trips with origin in zone 4 and destinations in zone 1, 2, and 3. These trips are represented by T_{41}, T_{42}, and T_{43}. The table on the right portion of the figure shows the OD table that is representative of the four zones.



		Destination			
		1	2	3	4
Origins	1	T_{11}	T_{12}	T_{13}	T_{14}
	2	T_{21}	T_{22}	T_{23}	T_{24}
	3	T_{31}	T_{32}	T_{33}	T_{34}
	4	T_{41}	T_{42}	T_{43}	T_{44}

Figure 1: Example of Origin-Destination Table

Synthetic OD estimation utilizes field measurements and a reference OD matrix. The reference OD matrix can be from sources such as a 4-step transportation planning model or travel survey. Synthetic OD estimation uses mathematical approaches to solve an OD matrix, which best replicates the field measurements while maintaining a good fit to the reference OD matrix. The most popular mathematical approaches are: generalized least squares, maximum likelihood, Bayes inference, and Kalman filtering. A literature review of static estimation approaches using traffic counts is outlined in Torgil

Abrahmsson's "Estimation of Origin-Destination Using Traffic Counts – A Literature Review", one of the formulations, generalized least squares is briefly described in a later section. The general minimization framework using traffic counts and target OD matrix can be describes mathematically as:

$$\min F(g,v) = \gamma_1 F_1(g,\hat{g}) + \gamma_2 F_2(v,\hat{v})$$

where:

v = assign(g)
$\hat{g}$ = target OD matrix
$\hat{v}$ = observed traffic counts
$F_1()$,$F_2()$ are distance measure between arrays

The goal of the OD estimation is to minimize the distance between the target OD and estimated OD matrix and between the observed and assigned traffic volumes. Assigned volumes are created through traffic assignment procedure. Traffic assignment either can be done exogenously from the OD estimation with a constant traffic assignment map or can be done iteratively with the OD minimization procedure. The iterative approach typically uses a bi-level structure where the upper problem solves the OD minimization procedure and the lower problem solves an equilibrium traffic assignment. After the OD table is created it is reassigned to the network using travel time cost calculations from the past iteration. The iterations need to continue until the minimization function stabilizes (Abrahamsson, 1998). QRS II software is capable of both constant traffic assignment and iterative bi-level OD estimation.

2.2.2: Static Verse Dynamic OD Estimation

A static OD estimation produces one OD matrix for a defined period of time. The OD matrix does not describe fluctuations in traffic demand or differences in OD patterns over time. A static OD matrix is appropriate for long range planning purposes. However, for engineering applications such as designing a reasonably optimal signal plan or modeling diversion, a more temporally detailed OD table is desired. The dynamic OD table consists of many short time interval static tables that are combined for a larger study period. Depending on the size of the transportation network and the amount of data available these tables can be broken into 5-minute, 10-minute, 15-minute, or longer time increments.

2.2.3: Synthetic OD Estimation Using Generalized Least Squares

One of the most popular mathematical frameworks for solving synthetic OD estimation problems is generalized least squares. With the generalized least squares approach it is assumed that the target OD matrix $\hat{g}$ is obtained for the true OD matrix g with a probabilistic error of η and the observed traffic counts $\hat{v}$ is obtained from the real traffic volumes, which is a function of the true OD matrix, with a probabilistic error ϵ.

$$\hat{g} = g + \eta$$

$$\hat{v} = v(g) + \epsilon$$

Given that the target OD matrix has error with a variance-covariance matrix Z and the traffic counts with a dispersion matrix W, and the OD matrix constrained to positive values, the GLS estimator can be obtained from the following expression (Abrahamsson, 1998).

$$min\ \frac{1}{2}(\hat{g}-g)'Z^{-1}(\hat{g}-g)+\frac{1}{2}(\hat{v}-v(g))'W^{-1}(\hat{v}-v(g))$$

s.t. $g_{ij} \geq 0$

2.2.4: Path Flow Data for OD Estimation

The traditional data source for synthetic OD estimation includes traffic volumes and the pattern OD matrix. The latest research in OD estimation involves the use of path flow data. Path flow data is normally collected from in-car GPS units and automatic vehicle identification systems. These systems record a vehicle's movement within a transport network. The data records include partial origin/destinations, route choice, and time of travel. The way this data is processed in the OD estimation procedure varies. Some researchers use the path flow data directly to produce a constant traffic assignment map based on the sample of path flows. Other researchers use an indirect method, which places the path flow within a traditional minimization routine that minimizes the distance between traffic counts, path flows, and seed OD table. Below is a literature review of the use of path flows in OD estimation.

The estimation of a time-dependent OD trip table with vehicle trajectory samples – Hyunmyung Kim and R. Jayakrishnan

The researchers used a path-based OD estimation procedure that relies on vehicle trajectory samples for supplementary data. The trajectory samples are used to build a constant dynamic traffic assignment map; with this method there is no need for a historic OD matrix or traffic assignment model. The model formulation is a bi-level structure with the upper problem using a maximum likelihood function to identify the pattern and size of the OD trips. The lower problem is a generalized least square function which finds path flows and assignment map that minimizes the deviation between observed and calculated traffic counts. The model was tested in numerical examples and showed applicability in real size networks.

Dynamic Origin-Destination Demand Estimation Using Automatic Vehicle Identification Data – Xuesong Zhou and Hani Mahmassani

The authors use automatic vehicle identification to estimate OD point to point split fractions for the population. Their model uses the path data indirectly by inputting to the minimization routine versus the direct method (constructing a constant assignment map). The model is a bi-level dynamic OD estimation framework to minimize the combined deviation with respect to historic OD matrix, link traffic counts, and AVI split fractions. Their minimization function for the AVI split fractions takes into account AVI identification and representativeness errors. They tested their model with synthetic data and made key findings on applicability and reliability of using AVI counts for OD estimation.

Trip matrix and path flow reconstruction and estimation based on plate scanning and link observations – Enrique Castillo, Jose Menendez, Pilar Jimenez

The authors look at constructing OD matrices from a combination of link traffic counts and path flow samples obtained from license plate scanning on selected links. They also propose a method for determining the minimum number of link locations to perform plate scanning to achieve accurate path flow samples as well as analyzing the effects of errors in plate scans on the resultant OD estimation. The model formulation for OD estimation is a quadratic objective function with the weighted sum of the differences between the predicted and the prior path flows. This formulation includes observations from plate scanning and link counts.

Dynamic Origin-Destination Flow Estimation Using Cellular Communication System – Keemin Sohn and Daehyun Kim

The authors propose a new approach for dynamic OD estimation using cell phones as traffic probes. Their method uses a cellular network which consists of interconnected cells with set geographic boundaries. When a vehicle crosses a boundary of a cell the time is recorded. So with this data source they are able to construct a traffic assignment map based on times and paths of vehicles in the cellular network. The authors tested their approach using the phone probe path data with the Kalman filter and generalized least square estimator with varying scenarios (market penetration rate, cell dimension, and cell boundary). They also tested their approach against the conventional path based data from AVI counts and the traditional use of historical OD matrices. Their findings revealed that the AVI counts produced the best estimates, followed by the probe phone data, and lastly the historical OD matrix.

2.3: Calculating Travel time

2.3.1: Overview

In a traditional traffic assignment procedure in which all road segments are non-tolled, travel time is the main factor for determining vehicle routes between trip OD pairs. Travel time is computed for two separate entities: the link travel time and the node travel time (or delay). The link travel time is acquired when a vehicle is traveling between intersections. Node travel time is the time needed to clear a traffic controlled intersection. The travel time on a link is a function of the volume to capacity ratio. The travel time through a node is a function of the type of traffic control and the priority of the traffic movement. The priority of a traffic movement is determined by the volume at the subject and adjacent approaches.

The QRS II planning model uses both link delay functions and node delay function for calculating route travel time for trip assignment. Other traffic planning models do not explicitly calculate node delay. Instead they calibrate the link travel time function to also represent the time needed to clear intersections. The following two sections will discuss link delay and node delay and the most popular formulations for both forms.

2.3.2: Link Delay

There have been many formulations in the literature for calculating travel time (speed) with respect to volume using a volume to capacity ratio. The most popular of these formulations are the Bureau of Public Roads curve, Spiess volume-delay function, and Akcelik functions. The basic characteristics of the volume delay functions are the same. When Spiess created his conical volume delay function he defined what is needed in an alternative function to the BPR curve; the requirements he listed include:

- Travel time as a function of volume is strictly increasing.

- Capacity is defined as the volume at which congested speed is half the free flow speed.

- The first derivative of the function is strictly increasing.

- $f'(1) = \alpha$. The parameters that defines how sudden the congestion effects change when capacity is reached.

- $f'(x) < M\alpha$, where M is a positive constant. The steepness of the congestion curve is limited; therefore, travel times will not get too high when considering v/c ratios greater than 1.

- $f'(0) > 0$. This condition guarantees uniqueness of the link volumes. It also renders the assignment stable regarding small coding errors in travel time and distributes volume on competing uncongested paths proportional to their capacity.

- The evaluation of f(x) should not take more computing time than the evaluations of the corresponding BPR functions (Spiess, 1990).

Using Spiess' requirement, Dowling and Skabardonis compared different functional forms and their acceptability to relate volume and travel time in their article "Urban Arterial Speed Flow Equations for Travel Demand Models." They compared generic functional forms to popular volume delay functions (BPR and Akcelik). Their comparison of the function forms is shown in Table 1. Note: in the figure S is speed, S_o is free flow speed, V is volume, C is capacity, a and b are empirical constants.

	Functional Form	Comments
Linear	$S = -a\dfrac{V}{C} + b$	Not acceptable. Reaches zero speed at high v/c
Logarithmic	$S = -aln\left(\dfrac{V}{C}\right) + b$	Not acceptable. Has no value at x = 0 (the logarithm of "x" approaches negative infinity)
Exponential	$S = as_0e^{-b\frac{V}{C}}$	Has all required traits for equilibrium assignment.
Power	$S = -\dfrac{a}{\left(\frac{V}{C}\right)^b}$	Not acceptable. It goes to infinity at v/c = x = 0.
Polynomial	$S = -a\left(\dfrac{V}{C}\right)^2 - b\left(\dfrac{V}{C}\right) + C$	Not acceptable. It reaches zero speed at high v/c.
BPR	$S = \dfrac{S_0}{(1 + a(\frac{V}{C})^b}$	Has all required traits for equilibrium assignment.
Akcelik	$S = \dfrac{L}{\left[\frac{L}{S_0} + .25\left\{\left(\frac{V}{C} - 1\right) + \sqrt{\left(\frac{V}{C} - 1\right)^2 + ax}\right\}\right]}$	Has all required traits for equilibrium assignment.

Table 1: Comparison of Functional Forms to Relate Speed and Volume (Dowling & Skabardonis, 2006).

The comparison in Figure 2, states that linear, logarithmic, power, and polynomial functions in their general forms are unable to relate the relationship between volume and travel time. The only general form function that is acceptable is the exponential function (Dowling & Skabardonis, 2006). Several brief descriptions of popular volume delay functions using the volume to capacity ratio are listed below.

Bureau of Public Roads (BPR) Curve 1965

The BPR function was the first function to relate travel time and volume, which is dependent on the volume to capacity ratio (U.S. Bureau of Public Roads, 1964). The formulation is as follows:

$$t = t_0(1+\propto \left(\frac{V}{C}\right)^{\beta})$$

Where t_0 is the free flow travel time, V is the volume, C is the capacity of the link, and β is an empirical constant. Alpha can be obtained from:

$$\propto = \left(\frac{S_0}{S_c}\right) - 1$$

where: S_0 = free flow speed

S_c = speed at capacity

Spiess Conical

Spiess' conical volume travel time function (Spiess, 1990) is formulated as:

$$t = t_0\{2 + \sqrt{\alpha^2 \left(1 - \frac{V}{C}\right)^2 + \beta^2} - \alpha\left(1 - \frac{V}{C}\right) - \beta\}$$

where: $\beta = \frac{2\alpha - 1}{2\alpha - 2}$

$\alpha \geq 1$

Davidson Equation

Davidson equation (Davidson, 1966) is formulated as:

$$t = t_0\left[1+\left(\frac{J_D\left(\frac{V}{C}\right)}{1-\frac{V}{C}}\right)\right]$$

where J_D is a delay parameter

Akcelik Function

The Akcelik function (Akcelik, 1991) is a time dependent form of the Davidson equation.

$$t = t_0 + \left\{0.25\mathrm{T}\left[\left(\frac{V}{C}-1\right)+\sqrt{\left(\frac{V}{C}-1\right)^2+\frac{8J_a x}{CT}}\right]\right\}$$

$$J_A = \frac{2C}{T}(t_c - t_0)^2$$

2.3.3: Node Delay

The macroscopic planning model QRS II relies on node delay functions in conjunction with link travel time functions for calculating segment travel time. Three types of traffic control can cause node delay: stops signs (all-way or 2 way stop), roundabouts, and traffic signals. Calculating node delay requires information about the traffic control plan such as number and type of lanes by approach and signal timing information. Node delay for signalized and unsignalized intersection is outlined in the 2010 Highway Capacity Manual.

Node delay is calculated for each lane group. HCM 2010 defines the following rules for determining lane groups for an intersection approach:

- Exclusive left-turn or right turn lanes should be designated as separate lane groups.
- Any shared lane should be designated as a separate lane group.
- Any lanes that are not exclusive turn lanes or shared lanes should be combined into one lane group.

The above rules result in the designation of the following lane groups: exclusive left-turn lanes; exclusive through and right turn lanes, shared left/through lanes, shared left/right turn lanes, shared right/through lanes, and shared left/through/right lanes.

2.3.4: Traffic Signal Node Delay

The delay at a signalized intersection is determined using the HCM 2010 control delay of signalized intersections equations. The control delay for an intersection approach has three delay components: uniform delay (d_1), incremental delay (d_2), and queue delay (d_3). The uniform delay assumes uniform arrival of vehicles at the intersection. Incremental delay corrects the uniform delay for the non-uniformity in vehicle arrival patterns. The queue delay is delay incurred from vehicles un-served by a previous analysis period. If there is no queue for a given lane group queue delay equals zero. The delay for all three of these equations is calculated for each lane group. For a full description of signalized control delay calculation see HCM 2010 chapter 18.

Uniform delay is shown below (HCM 2010 Eq. 18-20). The uniform delay is calculated based on the lane groups' green time portion or the cycle and the lane groups' volume to capacity ratio.

$$d_1 = \frac{0.5C\left(1-\frac{g}{c}\right)^2}{1-\left[min\left(1,\frac{V}{C}\right)\frac{g}{C}\right]} \text{ (HCM 2010 Eq. 18-20)}$$

where: C = cycle length (s)
g = effective green time for lane group (s)
V/C=volume to capacity ratio for the lane group

The amount of delay for a lane group is dependent on its green split (g/c), which is related to the amount of volume in the subject, opposing, and conflicting lane groups as well as the type of signal at the intersection (Highway Capacity Manual 2010, Volumes 1-4, 2010). To understand the complexity of the delay acquired at an intersection as a function of all volumes at all approaches it is important to understand the differences in traffic signal timing plans. There are three types of traffic signals: fixed time, actuated, and adaptive control.

Fixed Time Traffic Signals

Fixed time traffic signals are typically most effective in dense urban areas because with heavier traffic volumes the need for synchronization of traffic signals is greater. In addition, with heavier volumes there are fewer occasions where a minor street has no vehicles at a traffic signal. With fixed time signals, the amount of green time designated for each turning movement is fixed over a period of time. A fixed time signal can have multiple signal plans, which are used at different times of the day, based on previously measured traffic volumes. The traffic engineer determines a cycle length which is best for a particular intersection or which is best for signalized corridor. The green time is then distributed depending on the number of phases. A phase consists of turning movements, which are non-conflicting. For example, the northbound and southbound through movements or northbound and southbound lefts can be phases. The number of phases depends on the type of intersection and the amount of traffic per turning movement. The need for a protected left turn phase is a function of left turning vehicles for an approach and the conflicting through traffic. The amount of green time that an approach receives is dependent on the amount and distribution of volumes at an intersection.

Actuated Traffic Signals

An actuated traffic signal has many of the same characteristics as the fixed time signals in regards to phase structure. The only difference is actuated signals can adjust to observed volumes. The base signal plan for an actuated control has minimum and maximum green times, and green extension times. There are detectors in the form of in ground loop detectors or more recently video image detectors. During a phase, the minimum green time is set and additional green time is added for vehicles detected after the minimum green time. The amount of time added for each vehicle detection is the green extension time. This addition of time continues for each vehicle until the maximum green time is reached or when the movement phases out because the time between detections is greater than the green extension time. See HCM 2010 chapter 31 for more information on actuated signals.

Adaptive Signal Control

Adaptive signal control is a new form of traffic signal, which is implemented only in a few locations but is becoming popular. The purpose of this signal control is to optimize the entire road network by looking to optimize cycle lengths, phase lengths, and offsets. This is done by having nearly instantaneous knowledge of vehicle locations in the network and then making changes immediately to fluctuating demand.

2.3.5: Unsignalized Node Delay

Node delay is also calculated for unsignalized intersections, which includes 2-way stop control, all-way stop control, and roundabouts. The delay function for the three types of unsignalized intersection incorporate the volume/capacity ratio for the subject lane group as well as volume in the conflicting approaches to determine approach delay. The control delay for all-way stop is shown below. For more details on unsignalized intersections see HCM 2010 chapter 19 (2-way stop), 20 (all way stop), and 21 (roundabouts).

All-Way Stop Delay

$$d = t_s + 900T\left[(x-1) + \sqrt{(x-1)^2 + \frac{h_d x}{450T}}\right] + 5$$ *(HCM 2010 Eq. 20-30)*

where:

d= average control delay (s/veh)
x = degree of utilization ($vh_d/3600$)
t_s = service time (s)
h_d = departure headway (s)
T = length of analysis period (h)

2.3.6: Control Delay as a Function of Traffic on All Approaches

Node delay for a subject approach is dependent on volumes at all intersection approaches. Horowitz (1991) demonstrated the volume-travel time relationship in signalized corridors by using the HCM control delay functions in QRS II on four legs of an intersection. By setting constant volumes on three of the approaches he varied the subject approach and observed the change in delay for all approaches based on the volume change of the one. During this analysis the cycle length was fixed, but the green splits were set as a function of volume, which imitates an adaptive or a well-engineered fixed time signal. It was found that the delay for the subject approach and the conflicting approaches increased at similar rates when the volume increased. However, the opposing link had increased delay until the subject approach volume was sufficiently large to call for more green time and then the delay decreased. See Figure 2, for graph representing the delay on all approaches of a signalized intersection as a function of the volume on a single approach. The intersection was modeled with 25% right turning, 25% left turning, 800 vph(vehicles per hour) at the opposing and conflicting approaches, no exclusive lanes, 20 mph speed limit, and 3600 vph ideal saturation flow rate (Horowitz, Delay/Volume Relations for Travel Forecasting Based upon the 1985 Highway Capacity Manual, 1991).

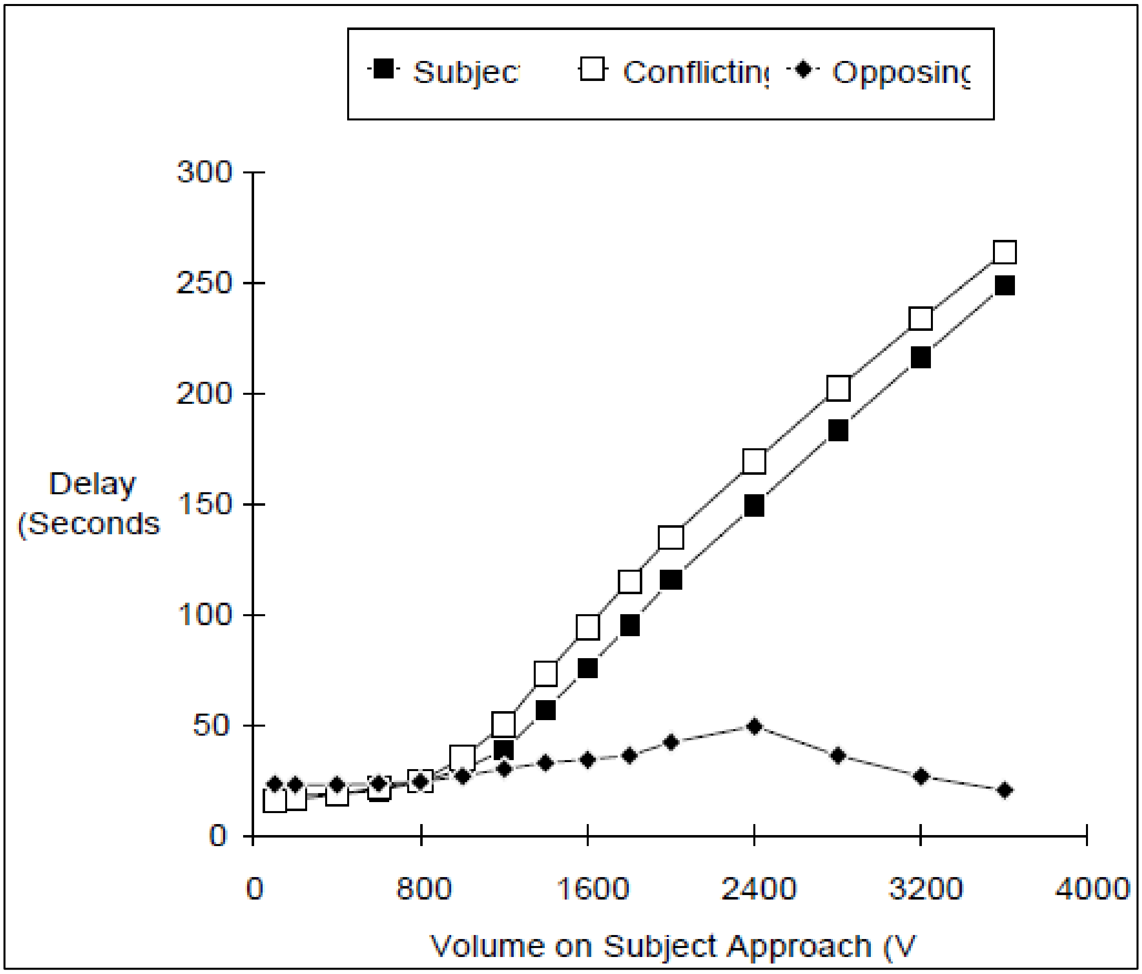


Figure 2: Approach Volume Verse Delay (Horowitz, Delay/Volume Relations for Travel Forecasting Based upon the 1985 Highway Capacity Manual, 1991)

2.4: Travel Time as a Function of Volume

The volume-travel time relationships have been traditionally formulated to solve for travel time given a volume. There is only one instance in the literature where a volume-travel time function is solved inversely for volume given travel time (Yi, Zhen, & Zhang, 2004). The researchers reversed the BPR curve to solve for travel time given volume. However, the study used simplified assumptions that generalized the BPR variables over a range of link characteristics and did not consider volume on the opposing and conflicting link approaches through intersection delay. This framework produces poor results on urban signalized arterials because travel time is a function of the volume/capacity ratio on the subject link as well as the volumes on the opposing/conflicting approaches.

CHAPTER 3: ACTUATED SIGNALS IN MACROSCOPIC MODELS

3.1: Overview

The traffic assignment algorithm within the QRS II framework explicitly calculates intersection delay and adds it to the travel time of the link. This section discusses how actuated traffic signals are incorporated into a macroscopic transportation planning model (QRS II). The actuated signal procedure is based on the HCM Chapter 31: Signalized Intersection chapter. The HCM actuated signal procedure calculates average phase duration and cycle length. The average green phase duration and cycle length is then applied as inputs into the node delay functions. The HCM actuated signal procedure involves many equations with detailed input variables. The number of variables needed to calculate the full procedure would be burdensome in a macroscopic planning model. To overcome this difficulty, assumptions were made to the HCM actuated procedure to simplify implementation into the macroscopic modeling framework. The next section will give an overview of the HCM procedure, the data needs for QRS II actuated signals, and the limitation of the QRS II implementation.

3.2: Actuated Signals in HCM 2010

Chapter 31, subsection of the 2010 HCM outlines a procedure for calculating average phase duration of actuated signals. This procedure mimics the operation of actuated signals by calculating probability of phase calls, phase extension, and max outs. The probability formulas require traffic inputs as well as the structure of the actuated signal. The HCM procedure can handle most types of actuation caused by variations in approach geometries, actuated-coordinated, and split phase timing. The steps for calculating average phase duration and cycle length for actuated signals is shown in Figure 3. Step 1, volume computations, is calculated at the beginning of the procedure and is not repeated. Steps 2 through 5 are dependent on each other, so iterations are required to converge on average phase duration and cycle length for an intersection.

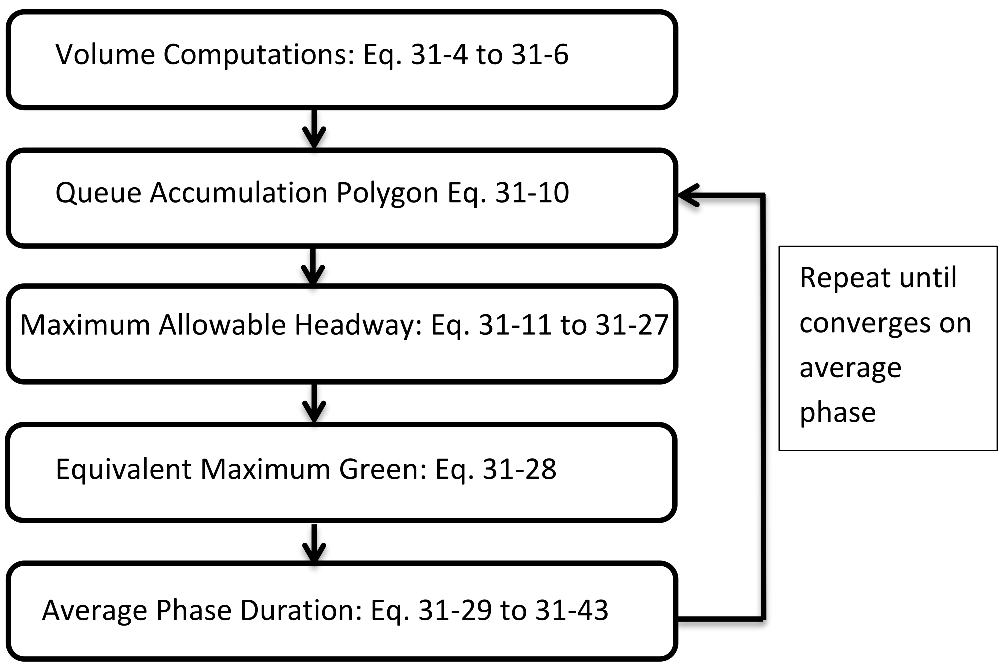


Figure 3: Step from the 2010 HCM for Calculating Average Actuated Phase Duration

The following section will go through each of the five steps and describe the HCM procedure and the assumptions made in the QRS II implementation. Not all of the equations in the HCM procedure were included in the QRS II actuation procedure.

Step 1: Volume computations: Equations 31-4 to 31-6

The volume computation procedure requires the input of lane group volumes (q_i), along with bunching factor variables (b_i) and headway of bunched vehicles (Δ_i) which have default values set in the QRS II actuation signal module. The number of lanes in the lane group defines the bunching and headway factor parameters. The number of lanes for each lane group is defined in QRS II by coding in approach codes for each link that describe the number and type of turns at the intersection approach.

The first part of the volume computation procedure calculates the proportion of free (unbunched) vehicles (φ_i) in each lane group i (HCM 2010 Eq. 31-6). The φ_i is used to calculate the flow rate parameter (λ_i) for each lane group (HCM 2010 Eq. 31-5). The flow rate parameter for the phase is calculated by summing the flow rate parameter for each lane group in the phase (λ^*).

$$\lambda^* = \sum_{i=1}^{m} \lambda_i \quad \text{(HCM 2010 Eq. 31-4)}$$

$$\lambda_i = \frac{\varphi_i q_i}{1-\Delta_i q_i} \quad \text{(HCM 2010 Eq. 31-5)}$$

$$\varphi_i = e^{-b_i \Delta_i q_i} \quad \text{(HCM 2010 Eq. 31-6)}$$

The proportion of free (unbunched) vehicles, equivalent headway of bunched vehicles, and the arrival flow rate are calculated for each phase with HCM 2010 equations 31-7, 31-8, and 31-9.

$$\varphi^* = e^{-\sum_{i=1}^{m} b_i \Delta_i q_i} \quad \text{(HCM 2010 Eq. 31-7)}$$

$$\Delta^* = \frac{\sum_{i=1}^{m} \lambda_i \Delta_i}{\lambda^*}$$ (HCM 2010 Eq. 31-8)

$$q^* = \sum_{i=1}^{m} q_i$$ (HCM 2010 eq. 31-9)

The HCM 2010 equations 31-7 to 31-9 are a summation of lane group's part of the phase as well as the lane group's part of the phase concurring at the same time. All these variables are used in later steps to calculate actuated phase duration. One limitation with the QRS II implementation is that simultaneous gap out is assumed for all phases. So, protected left turns opposite one another and through movements opposite one another will have their phases end at the same time.

Step 2: Queue Accumulation Polygon 31-10

This step calculates the green time needed to dissipate the queue during each phase. The queue is dependent on the cycle length (C), the arrival flow rate of the lane group (q), the saturation flow rate (s) and the proportion of green time given to the phase (P). The proportion of green time and the cycle length will change with iterations. The initial cycle length and phase splits are set based on the maximum green time, and loss time (l_1 + Y + R). Later iterations use the equivalent cycle length and phase green splits calculated at the end of each iteration.

$$g_s = \frac{qC(1-P)}{\frac{s}{3{,}600} - qC(\frac{P}{g})}$$ (HCM 2010 Eq. 31-10)

Step 3: Maximum Allowable Headway: 31-11 to 31-27

The maximum allowable headway (MAH) is calculated for each lane group. It is the maximum time between vehicles without terminating the phase by gap out. See

Figure 4, below for visual of the MAH value for individual lane groups as a function of detector length, vehicle length, passage time, and approach speed.

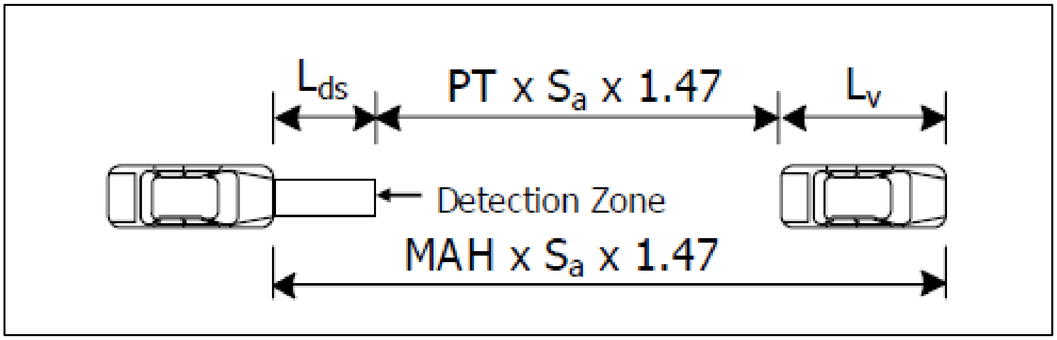


Figure 4: Visual of Maximum Allowable Headway Calculation

The parameters L_{ds} (detector length), L_v average length of vehicle (alias in QRS II = "Jam Density"), and PT (passage time or green extension) are default in the QRS II actuation module and are applied network wide. The detector lengths and passage time can be defined separately for left lanes and through/right lanes. Passage time is the maximum length of time between consecutive cars before the lane group green time expires. HCM assumes only one detector per lane. Because more than one detector is possible for a lane, the user needs to apply best judgment when specifying the detector length. Also, the detectors in QRS II are assumed to be presence detectors. To specify pulse detectors in QRS II set the detector length to zero.

The default vehicle length in QRS II is calculated by HCM 2010 equation 31-12 with the assumed car length of 8 feet, heavy truck length of 45 feet, and vehicle classification with five percent heavy trucks. If these default parameters do not match the study network, Equation 31-12 can be used to adjust the parameter "Jam Density" in the QRS II actuation module. The approach speed S_a is calculated based on the link speed limit (S_{pl}) coded in the GNE network along with HCM 2010 equation 31-13.

$$MAH = PT + \frac{L_{ds}+L_v}{1.47S_a} \quad \text{(HCM 2010 Eq. 31-11)}$$

$$L_v = L_{pc}(1 - 0.01P_{HV}) + 0.01L_{HV}P_{HV} - D_{sv} \quad \text{(HCM 2010 Eq. 31-12)}$$

$$S_a = 0.90(25.6 + 0.47S_{pl} \quad \text{(HCM 2010 Eq. 31-13)}$$

The HCM procedure gives MAH calculations for seven different types of lane groups (Eq. 31-14 to 31-20). The QRS II procedure only allows for 3 of those types: exclusive protected left (31-15), through movement (Eq. 31-14), and left exclusive permitted (31-17). QRS II skips the last 3 equations (31-18 to 31-20), which deal exclusively with right turns because right turns receive the same treatment as through movements.

In QRS II, one left exclusive lane is treated as protected-permitted mode, and greater than two left exclusive lanes are protected only. Protected verses permitted left turns in QRS II are based on volume warrants for protected left turns. If the left turn volume, and through opposing volume do not warrant a protected left turn QRS II will default to actuation with shared left/through green time.

Once the MAH is calculated for each lane group an equivalent MAH (MAH^*) is solved for lane groups part of the same phase or part of the phase that runs concurrently. Because QRS II assumes simultaneous gap out for all actuated signals, HCM 2010 equations 31-21 to 31-26 are not needed. Equation 31-27 is used to calculate MAH^* for intersection modeled in QRS II.

$$MAH^* = \frac{MAH \sum \lambda_i + MAH_c \sum \lambda_{c,i}}{\sum \lambda_i + \sum \lambda_{c,i}}$$ (HCM 2010 Eq. 31-27)

Step 4: Equivalent maximum green: 31-28

The equivalent maximum green step in the HCM procedure calculates the equivalent maximum green time when the signals are coordinated-actuated. QRS II implementation assumes uncoordinated signals for the timing calculations. Therefore, this step is not applicable for QRS II actuation.

Step 5: Average phase duration 31-29 to 31-43

Step 5 calculates a number of probabilities associated with the length of the phase. It uses the variables calculated in steps 1 through 4. The first equation calculates the number of extension before a phase will max out (HCM 2010 Eq. 31-29). This equations is based on the phase flow rate (q^*), maximum green (G_{max}), loss time (l_1), and green time needed to dissipate the queue (g_s). The maximum green times for actuated signals are coded in link attributes within the GNE network.

$$n = q^*[G_{max} - (g_s + l_1)] \geq 0.0$$ (HCM 2010 Eq. 31-29)

HCM 2010 equation 31-30 calculates the probability of a green extension (p).

$$p = 1 - \varphi^* e^{-\lambda^*(MAH^* - \Delta^*)}$$ (HCM 2010 Eq. 31-30)

HCM 2010 equation 31-31 uses the number of extensions before max out (n) and the probability of an extension (p) to calculate the average green extension time for each phase.

$$g_e = \frac{p^2(1-p^n)}{q^*(1-p)}$$ (HCM 2010 Eq. 31-31)

HCM 2010 equations 31-32 to 31-34 are used to calculate the probability of a phase call. These are applicable for signals that are semi-actuated and have no minimum or maximum recall. For the QRS II actuation, signals are assumed to be fully actuated. The probability of a recall (P_c) is then equal to one, because each phase will occur once during every cycle. This is a limitation of the QRS II procedure; however, semi-actuated signals may be properly modeled with QRS II adaptive signal control algorithm.

The unbalanced green duration is calculated for each phase using HCM 2010 equations 31-35 to 31-37. The no pedestrian, and fully actuated assumptions in QRS II simplifies these equations as $G_u = \max(l_1 + g_s + g_e, G_{min}) \leq G_{max}$.

$$G_u = G_{|veh,call} p_v (1 - p_p) + G_{|ped,call} p_p (1 - p_v) + max[G_{|veh,call}, G_{|ped,call}] p_v p_p \leq G_{max}$$

(HCM 2010 Eq. 31-35)

with:

$$G_{|veh,call} = max \begin{bmatrix} l_1 + g_s + g_e \\ G_{min} \end{bmatrix}$$ (HCM 2010 Eq. 31-36)

$$G_{|ped,call} = Walk + PC$$ (HCM 2010 Eq. 31-37)

The unbalanced phase duration (HCM 2010 Eq. 31-28) is equal to the sum of the unbalanced green duration plus yellow and red time.

$$D_{up = G_u + Y + R_c}$$ (HCM 2010 Eq. 31-38)

HCM 2010 Equations 31-39 to 31-41 are used to calculate the average phase duration. Because QRS II assumes simultaneous gap out for all phases these equations are not implemented. The purpose equations 31-39 to 31-41 is to calculate phase duration based on other phases gaping out early and leaving extra green time for the subject phase. QRS II assumes $D_{up} = D_p$.

The green interval duration is calculated from HCM 2010 equation 31-42, which subtracts the yellow and red time from the average phase duration.

$$G = D_p - Y - R_c \quad \text{(HCM 2010 Eq. 31-42)}$$

The equilibrium cycle length is calculated by summing all phase durations in ring one.

$$C_e = \sum^4 D_{p,i} \quad \text{(HCM 2010 Eq. 31-43)}$$

The equilibrium cycle length is carried to the Step 2 to start a new iteration that continues through iterations until the equilibrium cycle length converges to the input cycle length. The HCM 2010 procedure recommends using 0.1 s difference as the convergence criteria (Highway Capacity Manual 2010, Volumes 1-4, 2010). QRS II does not have convergence criteria, but allows for a set number of iterations to be specified in actuation module. The default number of iterations is six, which has proved to provide adequate convergence for typical actuated traffic signals.

CHAPTER 4: TRAVEL TIME FOR OD ESTIMATION

4.1: Overview of Methodology

Techniques for OD estimation mentioned in the literature review have utilized traffic volumes, path flows, and target OD matrices. However, within the literature there is no mention of travel time data. This could be partially due to the past difficulty in acquiring accurate and complete travel time data. GPS data from mapping vendors such as Navteq has the potential to provide accurate and complete travel time data that could be applied in OD estimation. One of the objectives of this thesis is to develop an approach to utilize travel time data to aid the OD estimation procedure.

The proposed OD estimation framework uses travel time data to split observed bi-directional traffic counts to produce directional traffic count targets for the minimization procedure (minimize distance between observed traffic counts and assigned traffic volumes). In order for the travel time to be used for this procedure, a relationship between volume and travel time needs to be developed for each link direction that has a bi-directional traffic count. Below is a flow chart showing how estimating split factors for bi-directional traffic counts fits into the proposed OD estimation framework.

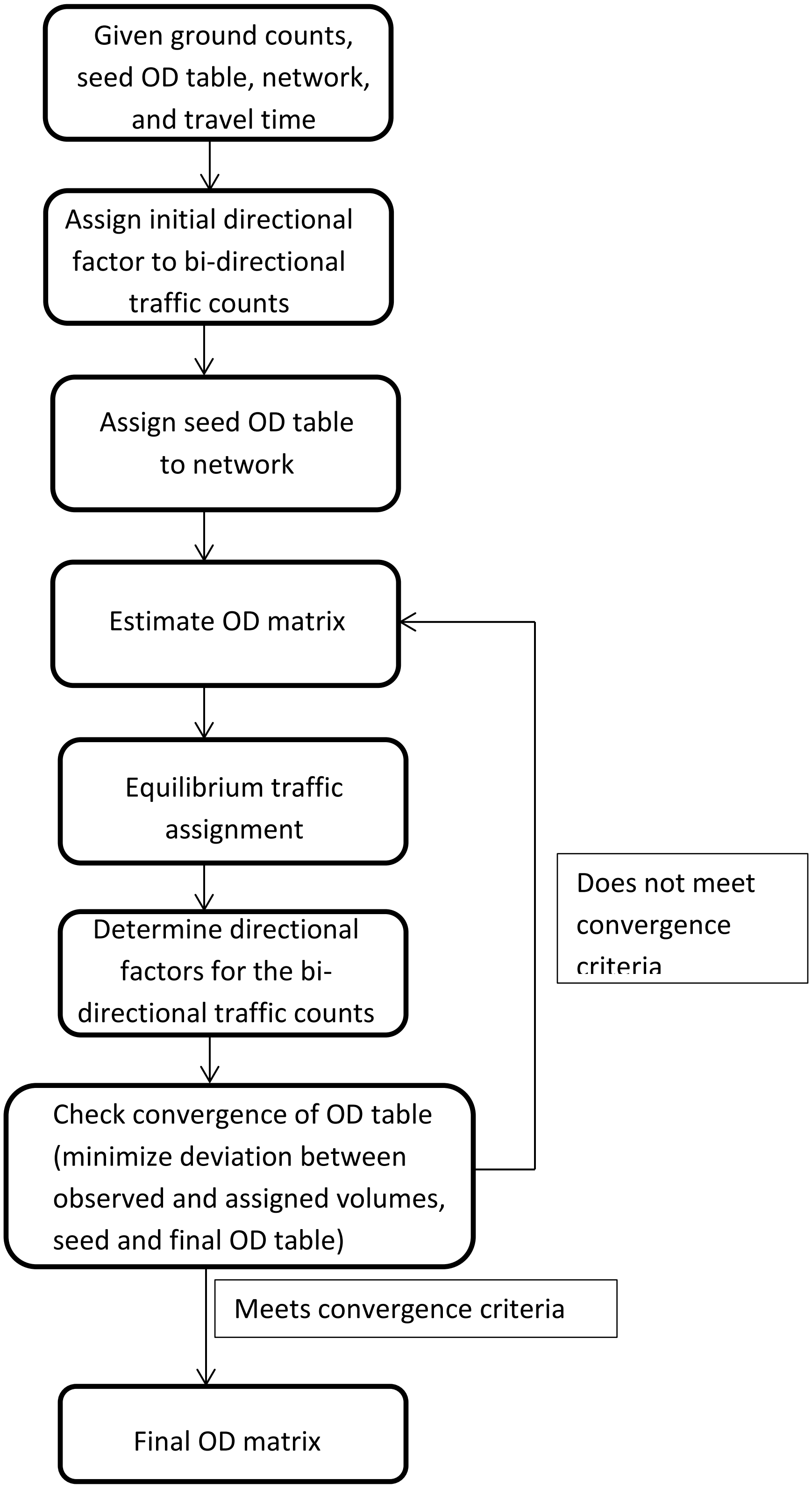


Figure 5 OD estimation using Travel Time to Set a Directional Factor for Bi-Directional Traffic Counts.

As shown in Figure 5, the proposed OD estimation using travel time follows a traditional iterative bi-level framework were the OD table is estimated and then assigned to the network. The procedure starts with the seed (reference) OD matrix, traffic counts, and directional travel time. The traffic counts measured as a total of both directions of travel are given an initial split factor based on knowledge of the corridor.

To start the OD estimation the seed OD table is assigned to the network. From this assignment each link direction has an assigned traffic volume. Using the assigned link volumes and the traffic counts a new OD table is created to minimize the difference between the two volume arrays. Once this new table is estimated, it is then reassigned to the network. Using the assigned volume, observed travel time, and modeled travel time a directional factor is determined for the bi-directional traffic counts. After the bi-directional traffic counts are split, there is a check for convergence of the OD matrix by comparing the change of the objective function from the previous iterations. If the objective function has not converged, then the OD estimation, assignment, and split factor calculations are repeated until there is convergence or until a predetermined number of iterations are completed.

4.2: Formulation to Solve Volume Given Travel Time

The travel time along a link and at the node is a function of the subject, cross, and opposing volumes. To overcome this complex relationship a modeling procedure is developed to solve link volume given link travel time. The modeling procedure creates a dataset of modeled travel time and modeled volumes on the subject approach by varying the modeled volume on all approaches. Having different volume combination at the intersection produces different delay for the subject approach because the allocation of green time will vary as volume on all approaches varies. The link and node model inputs (fixed time or actuated, cycle length, green passage, min/max green, capacity, etc.) and the volume at each approach determines the intersection green splits. To create the dataset of modeled travel time and volume, 81 volume/travel time data points are recorded for each link direction. Modeled volume-travel time data points are created by varying the assigned volumes on a link from the previous equilibrium iteration +-40 percent. For any modeled data point the subject, opposing, and crossing links can have volume of minus 40 percent of the assigned volume, the assigned volume, or plus 40 percent of the assigned volume. The 81 observations ($81=3^4$) make sure that each volume combination is sampled for every intersection with four approaches (see Figure 6).

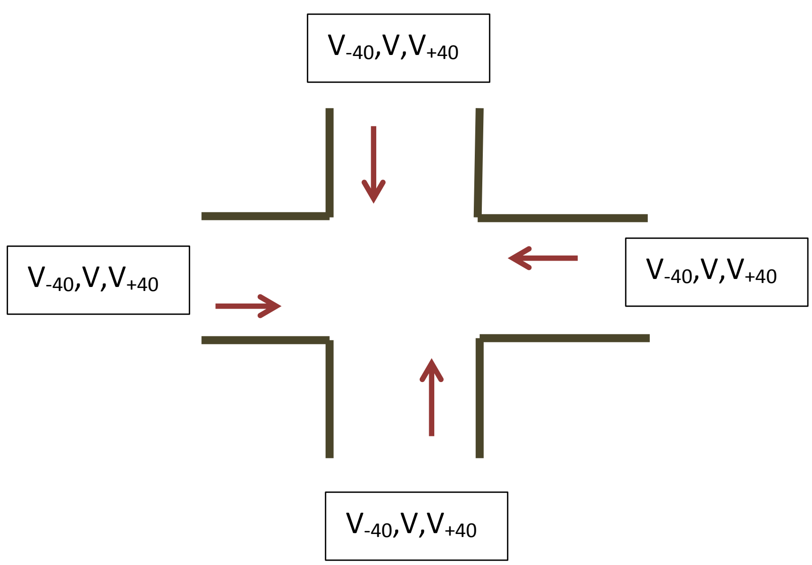


Figure 6: Dataset of Modelled Travel time and Volume Combinations

4.3: Functional Form

The BPR function was chosen to relate volume as a function of travel time for the 81 modeled travel time/volume data points. The BPR curve was chosen because it has the correct functional form, volume is zero at free travel time, and the inverse (V as a function of travel time) can be solved in closed form. The reverse BPR curve is as follows:

$$V = C\left(\frac{t - t_0}{\propto t_0}\right)^{1/\beta}$$

Generally, when using the BPR curve or other Volume-Delay functions it assumed that the capacity for a link is known. However, in this instance, the function is representing both the link and node delay. Therefore, the capacity becomes a function of the opposing, cross, and subject link volumes. To best represent the volume-travel time function for each link the variables of the BPR curve are estimated on an individual link level by fitting the BPR function through the 81 data points.

The alpha variable of the BPR function is removed from the formulation because it has little effect on the functional form of the equation when the other variables are optimized for each link. To simplify the equation further the 1/β term equals a new variable λ, which prevents divide by zero errors during the non-linear regression. The functional form becomes:

$$V = C\left(\frac{t - t_0}{t_0}\right)^{\lambda}$$

where: t = travel time on link

t_0 = free travel time

C, λ => optimized to fit individual link data points

The parameters C and λ are estimated using non-linear regression. Then individual volume-travel time curves are fit to data points modeled for each link direction.

Free Travel Time

Free travel time is the time needed to traverse a link and a node when there is no volume on the network, but all existing intersection traffic control devices are in place. The free travel time is calculated using the traffic signal green split times from the previous equilibrium traffic assignment. The recorded travel time for the subject approach is calculated using a new signal-timing plan based on the assigned volumes from the current equilibrium. The green split times will change with volume because the signal timing algorithms in the model seeks to set the signal-timing plan given the volumes on each approach. Because the free travel time is not fixed, it is possible that the modeled travel time for any of the 81 data points can be less than the free travel time. This situation would create an error during the non-linear regression because the modified BPR formulation has the difference in free and recorded travel time raised to a power ($(t_0 – t)^{\lambda}$). If the free travel time is greater than recorded travel time than the function cannot be evaluated because a negative value is raised to a non-integer value. To correct this problem, in the case where free travel time is greater than recorded travel time, the free time is adjusted to be 0.01 seconds less than the lowest recorded modeled travel time for the link direction; this allows the model to continue with its equilibrium iterations. The adjustment of free time should have minimal to no consequences on the final output because in most cases this situation will occur in early iterations of equilibrium traffic assignment were the volume changes on a link may be large enough to cause this anomaly.

4.4: Non-linear regression for C and λ

To solve C and λ a non-linear regression routine was obtained from the ALGIB library. The ALGLIB library is a cross-platform numerical analysis and data processing library (ALGIB). The ALGIB source code was modified and implemented into QRS II to solve C and λ when given 81 volume/travel time samples. The non-linear regression routine uses the first and second partial derivate of the modified BPR function to perform a search procedure for the optimal C and λ which minimizes the distance between observed volumes from the modeling procedure and the volume obtained from the derived function. Functions used within the non-linear regression routine are as follows:

Base equation: $V = C\left(\frac{t_1 - t_0}{t_0}\right)^{\lambda}$

First partial derivative with respect to C: $\frac{\partial V}{\partial C} = \left(\frac{t_1 - t_0}{t_0}\right)^{\lambda}$

First partial derivate with respect to λ: $\frac{\partial V}{\partial \lambda} = Cln\left(\frac{t_1 - t_0}{t_0}\right)\left(\frac{t_1 - t_0}{t_0}\right)^{\lambda}$

Second partial derivate with respect to C: $\frac{\partial^2 V}{\partial C^2} = 0$

Second partial derivative with respect to λ: $\frac{\partial^2 V}{\partial^2 \lambda} = -Cln\left(\frac{t_1 - t_0}{t_0}\right)^2\left(\frac{t_1 - t_0}{t_0}\right)^{\lambda}$

Second partial derivative with respect to C and λ: $\frac{\partial^2 V}{\partial\lambda\partial C} = ln\left(\frac{t_1-t_0}{t_0}\right)\left(\frac{t_1-t_0}{t_0}\right)^{\lambda}$

Once the functions parameters are derived for a specific link using the modeled volume/travel time dataset (81 samples) the function needs to be tested if it adequately relates to the dataset. The function check has three parts. First, the goodness of fit is tested to determine if the function produces a clear relationship between the modeled data points. The goodness of fit is calculated using the average relative error.

$$Relative\ Error = \frac{Observed - Calculated}{Observed}$$

From visual inspection of the modified BPR function against the 81 modeled travel time/volume samples it was determined that links with average relative errors above 0.3 need to be discarded because the data shows no trend that could be used to estimate volume given travel time. Secondly, the function need to be discarded if capacity is less than zero. A capacity less zero will cause the function to calculate a volume that is also less than zero. Thirdly, if λ variable is calculated as greater than 1 or less than 0.05 the function in not valid. If λ variable is greater than one, the result is a power function, which is contrary to the relationship between volume and travel time. If λ variable is less than 0.05 then the function becomes too flat with minimal to no volume increase given an increasing travel time.

4.5: Examples of Modified BPR Functional Form and Modeled Data Sets

Due to the sampling scheme of taking +-40% of the assigned volume the effects of increasing minor cross traffic has minimal effect on the subject link. This case is shown in Figure 7, were the 81 data points stack in positions on three levels of the subject link volume. In this example the cross traffic causes minimal to no change in the green split for the subject approach. The change in travel time that is shown is occurring mostly from a higher volume to capacity ratio on the subject link.

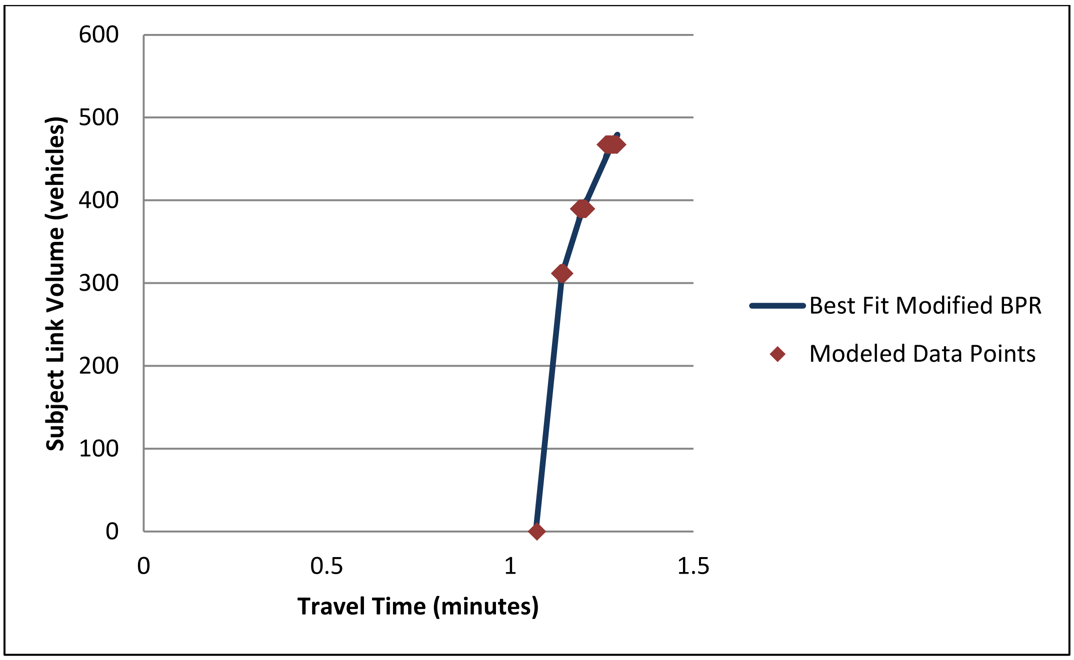


Figure 7: A Link Volume/Travel Time Relationship Not Affected by Cross and Opposing Traffic.

In contrast, the effects from variations in major cross traffic have great effect on the delay of the subject link (Figure 8).

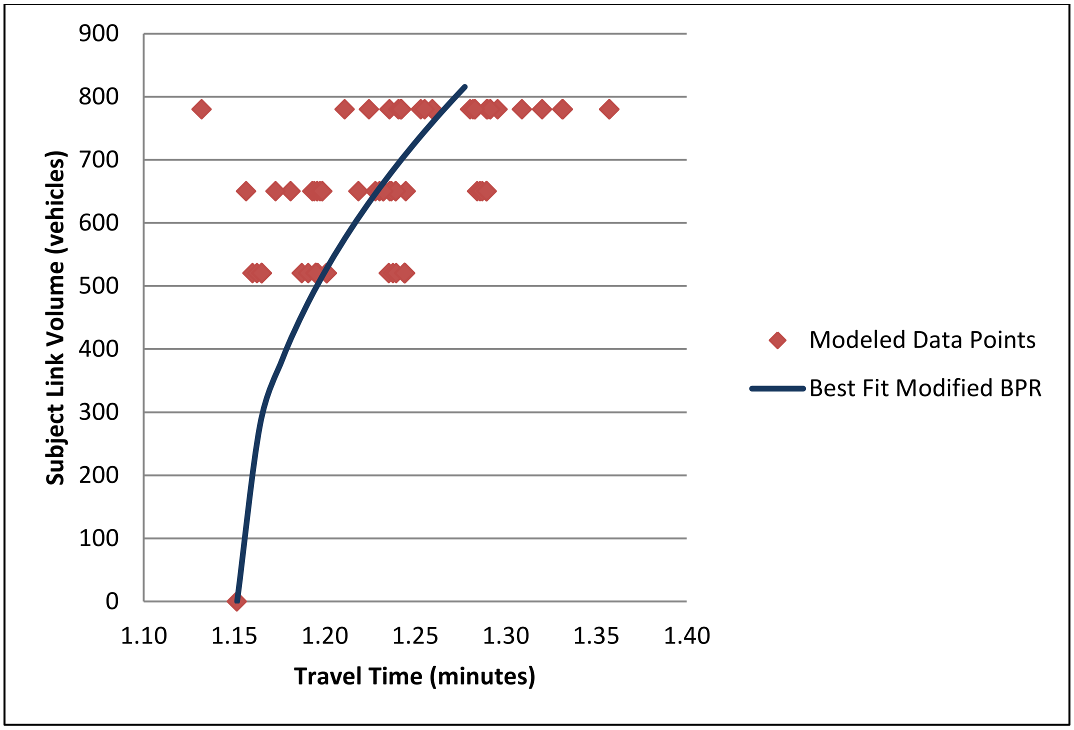


Figure 8: A Link Volume/Travel Time Relationship Affected by Cross and Opposing Traffic

As can be seen by Figure 8, it is more difficult deriving a volume-travel time function when the cross traffic of the subject link is a major road because perturbing the cross traffic will have great effect on the green split of the subject approach.

4.6: Procedure for Determining Directional Factors

The derived volume/travel time function along with observed travel time on a link-node segment is used to set directional factors for the original bi-directional traffic counts. Directional splits for the traffic counts are determined by calculating a volume based on the derived link function and observed input travel time for each link direction. The directional factor is calculated by dividing the calculated directional volume by the calculated two-way volume. That directional factor is then used in the next equilibrium traffic assignment for setting a target directional volume. If the derived function is determined to be unacceptable or the input travel time is less than the free travel time, the directional factor will revert to its original value. Given the potential sensitivity of the functions, max split values are used to keep values within realistic ranges depending on road type. The directional split refinement procedure occurs for all bi-directional traffic counts, with observed directional travel time, after each equilibrium traffic assignment. The complete directional split refinement procedure is outlined in Figure 9.

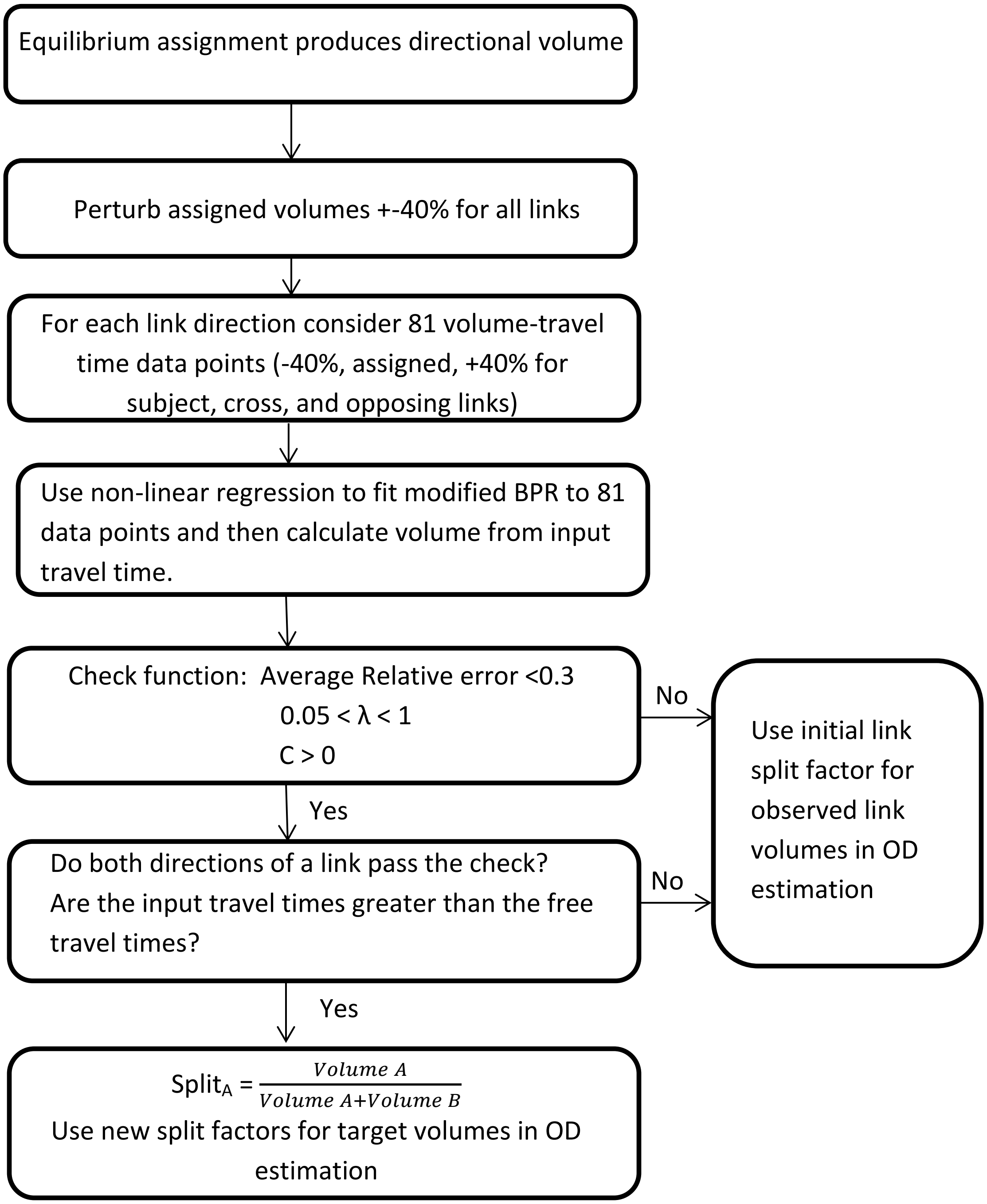


Figure 9: Procedure for Calculating Directional Factors for Observed Bi-directional Ground Counts

CHAPTER 5: DATA COLLECTION AND TESTS

5.1: Study Network

The study location for this thesis is the Mitchell Interchange (I-894 and I-94) in Milwaukee, WI. The network includes the I-94 and I-894 freeway system and arterials between Rawson Avenue on the south, Morgan Avenue on the north, Howell Avenue on the east, and 43th Street on the west. The network is used for comparing travel time sources, OD estimation, and testing actuated traffic signals. The Mitchell network shown in Figure 10Figure 10 was built using the General Network Editor. The black lines in the figure represent arterial and collector streets, the green lines represent the freeway system.

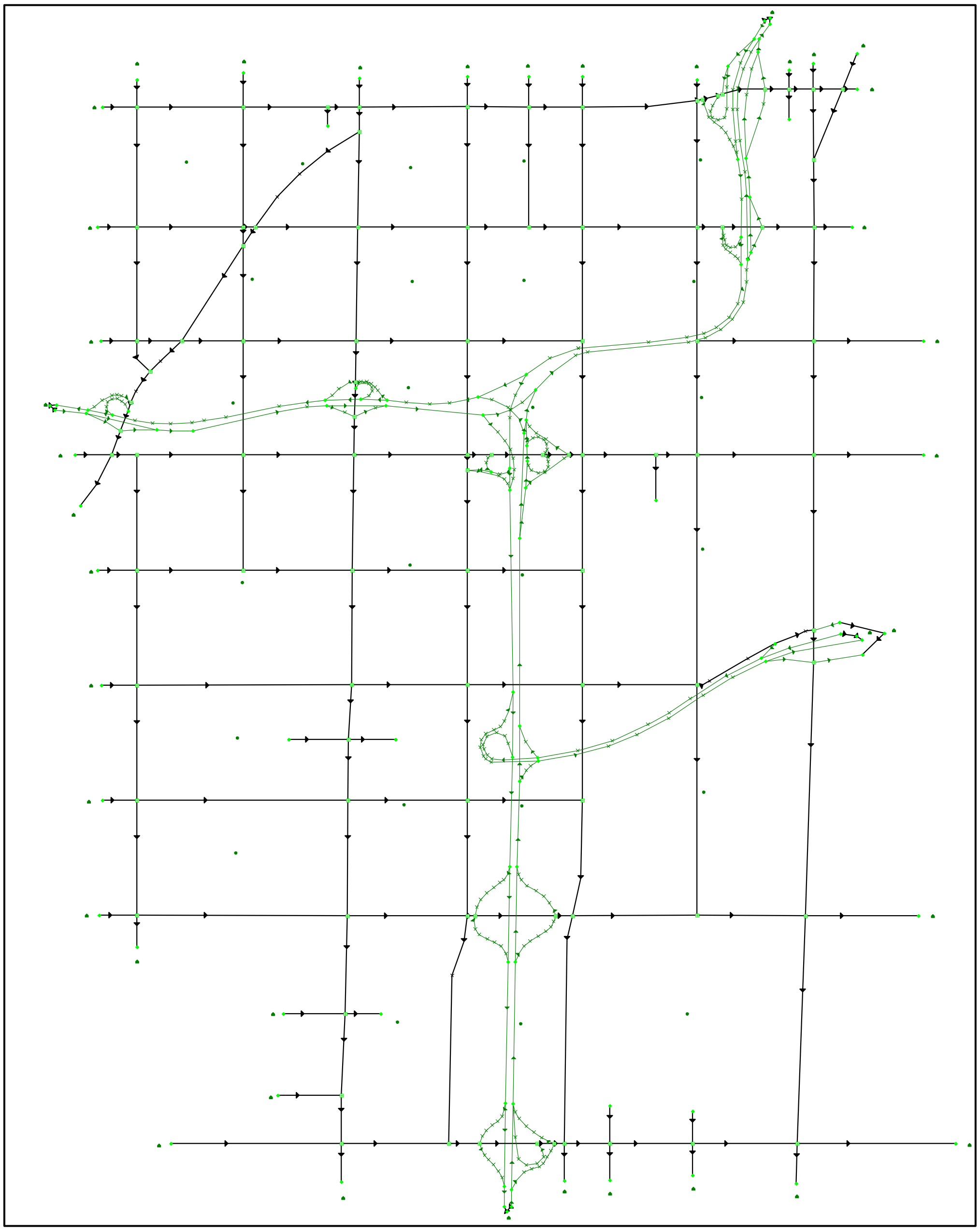


Figure 10: Mitchell Interchange Network

5.2: Actuated Signals in QRS II

The actuated signal module was tested using the Mitchell network. In real life, this network does not have actuated signals so a real life comparison of the output cannot be completed. However, the balance of turning volumes created an adequate environment for testing the sensitivity and performance of the actuation procedure.

For the actuation modeling tests, all intersection in the Mitchell network were set to have the following actuated timing variables:

- Total yellow = 12 seconds
- Total red = 4 seconds
- Thru/left passage time = 3 seconds
- Minimum though/left green = 2 seconds
- Maximum left = 20 seconds
- Maximum thru = 40 seconds
- Detector thru = 6, detector left = 10
- Iterations = 6
- Existing lane geometry

5.2.1: How to Code Actuation in QRS II

The intersection approach geometry is specified in the approach code of the link attributes. In the approach code there are multiple attributes describing the approach characteristics. The lane geometry in the approach code is under "Lane Geometry and

Sign A to B" and "Lane Geometry and Sign B to A." See the QRS II reference manual for more information on the approach codes (Horowitz, Reference Manual Quick Response System II, 2011).

QRS II determines if a left turn should be protected or permitted. Users cannot explicitly indicate that a left turn is shared with the through movement. Because of this all through and left turn minimum and maximum green attributes must be entered for a link. For most cases if the traffic signal is properly engineered, QRS II will correctly assign protected or permitted lefts based on the left and opposing through volume. Small minimum and maximum greens are recommended for default left turn parameters for nonprotected left turns. The maximum green time for the left or through movements of an approach are coded in the "Max g TR A to B", "Max g L A to B", "Max g TR A to B", and "Max g L A to B." The minimum green time is coded in the attributes "g TR Override A to B", "g L Override A to B", "g TR Override B to A", and "g L Override B to A." The green extension time is coded at an intersection level under the node (Intersection with Delay) attributes "Green Extension TR" and "Green Extension L." The total yellow time and all red time is also entered in this dialog box. The other attributes required as part of actuated timing plans is the detector length, and passage time for left and through movements, which are parameters defined at the network level within QRS II.

The QRS II dialog box for actuated signals allows the setting of default parameters for the actuation procedure based on the 2010 Highway Capacity Manual. Figure 11 shows the attributes within the QRS II actuation dialog box.

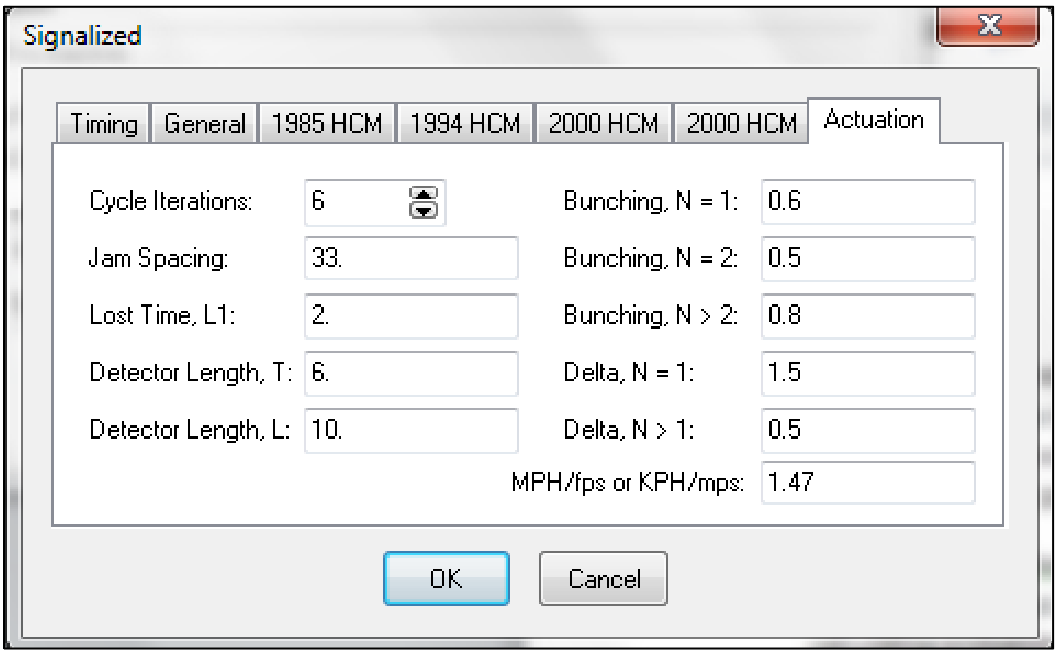


Figure 11: QRS II Actuation Default Parameters

The description of QRS II actuation parameters shown in Figure 11 are listed below:

Bunching Factors: Default recommended for HCM 2010 Equations 31-6 and 31-7

Delta: Headway of bunched vehicle stream in lane group. Default variables defined by number of lanes and recommended by HCM 2010 for use in equations 31-5, 31-8.

MPH/fps or KPH/mps: Conversion factor. This variable is default for US customary units. Should be changed to 0.28 if working with SI units.

Detector Length T and L : Detector length can be specified separately for through and left movements. The detector length is used in HCM Equation 31-11 to calculate Maximum Allowable Headway (MAH). As detector length increases so does the MAH, which increases the probability of additional phase extensions. Additional phase extension will increase the phase duration and average cycle length. Pulse detectors should be coded with a length of zero.

Lost Time, L1: Startup loss time from vehicles waiting at a signal. Loss time is used in HCM 2010 equations 31-29, and 31-36.

Jam Spacing: Average length of vehicles calculated by Equation 31-12. Default assumes 5% trucks, 8-feet cars, 45-feet heavy trucks.

Cycle Iterations: The actuation procedure requires iterations to converge on average phase duration and cycle length. The HCM has convergence criteria of 0.1 seconds difference between cycle lengths on proceeding iterations. QRS II accomplishes the same convergence process by assigning cycle iteration variable. From trials, it was shown that six iterations were sufficient to converge for most actuated signals.

5.2.2: Actuation Results in Mitchell Network

Using the Mitchell network with signals set to actuated using the variables described above, two intersections were examine closer by varying the input parameters. Both of the intersections have exclusive left turn lanes for all four approaches. The test for these two intersection included varying the min/max times, detector lengths, and passage time. The results from this analysis are summarized in Table 2. All model runs had equilibrium traffic assignment using 75 iterations.

The first intersection in the table (27th Street and Morgan Avenue) was determined by QRS II to be an 8-phase signal with protected lefts for each approach. The second intersection (27th Street and Cold Spring Rd/ Bolivar Ave) was determined by QRS II to be 6-phase signal with the major road (27th street) having protected left turns, while the minor street (Cold Spring/Bolivar) had permitted only left turns and shared the green time with the through movements.

		Scenario 1			Scenario 2			Scenario 3			Scenario 4		
		Detector Thru = 6 ft Detector Left = 10 ft Passage Left/thru = 3 s			Detector Thru = 6 ft Detector Left = 10 ft Passage Left/thru = 3 s			Detector Thru = 40 ft Detector Left = 40 ft Passage Left/thru = 3 s			Detector Thru = 6 ft Detector Left = 10 ft Passage Left/thru = 4 s		
Intersection	Movement	Volume	Min/Max	Green Time	Volume	Min/Max	Green Time	Volume	Min/Max	Green Time	Volume	Min/Max	Green Time
27th Street & Morgan Ave	WBR	189	2/40	14	189	10/50	16	189	2/40	17	189	10/50	19
	WBT	210			210			210			210		
	EBR	138	2/40	14	133	10/50	16	145	2/40	17	132	10/50	19
	EBT	163			163			163			163		
	WBL	400	2/20	11	393	10/30	13	445	2/20	11	393	10/30	13
	EBL	181	2/20	11	181	10/30	13	181	2/20	11	181	10/30	13
	NBR	483	2/40	36	480	10/50	43	485	2/40	38	480	10/50	47
	NBT	718			718			718			718		
	SBR	55	2/40	36	55	10/50	43	55	2/40	38	55	10/50	47
	SBT	1228			1228			1228			1228		
	NBL	442	2/20	19	432	10/30	26	445	2/20	19	431	10/30	28
	SBL	154	2/20	19	154	10/30	26	154	2/20	19	154	10/30	28
27th Street & Cold Spring Rd /Bolivar Ave	WBR	46	2/40	13	48	10/50	14	47	2/40	14	45	10/50	17
	WBT	176			178			176			178		
	EBR	55	2/40	13	48	10/50	14	49	2/40	14	46	10/50	17
	EBT	62			62			62			62		
	WBL	4	2/20	13	4	10/30	14	4	2/20	14	4	10/30	17
	EBL	39	2/20	13	35	10/30	14	38	2/20	14	28	10/30	17
	NBR	2	2/40	27	1	10/50	31	1	2/40	31	1	10/50	38
	NBT	882			865			875			871		
	SBR	27	2/40	27	27	10/50	31	27	2/40	31	27	10/50	38
	SBT	1065			1064			1064			1064		
	NBL	56	2/20	16	61	10/30	21	53	2/20	17	61	10/30	24
	SBL	227	2/20	16	219	10/30	21	230	2/20	17	218	10/30	24

Table 2: Actuation Sensitivity Test

The actuation modeling runs summarized in Table 2Table 2: Actuation Sensitivity Test indicates that approach green splits are sensitive to the variables passage time, max/min green, and detector length. This sensitivity from the parameters suggests the actuation procedure is working properly. It can also be observed that the traffic assignments for the turning movements see some shifts due to the variations in green splits.

All four scenarios in the table were modeled using 75 traffic assignment iterations. The four scenarios used six iterations for calculating the phase duration and cycle length. Scenario 1 was also tested using three iterations and the green splits converged within a couple tenths of a second to the results from the six iterations. This indicates that six cycle iterations are conservative and should be adequate for most actuated traffic signals.

The convergence of the equilibrium assignment with actuated signals was analyzed by first looking at the volume and travel time on approaches leading to the test actuated signal at 27th Street and Cold Spring. The convergence was analyzed by comparing the assigned volumes and travel time from the 74th and 75th equilibrium iterations. Minimal changes in travel time and volume for the actuated link approaches would indicate that the equilibrium assignment is converging. If there are major changes in volume or travel time than the equilibrium assignment is oscillating because the green times and cycle lengths from the actuated signals are changing by large increments. The approach volumes and travel time for the test intersection of 27th Street and Cold Spring using scenario 1 is shown in Table 3.

		74th Equilibrium Iteration		75th Equilibrium Iteration	
Intersection	Approach	Volume	Travel Time	Volume	Travel Time
27th Street & Cold Spring Road	EB	187	4.292	187	4.291
	WB	267	4.489	267	4.489
	NB	1275	1.989	1275	1.989
	SB	1451	3.725	1451	3.725

Table 3: Comparison of Volume and Travel Time between Equilibrium Iterations

The approach volume and travel time for 27th Street and Cold Spring Road in Table 3, indicate the Mitchell network with actuation converged at this intersection. At this intersection, the volume and travel times are nearly identical for the 74th and 75th equilibrium assignment iterations.

To further analyze the converge of the equilibrium assignment with actuated signals, the link volume Root Mean Square (RMS) change between consecutive assignment iterations was recorded. The RMS results for the actuated network are shown in Figure 12.

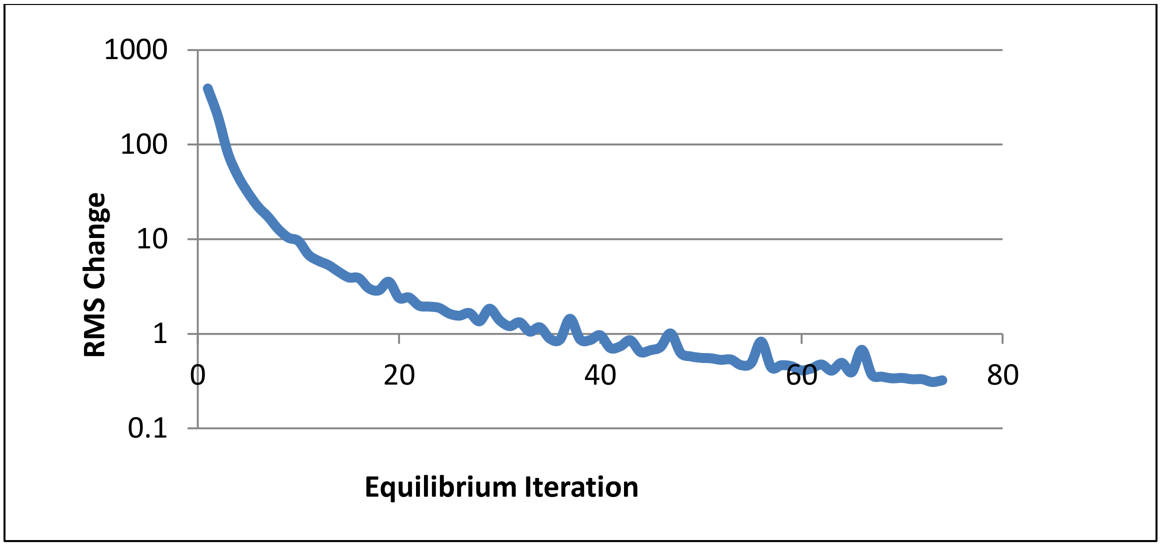


Figure 12: The link RMS change from previous equilibrium iterations using actuated signals

The RMS change shows that the Mitchell network continues to converge through the 75 equilibrium iterations.

5.3: Travel Time Collection

5.3.1: Overview

Travel time is an important data set for calibrating traffic models and has great potential for use in OD estimation procedures. The next section will discuss travel time from two sources: floating car runs and Navteq databases (a GPS mapping vendor). The travel time from these sources is compared to QRS II calculated travel time along arterials in a mid-sized network in Milwaukee, WI. The measured link direction (segment) travel time for the floating car runs and the QRS II calculations includes the delay at the upstream intersection (see Figure 13). Navteq does not include the intersection delay.

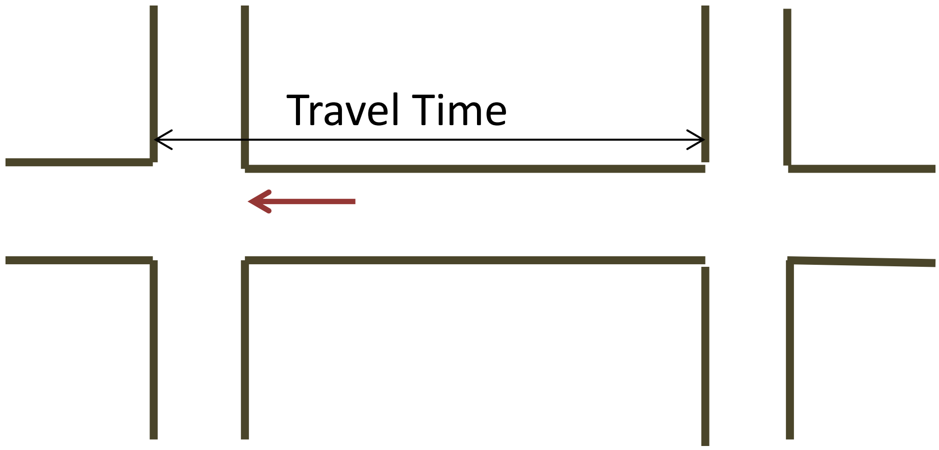


Figure 13: Segment Travel Time Including Upstream Intersection

5.3.2: Floating Car Runs

Travel time for the Mitchell network was collected using floating car runs during the PM peak (4:30-5:30 pm). The floating car runs took place in April 2012 over 14 weekdays with an average of four floating cars per day. The Floating car runs measured travel time for each link segment using GPS units. The GPS units in the floating cars took time stamps every second and the travel time between link segments was determined using longitude-latitude check points at the intersections. The time stamps measured the travel time between each link segment including the upstream intersection delay. An, additional travel time data set was created from floating car runs, which removed the time spend stopped at the intersections. This set of travel time with no intersection delay is used in the travel time comparison for an additional benchmark.

The floating car runs covered each link direction in the Mitchell network seven or eight times. These sample sizes sought to average deviations in intersection delay due to varying arrival times and provide statistical significance to the data.

5.3.3: Navteq Speed Data

Floating car data collection requires immense resources and would be difficult to complete for large projects. However, an alternative source of speed data is becoming available, which is sold by private sector vendors. The private vendor speed data, if proven accurate, could be implemented into large scale macroscopic models for a variety of calibration purposes. The private sector speed data is collected through a variety of means including: consumer GPS, commercial fleets, state installed sensors, cell phones, and Bluetooth systems. The private sector speed data that was analyzed for this thesis is from Navteq. The Navteq speed data is collected primarily with consumer GPS units. Other companies selling travel time data are Airsage, ATRI, INRIX, TomTom, and TrafficCast. There is an increased interest in using the private sector speed data by MPO's and state DOT's. A survey by the FHWA indicates the private sector speed data has been used primarily for traffic planning purposes such as congestion mitigation and calibrating/validating traffic models (FHWA, 2011).

The Navteq product includes the Navstreets network, which is a detailed road network that can be related to the speed database. Navteq sells two travel time database products: Analytics and Patterns. Both databases have 24 hour coverage with speed broken into 15 and 60 minute increments. The Analytics database is the most disaggregated database with speed data for annual days of the week in specific years related to Traffic Message Channels (TMC) positions, which are points or links in a network, which collect travel time data. The Patterns database is smoothed data that incorporates the last three years of travel time and is separated by days of the week.

The Navteq Analytics and Pattern speed database do not consider intersection delay. It is the responsibility of the end users to assign their own intersection delay algorithm to the link speed data. Another downside with the Navteq speed database is the maximum speed on a link is capped at the speed limit.

5.3.4: Working with Navteq Database

Navteq speed data is collected at points in the network called Traffic Message Channel (TMC) locations. At these TMC points the speeds are collected through the available sources (e.g. in-car GPS units). Links in the network are related to the nearest TMC. Therefore, the Navteq speed data is joined to the NavStreets map by using a TMC to link reference. The TMC referencing system is not an exact match to one link. In most cases there is one directional TMC that relates to multiple link segments. In rare cases there are multiple TMC's per link segment. In the Mitchell corridor there were no cases were multiple directional TMC's related to the same segment. See Figure 14 for an example of Link ID/TMC referencing spreadsheet provided by Navteq.

LINK_ID	TRAFFIC_CD	TMC
16877058	-107-12365	107N12365
16877058	+107+12366	107P12366
16877905	+107-09518	107N09518
16879004	-107-05812	107N05812
16879006	+107+05813	107P05813
16879007	+107+05813	107P05813
16879016	+107+05812	107P05812
16879018	-107N05811	107N05811
16879020	+107P05811	107P05811

Figure 14: Example of Link ID – TMC Referencing Spreadsheet

The TMC are joined to the network by first manipulating the TMC-link reference file so the link ID is in column A, the TMC 'N' direction (WB or SB) is in column B, and the TMC 'P' direction (EB or NB) is in column C. If there are multiple TMC's per link direction the table needs to be dealt with in a different manner, but for the Mitchell corridor and most areas in the United States this an adequate procedure for sorting the data. See Figure 15 for manipulated referencing spreadsheet.

LINK_ID	TMC NB_EB	TMC2 SB_WB
27675164	107N04865	0
27694455	107N15872	107P15873
27694456	107N15872	107P15873
27694525	0	107P14182
27694527	107N14181	0
27711540	0	107P14191
27711541	0	107P14191
27711542	0	107P14191
27711543	107N14190	0
27711544	107N14190	0
27711545	107N14190	0
27711546	0	107P14191
27711547	0	107P14191

Figure 15: Manipulated Link ID – TMC Referencing Spreadsheet

Once the data is in the order shown in Figure 15 it can be joined directly to the NavStreets network using ArcGIS by relating network link ID to the link ID in the TMC reference file. These TMC referencing can be saved to the Navteq Streets file with one column for the “WB/SB” TMC and the second for the “NB/EB” TMC. The Navteq Patterns and Analytics databases can be referenced to the TMC values in the network. See Figure 16, for example of TMC speed referencing.

TMC	H09_00	H10_00	H11_00	H12_00	H13_00	H14_00	H15_00	H16_00	H17_00	H18_00	H19_00	H20_00
106P10932	19	18	17	17	18	19	19	19	21	24	27	31
106P10933	18	18	18	19	19	19	18	16	17	21	27	33
106P10934	24	26	27	27	28	28	26	22	20	25	33	38
106P10935	36	34	31	29	27	28	30	33	36	38	39	40
106P10936	39	37	37	35	34	34	35	36	38	39	40	40
106P10938	31	28	25	23	21	22	23	27	30	33	34	35
106P10939	28	24	21	18	16	16	18	22	26	31	33	34
106P10940	31	28	25	22	20	20	21	24	28	32	34	35
106P10941	41	38	35	33	32	33	35	39	41	43	44	45
106P10942	41	40	37	35	35	34	36	38	40	43	44	45
106P10943	31	27	24	23	24	25	28	30	34	38	41	43

Figure 16: Example Analytics Speed Database with TMC Referencing

After the TMC and Pattern ID's are joined to the network it is important to search for referencing errors. The one error caught in the Mitchell network was "N" and "P" direction coded to both the northeast and southwest direction on Loomis Road.

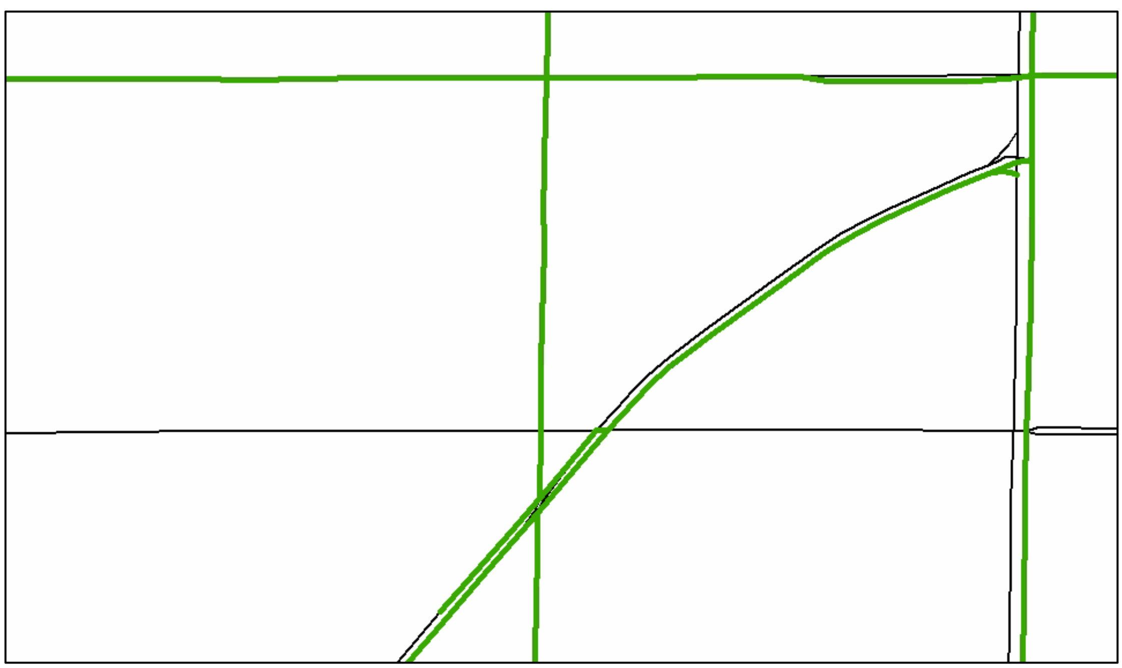


Figure 17: Navteq TMC Coding Error on Loomis Road

In Figure 17, the EB and SB direction should be highlighted in the thick green line. However, there are a few link segments in the southwest direction, which also are highlighted. Depending on the methodology for extracting the travel time data this has the potential to cause a double counting error. Luckily, this section of Loomis Road was the only place in the Mitchell network where this error occurs. This coding error most likely occurred because the road is not in a major cardinal direction.

To export the travel time data from the Navstreets network I geocoded the links to match the referencing in the GNE version of the Mitchell network. For instance, in the GNE network one link has the name HowardAve5, which is Howard Avenue between 27th and 20th Street. The same extent in the Navstreet network has 10 link segments. So, all these 10 segment were selected and given the Mitchell ID of "HowardAve5" (See Figure 18). This geocoding was done to synchronize the Navstreets network with the GNE network.

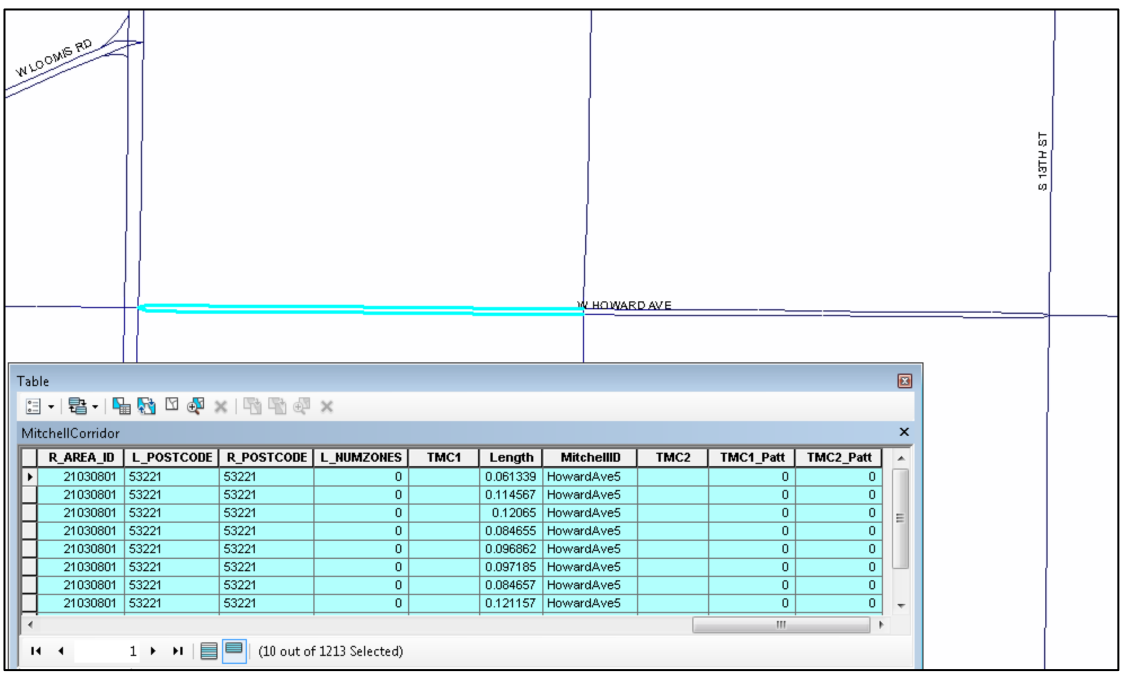


Figure 18: Example of Geocoding the Navstreets Network

In order to calculate travel time for each link the length of the link needs be calculated. The Navstreets database is in GCS_WGS_1984 coordinate system. To get the length dimension of each link the coordinate system is projected to the Southern

Wisconsin State Plane Coordinate system. Projecting Navstreet network for Mitchell cut out network allows for the link length to be calculated in feet. The length dimension is used to convert the speeds to link travel time. The speed and travel time data was post processed using Excel spreadsheets to aggregate the Navstreet links to replicate the links in the GNE network.

5.4: QRS II Travel Time Data

5.4.1: Overview

Modeled travel time in QRS II is calculated for both the node (control delay) and the link. The travel time is calculated using the HCM procedure for the node delay and the BPR curve for the link travel time. Both of these procedures were outlined in the literature review under “Node Delay” and “Link Delay.” The QRS II travel time for the Mitchell network is calculated from an equilibrium assignment of an OD matrix which was estimated using PM peak hour traffic counts.

5.4.2: Mitchell Network with Closures

The floating car runs were collected while the Mitchell Interchange was under construction. The construction caused traffic to shift from the freeways to the arterials, which created arterial travel times that were slower than normal conditions. This makes it more difficult to compare the floating car runs to the alternative travel time databases.

To relate the floating car runs to the QRS II travel time, the Mitchell Network was modified to replicate the construction activities during the month the floating car runs were performed (April 2012). Construction schedules for the I-43 corridor were collected in order to identify major construction activity for April 2012. The following Mitchell network adjustments were applied to replicate the construction on I-43:

1. Closed: I-94 East (SB) exit to Layton Ave
2. Closed: College Ave entrance ramp to NB I-94/43
3. Closed: Airport Spur entrance ramp access to NB I-94/43
4. Closed: I-43/94 SB access to Airport Spur exit
5. Closed: NB 27th St. entrance ramp to I-43/94 NB
6. Closed: Layton Ave ramp to I-43/94 NB
7. Closed: Howard Ave entrance ramp to I-43/94 SB
8. Closed: WB Airport Spur
9. 1 lane closure: EB Airport Spur
10. 1 lane closure: Mitchell N-W ramp
11. 1 lane closure: Mitchell W-N ramp
12. 1 lane closure: Mitchell W-S ramp
13. 1 lane closure: Mitchell S-W ramp

In addition to these closures the capacity of the freeway was reduced to 1800 pc/mi/ln to represent the work zone freeway capacity. Full closures were modeled by adding 1,000 minutes of extra time to the closed link. One-lane closures were modeled by removing one lane equivalent of capacity, which for the mainline segments equal an 1800 pc/mi/ln capacity reduction.

5.5: Comparison of Travel Time Data

Travel time data for the Mitchell network is compared along select segments for the floating car runs with and without control delay, QRS II travel time output with and without construction closures, Navteq Analytics, and Navteq Patterns databases. The travel time along ten arterials within the Mitchell network are shown in Table 4. The segments presented in the table were aggregated to smooth link specific variations. The average minutes per mile along the same ten segments are in Table 5. A graphical display comparing the travel time along 20th Street and Layton Avenue is shown in Figure 19.

			Travel Time (Min)											
			Floating Car		Floating Car w/o Stop Delay		QRS II Output		QRS II Closure		Navteq Analytics		Navteq Patterns	
Segment	Cross Streets	Length (Mile)	EB/SB	WB/NB	EB/SB	WB/NB	EB/SB	WB/NB	EB/SB	WB/NB	EB/SB	WB/NB	EB/SB	WB/NB
Layton Avenue	Loomis Rd. to Howell	3	9.8	10.4	7.8	8.4	7.1	8.0	7.5	8.3	5.8	6.0	6.6	6.8
13th Street	Morgan Ave to Rawson	4.5	10.7	10.5	9.4	9.4	9.8	9.6	10.2	9.9	8.9	8.8	9.4	10.0
20th Street	Morgan Ave to College Ave	3.5	9.2	9.1	8.6	8.3	8.2	7.9	8.6	8.7	7.9	8.3	7.8	7.9
27th Street	Morgan Ave to Rawson	4.4	12.7	10.8	10.8	9.6	11.8	10.8	12.0	11.7	8.7	8.6	8.6	10.0
35th Street	Morgan Ave to Layton Ave	1.5	3.9	4.2	3.7	3.7	4.1	4.2	4.4	4.2	3.6	3.6	4.0	3.8
College Avenue	43rd St. to Howell Ave.	3	8.5	7.5	7.2	6.5	6.1	6.2	6.2	6.3	6.2	6.1	6.3	7.0
Grange Avenue	43rd St. to Howell	3	9.6	8.2	8.1	7.3	5.9	6.1	6.0	6.5	7.3	7.0	7.8	7.6
Loomis Road	27th St to Layton	2.1	4	4.8	3.6	4.0	3.3	3.3	3.4	3.4	4.4	4.4	4.7	4.6
Morgan Avenue	35th St. to 6th St	2	6.2	6.3	5.3	5.5	4.9	5.2	4.9	5.3	5.2	4.8	5.0	4.9
Rawson Avenue	27th St. to Howell	2	5.8	4.7	4.8	4.2	4.3	4.2	4.4	4.3	3.6	3.7	4.1	4.5

Table 4: Average Travel Time: Comparison of Travel Time on select roadways in Mitchell Network

			Average Minutes per mile											
			Floating Car TT		Floating Car w/o Stop Delay		QRS II without Closure		QRS II with Closure		Navteq Analytics		Navteq Patterns	
Segment	Cross Streets	Length (Mile)	EB/SB	WB/NB	EB/SB	WB/NB	EB/SB	WB/NB	EB/SB	WB/NB	EB/SB	WB/NB	EB/SB	WB/NB
Layton Avenue	Loomis Rd. to Howell	3	3.3	3.5	2.6	2.8	2.4	2.7	2.5	2.8	1.9	2.0	2.2	2.3
13th Street	Morgan Ave to Rawson	4.5	2.4	2.3	2.1	2.1	2.2	2.1	2.3	2.2	2.0	2.0	2.1	2.2
20th Street	Morgan Ave to College Ave	3.5	2.6	2.6	2.4	2.4	2.3	2.3	2.5	2.5	2.3	2.4	2.2	2.2
27th Street	Morgan Ave to Rawson	4.4	2.9	2.5	2.5	2.2	2.7	2.5	2.7	2.7	2.0	2.0	1.9	2.3
35th Street	Morgan Ave to Layton Ave	1.5	2.6	2.8	2.4	2.5	2.7	2.8	3.0	2.8	2.4	2.4	2.7	2.5
College Avenue	43rd St. to Howell Ave.	3	2.8	2.5	2.4	2.2	2.0	2.1	2.1	2.1	2.1	2.0	2.1	2.3
Grange Avenue	43rd St. to Howell	3	3.2	2.7	2.7	2.4	2.0	2.0	2.0	2.2	2.4	2.3	2.6	2.5
Loomis Road	27th St to Layton	2.1	1.9	2.3	1.7	1.9	1.6	1.6	1.6	1.6	2.1	2.1	2.2	2.2
Morgan Avenue	35th St. to 6th St	2	3.1	3.2	2.6	2.7	2.5	2.6	2.4	2.6	2.6	2.4	2.5	2.5
Rawson Avenue	27th St. to Howell	2	2.9	2.4	2.4	2.1	2.1	2.1	2.2	2.1	1.8	1.8	2.1	2.2

Table 5: Average Minutes per Mile: Comparison of Travel Time Sources on Select Roadways in Mitchell Network

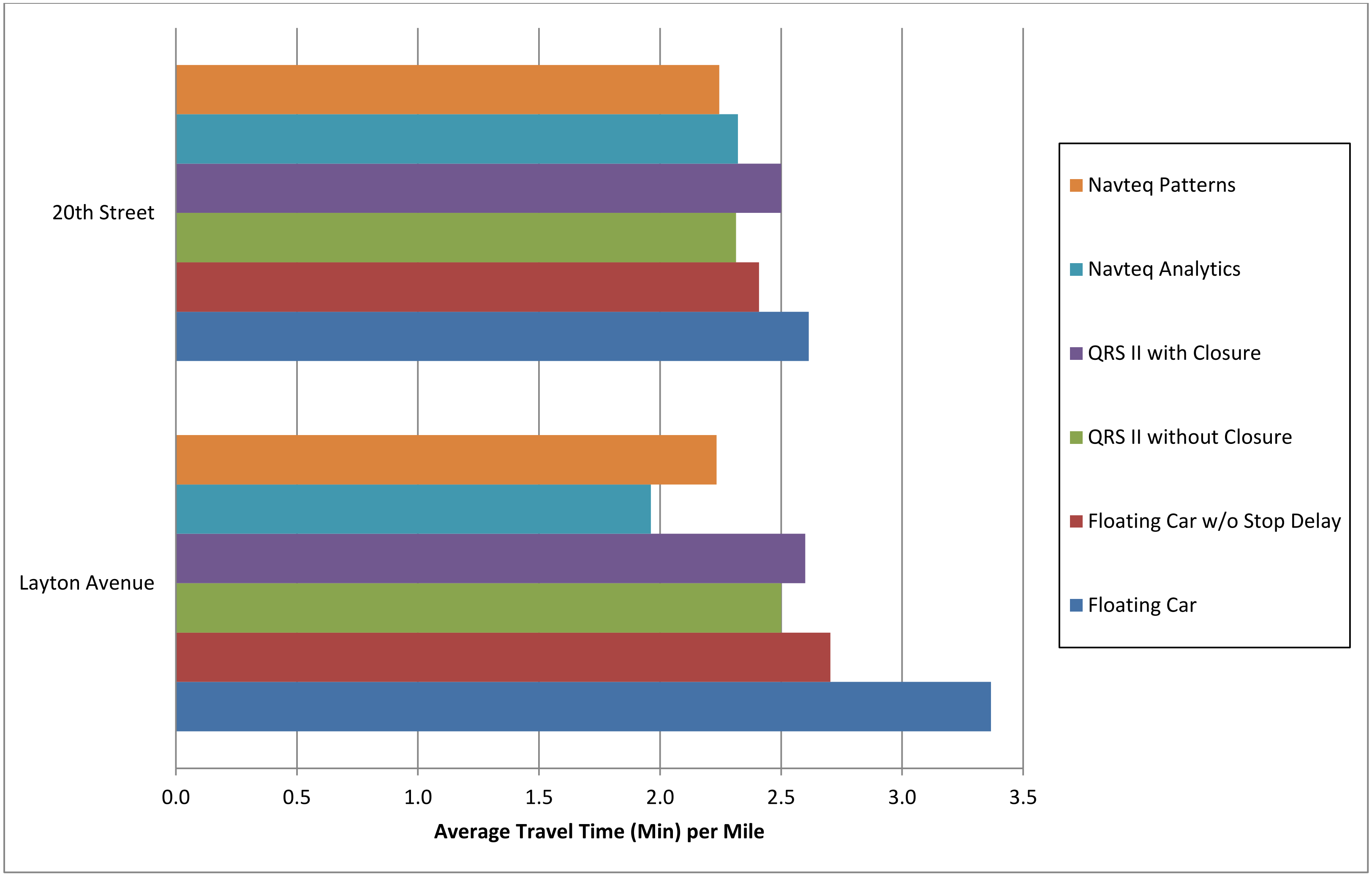


Figure 19: Comparison of Travel Time Data for 20th St and Layton Ave

The travel time comparison for the arterials in the Mitchell network indicates that the floating car runs are experiencing delay, which is not captured through any of the other data sources. Part of the travel time issues is the unknown impact due to the construction in the area. On average both Navteq databases have travel time lower than QRS II and the floating car runs. The lower travel time was expected on these segments because Navteq does not include delay accumulated at intersections. The Navteq Patterns database has higher travel times in comparison to the Analytics database, which better represented the floating car and QRS II travel time.

To compare travel time along individual link segments, Figure 20 and Figure 21 were created which show the individual link segment travel time as well as the accumulated travel time throughout the corridor. The travel time is compared in the northbound direction of 27th Street for the observed floating car and QRS II modeled with Mitchell Interchange construction.

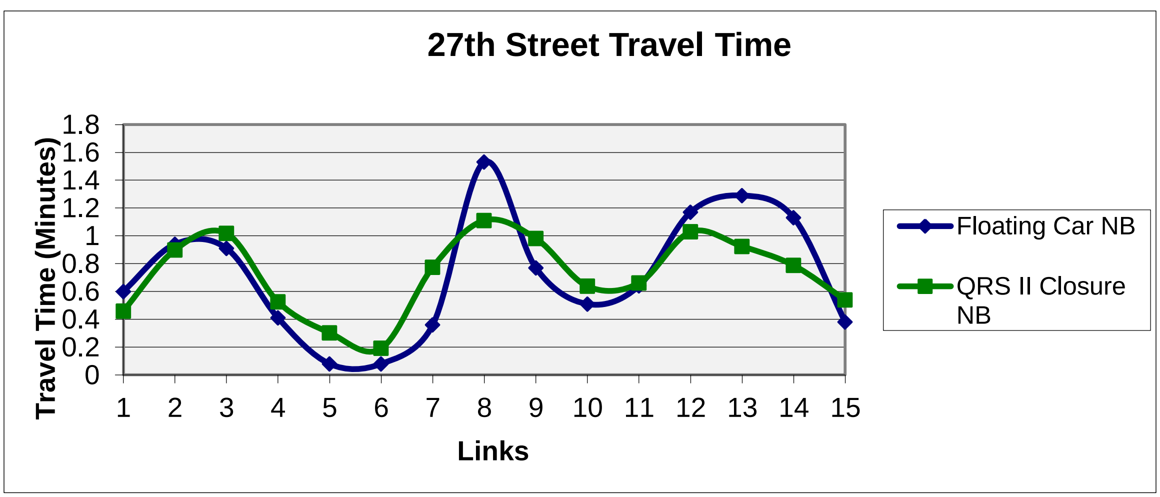


Figure 20: Travel Time Comparison showing Individual Link Travel Time between Floating Car and QRS II with Closure

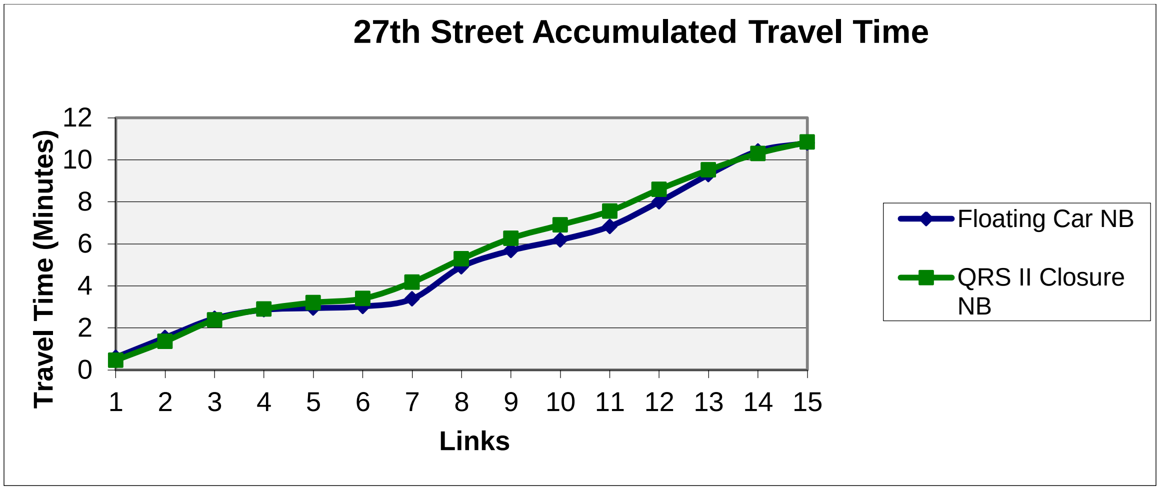


Figure 21: Travel Time Comparison Showing Accumulated Travel Time in a Corridor

The above figures show that although 27th Street travel time is comparable between floating car data and QRS II modeled when averaged over a corridor, the individual length segments are considerable deviations.

5.6: Navteq Speed Data for Freeway Mainlines

The Navteq speed data has been shown to have poor accuracy in signalized arterials because Navteq does not capture delay at intersections. To explore alternative uses of the Navteq data this case study looks at travel time from both Navteq Patterns and Analytics along freeway segments. The comparison looks at a stretch of I-43 from Hampton Avenue in Milwaukee, WI to STH 32 north of Grafton, WI. This section of freeway has 55 mph speed limit in the southern urban area and then transitions into a rural area with a speed limit of 65 mph.

One of the major disadvantages of the Navteq data is that the maximum speed is capped at the speed limit. This indicates the speed data cannot be used for establishing free flow speed of a facility, which limits the usefulness of the speed data to congested peak hours.

A comparison of the two Navteq databases are shown in Figure 22, Figure 23, and Figure 24 The comparisons show speeds in 15 minute bins between the hours of 5:00 AM and 10:00 PM. For the most part the two databases are very similar. Both databases show dips from the speed limit during the AM peak hour in the southbound direction towards Milwaukee south of the Mequon interchange, and in the northbound direction during the PM peak hour south of the Good Hope interchange. The major difference in the databases is the Pattern database has a dip in speed from Hampton Avenue to Good Hope Road on NB I-43 through most of the day, with the lowest speed occurring at 10:00 AM. Having a speed distribution that slows through the middle of the day and then having increased speeds during the AM and PM peak hours is unlikely. This example suggests the two Navteq databases are best used together to identify potential errors. Also, utilizing alternative travel time data in any calibration procedure may be important for validating the Navteq travel time data as well as for identifying locations were the average speed is above the speed limit.

5:00 AM to 10:00 PM

Facility Name	Point A	Point B	Length	5:00 AM				6:00 AM				7:00 AM				8:00 AM				9:00 AM				10:00 AM				11:00 AM				12:00 PM			
I-43 NB	South of Hampton	Hampton off ramp 1	2498	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 NB	Hampton off ramp 1	Hampton off ramp 2	1371	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 NB	Hampton off ramp 2	Port Washington Off ramp	2996	55	55	55	55	55	55	55	55	55	55	54	54	53	52	52	50	49	48	47	46	46	45	45	45	45	46	46	46	46	46	47	47
I-43 NB	Silver Spring Off ramp	Silver Spring On ramp	3417	55	55	55	55	55	55	55	55	55	55	54	54	53	52	52	50	49	48	47	46	46	45	45	45	45	46	46	46	46	46	47	47
I-43 NB	Silver Spring On ramp	Good Hope Off ramp	8602	55	55	55	55	55	55	55	54	53	53	52	51	50	48	46	44	42	40	39	38	38	38	39	39	40	40	41	41	42	42	42	43
I-43 NB	Good Hope Off ramp	Good Hope on ramp	2386	55	55	55	55	55	55	55	54	53	53	52	51	50	48	46	44	42	40	39	38	38	38	39	39	40	40	41	41	42	42	42	43
I-43 NB	Good Hope on ramp	1st Brown Deer off ramp	8462	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	54	54	53	53	53	53	53	52	52	52	52	52	51	51	51	51
I-43 NB	1st Brown Deer off ramp	1st Brown Deer on ramp	573	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	54	54	53	53	53	53	53	52	52	52	52	52	51	51	51	51
I-43 NB	1st Brown Deer on ramp	2nd Brown Deer off ramp	429	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	54	54	53	53	53	53	53	52	52	52	52	52	51	51	51	51
I-43 NB	2nd Brown Deer off ramp	2nd Brown Deer on ramp	671	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	54	54	53	53	53	53	53	52	52	52	52	52	51	51	51	51
I-43 NB	2nd Brown Deer on ramp	County Line off ramp	2201	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 NB	County Line off ramp	Mequon off ramp	12712	62	62	62	62	62	62	62	61	61	61	61	61	61	61	61	62	62	62	62	63	63	63	63	63	63	63	63	63	63	63	63	63
I-43 NB	Mequon off ramp	Mequon on ramp	2202	64	64	64	64	64	64	64	63	63	63	63	63	63	63	63	64	64	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 NB	Mequon on ramp	CTH off ramp	19315	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 NB	CTH off ramp	CTH C on ramp	2296	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 NB	CTH C on ramp	STH 60 off ramp	12324	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 NB	STH 60 off ramp	STH 60 on ramp	2522	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 NB	STH 60 on ramp	STH 32 off ramp	3938	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 NB	STH 32 off ramp	STH 32 on ramp	4154	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 NB	STH 32 on ramp	North of STH 32	13089	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	North of STH 32	STH 32 on ramp	13387	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	STH 32 on ramp	STH 32 off ramp	3734	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	STH 32 off ramp	STH 60 off ramp	4020	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	STH 60 off ramp	STH 60 on ramp	2503	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	STH 60 on ramp	CTH C off ramp	12378	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	CTH C off ramp	CTH C on ramp	2122	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	CTH C on ramp	Mequon off ramp	19421	65	65	65	64	64	63	63	63	62	62	62	63	63	63	64	64	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	Mequon off ramp	Mequon on ramp	2824	65	65	65	64	64	63	63	63	62	62	62	63	63	63	64	64	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	Mequon on ramp	County Line on ramp	10098	62	61	60	58	56	53	50	47	44	42	42	42	43	45	48	51	54	57	58	60	61	61	62	63	63	63	63	63	63	63	63	63
I-43 SB	County Line on ramp	1st Brown Deer off ramp	3452	54	53	52	51	49	47	45	43	40	39	39	38	39	40	42	44	46	48	49	50	52	52	53	53	54	55	55	55	55	55	55	55
I-43 SB	1st Brown Deer off ramp	1st Brown Deer on ramp	699	54	53	52	51	49	47	45	43	40	39	39	38	39	40	42	44	46	48	49	50	52	52	53	53	54	55	55	55	55	55	55	55
I-43 SB	1st Brown Deer on ramp	2nd Brown Deer off ramp	847	54	53	52	51	49	47	45	43	40	39	39	38	39	40	42	44	46	48	49	50	52	52	53	53	54	55	55	55	55	55	55	55
I-43 SB	2nd Brown Deer off ramp	2nd Brown Deer on ramp	570	54	53	52	51	49	47	45	43	40	39	39	38	39	40	42	44	46	48	49	50	52	52	53	53	54	55	55	55	55	55	55	55
I-43 SB	2nd Brown Deer on ramp	Good Hope off ramp	8472	55	54	54	53	52	51	49	48	47	45	45	45	45	46	47	48	49	50	52	52	53	53	53	53	54	54	54	54	54	54	54	54
I-43 SB	Good Hope off ramp	Good Hope on ramp	2213	55	54	54	53	52	51	49	48	47	45	45	45	45	46	47	48	49	50	52	52	53	53	53	53	54	54	54	54	54	54	54	54
I-43 SB	Good Hope on ramp	Silver Spring off ramp	8882	55	55	55	55	54	53	53	53	52	52	51	51	51	52	52	52	53	53	53	54	54	55	55	55	55	55	55	55	55	55	55	55
I-43 SB	Silver Spring off ramp	Silver Spring on ramp	2722	55	55	55	55	54	53	53	53	52	52	51	51	51	52	52	52	53	53	53	54	54	55	55	55	55	55	55	55	55	55	55	55
I-43 SB	Silver Spring on ramp	Hampton on ramp	4969	55	54	53	52	51	50	47	45	42	40	37	37	36	37	39	41	43	46	48	50	52	53	54	55	55	55	55	55	55	55	55	55
I-43 SB	Hampton on ramp	South of Hampton	2521	54	53	53	52	51	49	47	45	42	40	39	37	37	38	39	42	43	46	48	50	52	52	53	53	54	54	54	55	55	55	55	55

Facility Name	Point A	Point B	1:00 PM				2:00 PM				3:00 PM				4:00 PM				5:00 PM				6:00 PM				7:00 PM				8:00 PM				9:00 PM			
I-43 NB	South of Hampton	Hampton off ramp 1	55	55	55	55	55	55	55	55	55	55	54	54	53	53	53	53	53	53	53	54	54	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 NB	Hampton off ramp 1	Hampton off ramp 2	55	55	55	55	55	55	55	55	55	55	54	54	53	53	53	53	53	53	53	54	54	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 NB	Hampton off ramp 2	Port Washington Off ramp	47	48	48	49	49	50	50	51	52	52	52	52	52	51	50	50	50	50	50	50	51	52	52	53	53	54	55	55	55	55	55	55	55	55	55	55
I-43 NB	Silver Spring Off ramp	Silver Spring On ramp	47	48	48	49	49	50	50	51	52	52	52	52	52	51	50	50	50	50	50	50	51	52	52	53	53	54	55	55	55	55	55	55	55	55	55	55
I-43 NB	Silver Spring On ramp	Good Hope Off ramp	43	45	45	46	47	48	48	48	49	49	49	48	48	47	46	45	45	44	45	45	46	47	48	50	50	51	52	52	52	52	52	52	52	53	53	53
I-43 NB	Good Hope Off ramp	Good Hope on ramp	43	45	45	46	47	48	48	48	49	49	49	48	48	47	46	45	45	44	45	45	46	47	48	50	50	51	52	52	52	52	52	52	52	53	53	53
I-43 NB	Good Hope on ramp	1st Brown Deer off ramp	52	52	52	52	53	53	53	53	54	54	54	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	54	54	54	54	54	54	54	54	54	54
I-43 NB	1st Brown Deer off ramp	1st Brown Deer on ramp	52	52	52	52	53	53	53	53	54	54	54	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	54	54	54	54	54	54	54	54	54	54
I-43 NB	1st Brown Deer on ramp	2nd Brown Deer off ramp	52	52	52	52	53	53	53	53	54	54	54	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	54	54	54	54	54	54	54	54	54	54
I-43 NB	2nd Brown Deer off ramp	2nd Brown Deer on ramp	52	52	52	52	53	53	53	53	54	54	54	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	54	54	54	54	54	54	54	54	54	54
I-43 NB	2nd Brown Deer on ramp	County Line off ramp	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 NB	County Line off ramp	Mequon off ramp	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	62	62	62	62	61	61	61	61	62	62	62	62	62
I-43 NB	Mequon off ramp	Mequon on ramp	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	64	64	63	63	63	63	64	64	64	65	65
I-43 NB	Mequon on ramp	CTH off ramp	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 NB	CTH off ramp	CTH C on ramp	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 NB	CTH C on ramp	STH 60 off ramp	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	64	64	64	63	63	63	63	63	63	63	64	64
I-43 NB	STH 60 off ramp	STH 60 on ramp	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	64	64	64	63	63	63	63	63	63	63	64	64
I-43 NB	STH 60 on ramp	STH 32 off ramp	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 NB	STH 32 off ramp	STH 32 on ramp	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 NB	STH 32 on ramp	North of STH 32	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	North of STH 32	STH 32 on ramp	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	STH 32 on ramp	STH 32 off ramp	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	STH 32 off ramp	STH 60 off ramp	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	STH 60 off ramp	STH 60 on ramp	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	64	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	STH 60 on ramp	CTH C off ramp	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	CTH C off ramp	CTH C on ramp	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	CTH C on ramp	Mequon off ramp	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	Mequon off ramp	Mequon on ramp	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	Mequon on ramp	County Line on ramp	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	62	62	62	61	61	61	62	62	62	63	63	63	63
I-43 SB	County Line on ramp	1st Brown Deer off ramp	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	54	54	54	53	53	53	53	52	52	52	53	53	53	53	54	54	55	55
I-43 SB	1st Brown Deer off ramp	1st Brown Deer on ramp	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	54	54	54	53	53	53	53	52	52	52	53	53	53	53	54	54	55	55
I-43 SB	1st Brown Deer on ramp	2nd Brown Deer off ramp	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	54	54	54	53	53	53	53	52	52	52	53	53	53	53	54	54	55	55
I-43 SB	2nd Brown Deer off ramp	2nd Brown Deer on ramp	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	54	54	54	53	53	53	53	52	52	52	53	53	53	53	54	54	55	55
I-43 SB	2nd Brown Deer on ramp	Good Hope off ramp	54	54	54	54	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 SB	Good Hope off ramp	Good Hope on ramp	54	54	54	54	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 SB	Good Hope on ramp	Silver Spring off ramp	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 SB	Silver Spring off ramp	Silver Spring on ramp	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 SB	Silver Spring on ramp	Hampton on ramp	55	55	55	55	55	55	55	55	55	55	55	55	54	53	53	53	52	52	52	52	52	52	52	52	52	53	53	53	53	53	54	54	55	55	55	55
I-43 SB	Hampton on ramp	South of Hampton	55	55	54	54	54	54	54	53	53	53	52	52	51	50	50	49	48	48	48	48	49	50	50	51	51	52	52	52	53	53	53	53	54	54	55	55

Figure 22: Patterns Tuesday Speed Data for Segment on I-43 between Hampton Ave and STH 32

5:00 AM to 10:00 PM

Facility Name	Point A	Point B	Length	5:00 AM				6:00 AM				7:00 AM				8:00 AM				9:00 AM				10:00 AM				11:00 AM				12:00 PM			
I-43 NB	South of Hampton	Hampton off ramp 1	2498	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 NB	Hampton off ramp 1	Hampton off ramp 2	1371	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 NB	Hampton off ramp 2	Port Washington Off ramp	2996	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 NB	Silver Spring Off ramp	Silver Spring On ramp	3417	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 NB	Silver Spring On ramp	Good Hope Off ramp	8602	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 NB	Good Hope Off ramp	Good Hope on ramp	2386	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 NB	Good Hope on ramp	1st Brown Deer off ramp	8462	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 NB	1st Brown Deer off ramp	1st Brown Deer on ramp	573	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 NB	1st Brown Deer on ramp	2nd Brown Deer off ramp	429	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 NB	2nd Brown Deer off ramp	2nd Brown Deer on ramp	671	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 NB	2nd Brown Deer on ramp	County Line off ramp	2201	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 NB	County Line off ramp	Mequon off ramp	12712	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63
I-43 NB	Mequon off ramp	Mequon on ramp	2202	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 NB	Mequon on ramp	CTH off ramp	19315	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 NB	CTH off ramp	CTH C on ramp	2296	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 NB	CTH C on ramp	STH 60 off ramp	12324	65	65	65	65	65	64	64	64	64	64	64	64	64	64	64	64	64	64	64	64	64	65	65	65	65	65	65	65	65	65	65	65
I-43 NB	STH 60 off ramp	STH 60 on ramp	2522	65	65	65	65	65	64	64	64	64	64	64	64	64	64	64	64	64	64	64	64	64	65	65	65	65	65	65	65	65	65	65	65
I-43 NB	STH 60 on ramp	STH 32 off ramp	3938	64	64	64	64	64	64	64	64	64	64	63	63	63	63	63	63	63	63	64	64	64	64	64	64	65	65	65	65	65	65	65	65
I-43 NB	STH 32 off ramp	STH 32 on ramp	4154	64	64	64	64	64	64	64	64	64	64	63	63	63	63	63	63	63	63	64	64	64	64	64	64	65	65	65	65	65	65	65	65
I-43 NB	STH 32 on ramp	North of STH 32	13089	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	North of STH 32	STH 32 on ramp	13387	65	65	65	65	65	65	65	65	65	64	64	63	63	63	62	62	61	61	60	60	60	60	61	62	62	62	63	63	63	63	64	64
I-43 SB	STH 32 on ramp	STH 32 off ramp	3734	65	65	65	65	65	65	65	65	65	64	64	63	63	63	62	62	61	61	60	60	60	60	61	62	62	62	63	63	63	63	64	64
I-43 SB	STH 32 off ramp	STH 60 off ramp	4020	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	STH 60 off ramp	STH 60 on ramp	2503	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	STH 60 on ramp	CTH C off ramp	12378	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	CTH C off ramp	CTH C on ramp	2122	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	CTH C on ramp	Mequon off ramp	19421	65	65	65	65	65	65	65	64	64	64	63	63	63	63	63	63	63	62	62	62	62	62	62	62	63	63	63	63	63	63	63	63
I-43 SB	Mequon off ramp	Mequon on ramp	2824	65	65	65	65	65	65	65	64	64	64	63	63	63	63	63	63	63	62	62	62	62	62	62	62	63	63	63	63	63	63	63	63
I-43 SB	Mequon on ramp	County Line on ramp	10098	62	62	61	60	58	56	53	51	49	47	46	45	46	47	49	50	52	54	55	56	57	57	57	57	58	58	58	58	58	58	58	59
I-43 SB	County Line on ramp	1st Brown Deer off ramp	3452	55	54	54	53	52	50	49	47	45	44	43	43	43	44	45	47	48	50	52	52	53	53	54	54	55	55	55	55	55	55	55	55
I-43 SB	1st Brown Deer off ramp	1st Brown Deer on ramp	699	55	54	54	53	52	50	49	47	45	44	43	43	43	44	45	47	48	50	52	52	53	53	54	54	55	55	55	55	55	55	55	55
I-43 SB	1st Brown Deer on ramp	2nd Brown Deer off ramp	847	55	54	54	53	52	50	49	47	45	44	43	43	43	44	45	47	48	50	52	52	53	53	54	54	55	55	55	55	55	55	55	55
I-43 SB	2nd Brown Deer off ramp	2nd Brown Deer on ramp	570	55	54	54	53	52	50	49	47	45	44	43	43	43	44	45	47	48	50	52	52	53	53	54	54	55	55	55	55	55	55	55	55
I-43 SB	2nd Brown Deer on ramp	Good Hope off ramp	8472	55	55	55	54	53	53	52	51	50	49	48	48	48	48	50	50	52	52	53	53	54	54	55	55	55	55	55	55	55	55	55	55
I-43 SB	Good Hope off ramp	Good Hope on ramp	2213	55	55	55	54	53	53	52	51	50	49	48	48	48	48	50	50	52	52	53	53	54	54	55	55	55	55	55	55	55	55	55	55
I-43 SB	Good Hope on ramp	Silver Spring off ramp	8882	55	55	55	55	55	55	55	55	55	54	53	53	53	53	53	53	54	54	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 SB	Silver Spring off ramp	Silver Spring on ramp	2722	55	55	55	55	55	55	55	55	55	54	53	53	53	53	53	53	54	54	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 SB	Silver Spring on ramp	Hampton on ramp	4969	55	55	54	53	53	52	50	48	46	44	42	41	40	40	42	43	45	47	48	50	52	53	53	54	54	55	55	55	55	55	55	55
I-43 SB	Hampton on ramp	South of Hampton	2521	54	54	53	52	52	50	48	46	43	41	39	39	38	39	40	42	44	46	48	50	51	52	53	53	54	55	55	55	55	55	55	55

Facility Name	Point A	Point B	Length	1:00 PM				2:00 PM				3:00 PM				4:00 PM				5:00 PM				6:00 PM				7:00 PM				8:00 PM				9:00 PM			
I-43 NB	South of Hampton	Hampton off ramp 1	2498	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 NB	Hampton off ramp 1	Hampton off ramp 2	1371	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 NB	Hampton off ramp 2	Port Washington Off ramp	2996	55	55	55	55	55	55	55	55	54	54	53	53	52	51	50	50	49	49	50	50	51	52	53	53	54	54	55	55	55	55	55	55	55	55	55	55
I-43 NB	Silver Spring Off ramp	Silver Spring On ramp	3417	55	55	55	55	55	55	55	55	54	54	53	53	52	51	50	50	49	49	50	50	51	52	53	53	54	54	55	55	55	55	55	55	55	55	55	55
I-43 NB	Silver Spring On ramp	Good Hope Off ramp	8602	55	55	55	55	55	55	55	55	54	54	53	53	52	51	50	50	49	48	48	48	49	50	50	52	52	53	53	54	54	55	55	55	55	55	55	55
I-43 NB	Good Hope Off ramp	Good Hope on ramp	2386	55	55	55	55	55	55	55	55	54	54	53	53	52	51	50	50	49	48	48	48	49	50	50	52	52	53	53	54	54	55	55	55	55	55	55	55
I-43 NB	Good Hope on ramp	1st Brown Deer off ramp	8462	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 NB	1st Brown Deer off ramp	1st Brown Deer on ramp	573	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 NB	1st Brown Deer on ramp	2nd Brown Deer off ramp	429	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 NB	2nd Brown Deer off ramp	2nd Brown Deer on ramp	671	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 NB	2nd Brown Deer on ramp	County Line off ramp	2201	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	54	54	54	54	54	54	54	54
I-43 NB	County Line off ramp	Mequon off ramp	12712	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	63	62	61	61	61	61	61	60	60	60	60	60	60	60	60	61	61	61
I-43 NB	Mequon off ramp	Mequon on ramp	2202	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	64	63	63	63	63	63	62	62	62	62	62	62	62	62	63	63	63
I-43 NB	Mequon on ramp	CTH off ramp	19315	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	64	64	64	64	63	63	63	63	63	63	63	64	64
I-43 NB	CTH off ramp	CTH C on ramp	2296	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	64	64	64	64	63	63	63	63	63	63	63	64	64
I-43 NB	CTH C on ramp	STH 60 off ramp	12324	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	64	64	64	64	64	63	63	63	63	63	63	64	64	64
I-43 NB	STH 60 off ramp	STH 60 on ramp	2522	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	64	64	64	64	64	63	63	63	63	63	63	64	64	64
I-43 NB	STH 60 on ramp	STH 32 off ramp	3938	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	64	64	63	63	63	63	63	63	63	63	63	63	63	63	64
I-43 NB	STH 32 off ramp	STH 32 on ramp	4154	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	64	64	63	63	63	63	63	63	63	63	63	63	63	63	64
I-43 NB	STH 32 on ramp	North of STH 32	13089	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	64	64	64	64	64	63	63	63	63	64	64	64	64	65
I-43 SB	North of STH 32	STH 32 on ramp	13387	64	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	64	64	64	64	64	64	64	64	64	64	65	65	65	65	65	65	65	65	65
I-43 SB	STH 32 on ramp	STH 32 off ramp	3734	64	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	64	64	64	64	64	64	64	64	64	64	65	65	65	65	65	65	65	65	65
I-43 SB	STH 32 off ramp	STH 60 off ramp	4020	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	64	64	64	64	64	64	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	STH 60 off ramp	STH 60 on ramp	2503	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	64	64	64	64	64	64	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	STH 60 on ramp	CTH C off ramp	12378	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	64	64	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	CTH C off ramp	CTH C on ramp	2122	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	64	64	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	CTH C on ramp	Mequon off ramp	19421	63	63	64	64	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	Mequon off ramp	Mequon on ramp	2824	63	63	64	64	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65	65
I-43 SB	Mequon on ramp	County Line on ramp	10098	60	60	60	61	61	62	62	62	63	63	63	63	63	63	63	63	63	63	63	62	62	62	62	62	62	61	61	61	62	62	62	62	62	63	63	63
I-43 SB	County Line on ramp	1st Brown Deer off ramp	3452	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 SB	1st Brown Deer off ramp	1st Brown Deer on ramp	699	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 SB	1st Brown Deer on ramp	2nd Brown Deer off ramp	847	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 SB	2nd Brown Deer off ramp	2nd Brown Deer on ramp	570	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 SB	2nd Brown Deer on ramp	Good Hope off ramp	8472	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 SB	Good Hope off ramp	Good Hope on ramp	2213	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 SB	Good Hope on ramp	Silver Spring off ramp	8882	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 SB	Silver Spring off ramp	Silver Spring on ramp	2722	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 SB	Silver Spring on ramp	Hampton on ramp	4969	55	55	55	55	55	55	55	55	55	55	55	55	54	54	54	54	54	54	54	54	54	54	55	55	55	55	55	55	55	55	55	55	55	55	55	55
I-43 SB	Hampton on ramp	South of Hampton	2521	55	55	55	55	55	55	55	54	54	53	53	53	53	52	52	52	52	52	52	52	52	52	52	52	52	52	52	52	53	53	53	53	54	54	55	55

Figure 23: Navteq Analytics Tuesday Speed Data for I-43 between Hampton Ave and STH 32

5:00 AM to 10:00 PM

Facility Name	Point A	Point B	Length (ft)	5:00 AM				6:00 AM				7:00 AM				8:00 AM				9:00 AM				10:00 AM				11:00 AM				12:00 PM			
I-43 NB	South of Hampton	Hampton off ramp 1	2498	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
I-43 NB	Hampton off ramp 1	Hampton off ramp 2	1371	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
I-43 NB	Hampton off ramp 2	Port Washington Off ramp	2996	0	0	0	0	0	0	0	0	0	0	-1	-1	-2	-3	-3	-5	-6	-7	-8	-9	-9	-10	-10	-10	-10	-9	-9	-9	-9	-9	-8	-8
I-43 NB	Silver Spring Off ramp	Silver Spring On ramp	3417	0	0	0	0	0	0	0	0	0	0	-1	-1	-2	-3	-3	-5	-6	-7	-8	-9	-9	-10	-10	-10	-10	-9	-9	-9	-9	-9	-8	-8
I-43 NB	Silver Spring On ramp	Good Hope Off ramp	8602	0	0	0	0	0	0	0	-1	-2	-2	-3	-4	-5	-7	-9	-11	-13	-15	-16	-17	-17	-17	-16	-16	-15	-15	-14	-14	-13	-13	-13	-12
I-43 NB	Good Hope Off ramp	Good Hope on ramp	2386	0	0	0	0	0	0	0	-1	-2	-2	-3	-4	-5	-7	-9	-11	-13	-15	-16	-17	-17	-17	-16	-16	-15	-15	-14	-14	-13	-13	-13	-12
I-43 NB	Good Hope on ramp	1st Brown Deer off ramp	8462	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-1	-1	-2	-2	-2	-2	-2	-3	-3	-3	-3	-3	-4	-4	-4	-4
I-43 NB	1st Brown Deer off ramp	1st Brown Deer on ramp	573	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-1	-1	-2	-2	-2	-2	-2	-3	-3	-3	-3	-3	-4	-4	-4	-4
I-43 NB	1st Brown Deer on ramp	2nd Brown Deer off ramp	429	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-1	-1	-2	-2	-2	-2	-2	-3	-3	-3	-3	-3	-4	-4	-4	-4
I-43 NB	2nd Brown Deer off ramp	2nd Brown Deer on ramp	671	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-1	-1	-2	-2	-2	-2	-2	-3	-3	-3	-3	-3	-4	-4	-4	-4
I-43 NB	2nd Brown Deer on ramp	County Line off ramp	2201	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
I-43 NB	County Line off ramp	Mequon off ramp	12712	-1	-1	-1	-1	-1	-1	-1	-2	-2	-2	-2	-2	-2	-2	-2	-1	-1	-1	-1	0	0	0	0	0	0	0	0	0	0	0	0	0
I-43 NB	Mequon off ramp	Mequon on ramp	2202	-1	-1	-1	-1	-1	-1	-1	-2	-2	-2	-2	-2	-2	-2	-2	-1	-1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
I-43 NB	Mequon on ramp	CTH off ramp	19315	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
I-43 NB	CTH off ramp	CTH C on ramp	2296	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
I-43 NB	CTH C on ramp	STH 60 off ramp	12324	0	0	0	0	0	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	0	0	0	0	0	0	0	0	0	0	0
I-43 NB	STH 60 off ramp	STH 60 on ramp	2522	0	0	0	0	0	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	0	0	0	0	0	0	0	0	0	0	0
I-43 NB	STH 60 on ramp	STH 32 off ramp	3938	1	1	1	1	1	1	1	1	1	1	2	2	2	2	2	2	2	2	1	1	1	1	1	1	0	0	0	0	0	0	0	0
I-43 NB	STH 32 off ramp	STH 32 on ramp	4154	1	1	1	1	1	1	1	1	1	1	2	2	2	2	2	2	2	2	1	1	1	1	1	1	0	0	0	0	0	0	0	0
I-43 NB	STH 32 on ramp	North of STH 32	13089	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
I-43 SB	North of STH 32	STH 32 on ramp	13387	0	0	0	0	0	0	0	0	0	1	1	2	2	2	3	3	4	4	5	5	5	5	4	3	3	3	2	2	2	2	1	1
I-43 SB	STH 32 on ramp	STH 32 off ramp	3734	0	0	0	0	0	0	0	0	0	1	1	2	2	2	3	3	4	4	5	5	5	5	4	3	3	3	2	2	2	2	1	1
I-43 SB	STH 32 off ramp	STH 60 off ramp	4020	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
I-43 SB	STH 60 off ramp	STH 60 on ramp	2503	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
I-43 SB	STH 60 on ramp	CTH C off ramp	12378	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
I-43 SB	CTH C off ramp	CTH C on ramp	2122	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
I-43 SB	CTH C on ramp	Mequon off ramp	19421	0	0	0	-1	-1	-2	-2	-1	-2	-2	-1	0	0	0	1	1	2	3	3	3	3	3	3	3	2	2	2	2	2	2	2	2
I-43 SB	Mequon off ramp	Mequon on ramp	2824	0	0	0	-1	-1	-2	-2	-1	-2	-2	-1	0	0	0	1	1	2	3	3	3	3	3	3	3	2	2	2	2	2	2	2	2
I-43 SB	Mequon on ramp	County Line on ramp	10098	0	-1	-1	-2	-2	-3	-3	-4	-5	-5	-4	-3	-3	-2	-1	1	2	3	3	4	4	4	5	6	5	5	5	5	5	5	5	4
I-43 SB	County Line on ramp	1st Brown Deer off ramp	3452	-1	-1	-2	-2	-3	-3	-4	-4	-5	-5	-4	-5	-4	-4	-3	-3	-2	-2	-3	-2	-1	-1	-1	-1	-1	0	0	0	0	0	0	0
I-43 SB	1st Brown Deer off ramp	1st Brown Deer on ramp	699	-1	-1	-2	-2	-3	-3	-4	-4	-5	-5	-4	-5	-4	-4	-3	-3	-2	-2	-3	-2	-1	-1	-1	-1	-1	0	0	0	0	0	0	0
I-43 SB	1st Brown Deer on ramp	2nd Brown Deer off ramp	847	-1	-1	-2	-2	-3	-3	-4	-4	-5	-5	-4	-5	-4	-4	-3	-3	-2	-2	-3	-2	-1	-1	-1	-1	-1	0	0	0	0	0	0	0
I-43 SB	2nd Brown Deer off ramp	2nd Brown Deer on ramp	570	-1	-1	-2	-2	-3	-3	-4	-4	-5	-5	-4	-5	-4	-4	-3	-3	-2	-2	-3	-2	-1	-1	-1	-1	-1	0	0	0	0	0	0	0
I-43 SB	2nd Brown Deer on ramp	Good Hope off ramp	8472	0	-1	-1	-1	-1	-2	-3	-3	-3	-4	-3	-3	-3	-2	-3	-2	-3	-2	-1	-1	-1	-1	-2	-2	-1	-1	-1	-1	-1	-1	-1	-1
I-43 SB	Good Hope off ramp	Good Hope on ramp	2213	0	-1	-1	-1	-1	-2	-3	-3	-3	-4	-3	-3	-3	-2	-3	-2	-3	-2	-1	-1	-1	-1	-2	-2	-1	-1	-1	-1	-1	-1	-1	-1
I-43 SB	Good Hope on ramp	Silver Spring off ramp	8882	0	0	0	0	-1	-2	-2	-2	-3	-2	-2	-2	-2	-1	-1	-1	-1	-1	-2	-1	-1	0	0	0	0	0	0	0	0	0	0	0
I-43 SB	Silver Spring off ramp	Silver Spring on ramp	2722	0	0	0	0	-1	-2	-2	-2	-3	-2	-2	-2	-2	-1	-1	-1	-1	-1	-2	-1	-1	0	0	0	0	0	0	0	0	0	0	0
I-43 SB	Silver Spring on ramp	Hampton on ramp	4969	0	-1	-1	-1	-2	-2	-3	-3	-4	-4	-5	-4	-4	-3	-3	-2	-2	-1	0	0	0	0	1	1	1	0	0	0	0	0	0	0
I-43 SB	Hampton on ramp	South of Hampton	2521	0	-1	0	0	-1	-1	-1	-1	-1	-1	0	-2	-1	-1	-1	0	-1	0	0	0	1	0	0	0	0	-1	-1	0	0	0	0	0

Facility Name	Point A	Point B	1:00 PM				2:00 PM				3:00 PM				4:00 PM				5:00 PM				6:00 PM				7:00 PM				8:00 PM				9:00 PM			
I-43 NB	South of Hampton	Hampton off ramp 1	0	0	0	0	0	0	0	0	0	0	-1	-1	-2	-2	-2	-2	-2	-2	-2	-1	-1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
I-43 NB	Hampton off ramp 1	Hampton off ramp 2	0	0	0	0	0	0	0	0	0	0	-1	-1	-2	-2	-2	-2	-2	-2	-2	-1	-1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
I-43 NB	Hampton off ramp 2	Port Washington Off ramp	-8	-7	-7	-6	-6	-5	-5	-4	-2	-2	-1	-1	0	0	0	0	1	1	0	0	0	0	-1	0	-1	0	0	0	0	0	0	0	0	0	0	0
I-43 NB	Silver Spring Off ramp	Silver Spring On ramp	-8	-7	-7	-6	-6	-5	-5	-4	-2	-2	-1	-1	0	0	0	0	1	1	0	0	0	0	-1	0	-1	0	0	0	0	0	0	0	0	0	0	0
I-43 NB	Silver Spring On ramp	Good Hope Off ramp	-12	-10	-10	-9	-8	-7	-7	-7	-5	-5	-4	-5	-4	-4	-4	-5	-4	-4	-3	-3	-3	-3	-2	-2	-2	-2	-1	-2	-2	-3	-3	-3	-3	-2	-2	-2
I-43 NB	Good Hope Off ramp	Good Hope on ramp	-12	-10	-10	-9	-8	-7	-7	-7	-5	-5	-4	-5	-4	-4	-4	-5	-4	-4	-3	-3	-3	-3	-2	-2	-2	-2	-1	-2	-2	-3	-3	-3	-3	-2	-2	-2
I-43 NB	Good Hope on ramp	1st Brown Deer off ramp	-3	-3	-3	-3	-2	-2	-2	-2	-1	-1	-1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-1	-1	-1	-1	-1	-1	-1	-1	-1	-1
I-43 NB	1st Brown Deer off ramp	1st Brown Deer on ramp	-3	-3	-3	-3	-2	-2	-2	-2	-1	-1	-1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-1	-1	-1	-1	-1	-1	-1	-1	-1	-1
I-43 NB	1st Brown Deer on ramp	2nd Brown Deer off ramp	-3	-3	-3	-3	-2	-2	-2	-2	-1	-1	-1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-1	-1	-1	-1	-1	-1	-1	-1	-1	-1
I-43 NB	2nd Brown Deer off ramp	2nd Brown Deer on ramp	-3	-3	-3	-3	-2	-2	-2	-2	-1	-1	-1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-1	-1	-1	-1	-1	-1	-1	-1	-1	-1
I-43 NB	2nd Brown Deer on ramp	County Line off ramp	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	1	1	1	1	1	1
I-43 NB	County Line off ramp	Mequon off ramp	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	2	2	1	1	2	2	1	1	1	1	2	2	1	1	1
I-43 NB	Mequon off ramp	Mequon on ramp	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	2	2	2	2	2	2	1	1	1	1	2	2	1	2	2
I-43 NB	Mequon on ramp	CTH off ramp	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	1	1	2	2	2	2	2	2	2	1	1
I-43 NB	CTH off ramp	CTH C on ramp	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	1	1	2	2	2	2	2	2	2	1	1
I-43 NB	CTH C on ramp	STH 60 off ramp	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	0	0	0	0	0	0	0	0	0	-1	0	0
I-43 NB	STH 60 off ramp	STH 60 on ramp	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	0	0	0	0	0	0	0	0	0	-1	0	0
I-43 NB	STH 60 on ramp	STH 32 off ramp	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	2	2	2	2	2	2	2	2	2	2	2	2	1
I-43 NB	STH 32 off ramp	STH 32 on ramp	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	2	2	2	2	2	2	2	2	2	2	2	2	1
I-43 NB	STH 32 on ramp	North of STH 32	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	1	1	1	2	2	2	2	1	1	1	1	0
I-43 SB	North of STH 32	STH 32 on ramp	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	1	1	1	1	1	1	1	1	0	0	0	0	0	0	0	0	0
I-43 SB	STH 32 on ramp	STH 32 off ramp	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	1	1	1	1	1	1	1	1	0	0	0	0	0	0	0	0	0
I-43 SB	STH 32 off ramp	STH 60 off ramp	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	1	1	1	1	0	0	0	0	0	0	0	0	0	0	0	0
I-43 SB	STH 60 off ramp	STH 60 on ramp	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	1	1	1	1	0	0	0	0	0	0	0	0	0	0	0	0
I-43 SB	STH 60 on ramp	CTH C off ramp	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	0	0	0	0	0	0	0	0	0	0	0	0
I-43 SB	CTH C off ramp	CTH C on ramp	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	0	0	0	0	0	0	0	0	0	0	0	0
I-43 SB	CTH C on ramp	Mequon off ramp	2	2	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
I-43 SB	Mequon off ramp	Mequon on ramp	2	2	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
I-43 SB	Mequon on ramp	County Line on ramp	3	3	3	2	2	1	1	1	0	0	0	0	0	0	0	0	0	0	0	1	1	1	1	0	0	1	0	0	-1	0	0	0	1	0	0	0
I-43 SB	County Line on ramp	1st Brown Deer off ramp	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-1	-1	-1	-2	-2	-2	-2	-3	-3	-3	-2	-2	-2	-2	-1	-1	0	0
I-43 SB	1st Brown Deer off ramp	1st Brown Deer on ramp	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-1	-1	-1	-2	-2	-2	-2	-3	-3	-3	-2	-2	-2	-2	-1	-1	0	0
I-43 SB	1st Brown Deer on ramp	2nd Brown Deer off ramp	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-1	-1	-1	-2	-2	-2	-2	-3	-3	-3	-2	-2	-2	-2	-1	-1	0	0
I-43 SB	2nd Brown Deer off ramp	2nd Brown Deer on ramp	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-1	-1	-1	-2	-2	-2	-2	-3	-3	-3	-2	-2	-2	-2	-1	-1	0	0
I-43 SB	2nd Brown Deer on ramp	Good Hope off ramp	-1	-1	-1	-1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
I-43 SB	Good Hope off ramp	Good Hope on ramp	-1	-1	-1	-1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
I-43 SB	Good Hope on ramp	Silver Spring off ramp	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
I-43 SB	Silver Spring off ramp	Silver Spring on ramp	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
I-43 SB	Silver Spring on ramp	Hampton on ramp	0	0	0	0	0	0	0	0	0	0	0	0	0	-1	-1	-1	-2	-2	-2	-2	-2	-2	-3	-3	-3	-2	-2	-2	-2	-2	-1	-1	0	0	0	0
I-43 SB	Hampton on ramp	South of Hampton	0	0	-1	-1	-1	-1	-1	-1	-1	0	-1	-1	-2	-2	-2	-3	-4	-4	-4	-4	-3	-2	-2	-1	-1	0	0	0	0	0	0	0	0	0	0	0

Figure 24: The speed difference between the Patterns speeds minus Analytics speed

5.7: Test Proposed OD Estimation Procedure Using Travel Time

5.7.1: Description of Test and Data

The OD estimation framework using travel time to split bi-directional traffic counts was tested using a mid-sized urban network in Milwaukee, Wisconsin (shown in Figure 10). The OD estimation analysis used a seed OD table derived from the Gravity Model and used the minimum number of turns as the measure for impedance. The link traffic counts were gathered from a variety of data sources that are used by the Wisconsin Department of Transportation to calculate the AADT for freeway and arterials. The data sources include continuous automatic traffic recording (ATR) taken mostly along the freeways and 48 hour tube counts for most of the arterials. Observations of the Mitchell network traffic counts suggests the traffic counts taken on divided arterials mostly have directionality while undivided roadways are mostly recorded as two-way volumes. Figure 25 highlights the bi-directional traffic counts (thick lines) in the Mitchell network. As seen by the figure, the bi-directional traffic counts are mostly on the west side of the network, which is residential area with lower traffic demand.

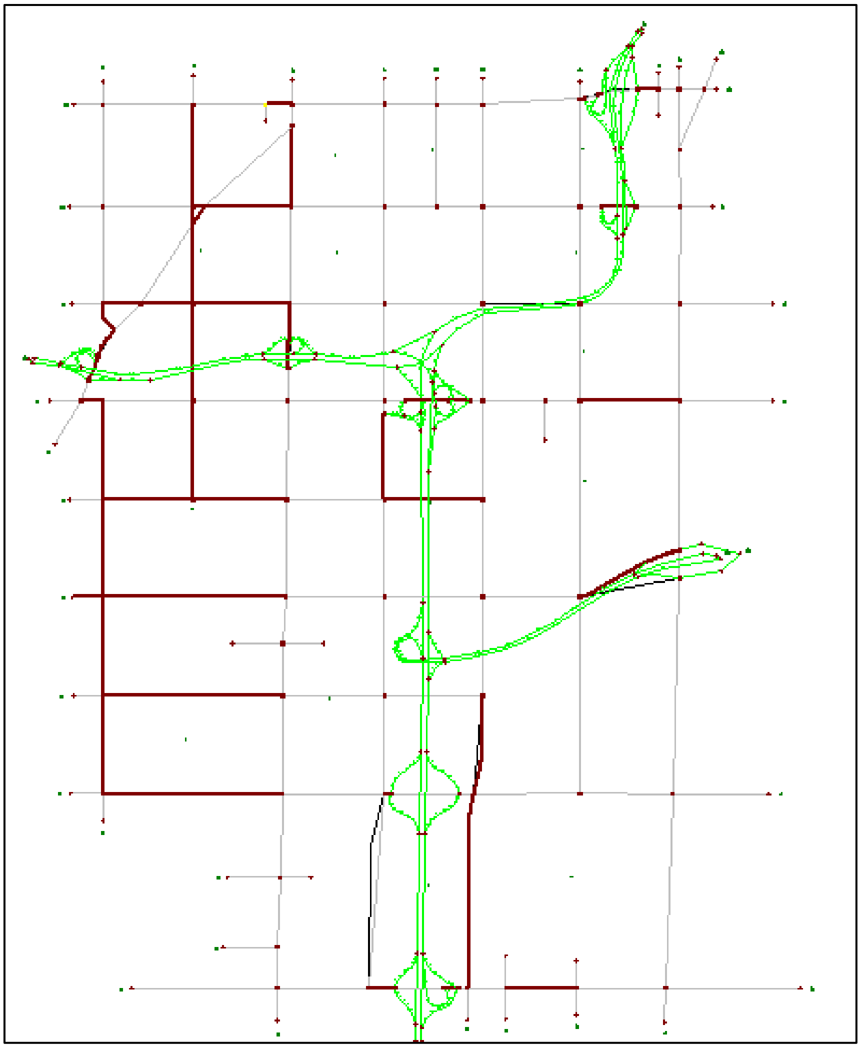


Figure 25: Bidirectional Traffic Counts in Mitchell Network

The travel time from both the floating car runs and the Navteq Patterns database were examined for this analysis. The floating car runs collected data for all of the bi-directional traffic counts shown in Figure 25. The Navteq Patterns database has limited link coverage for bi-directional traffic counts in the Mitchell network as is shown in Figure 26.

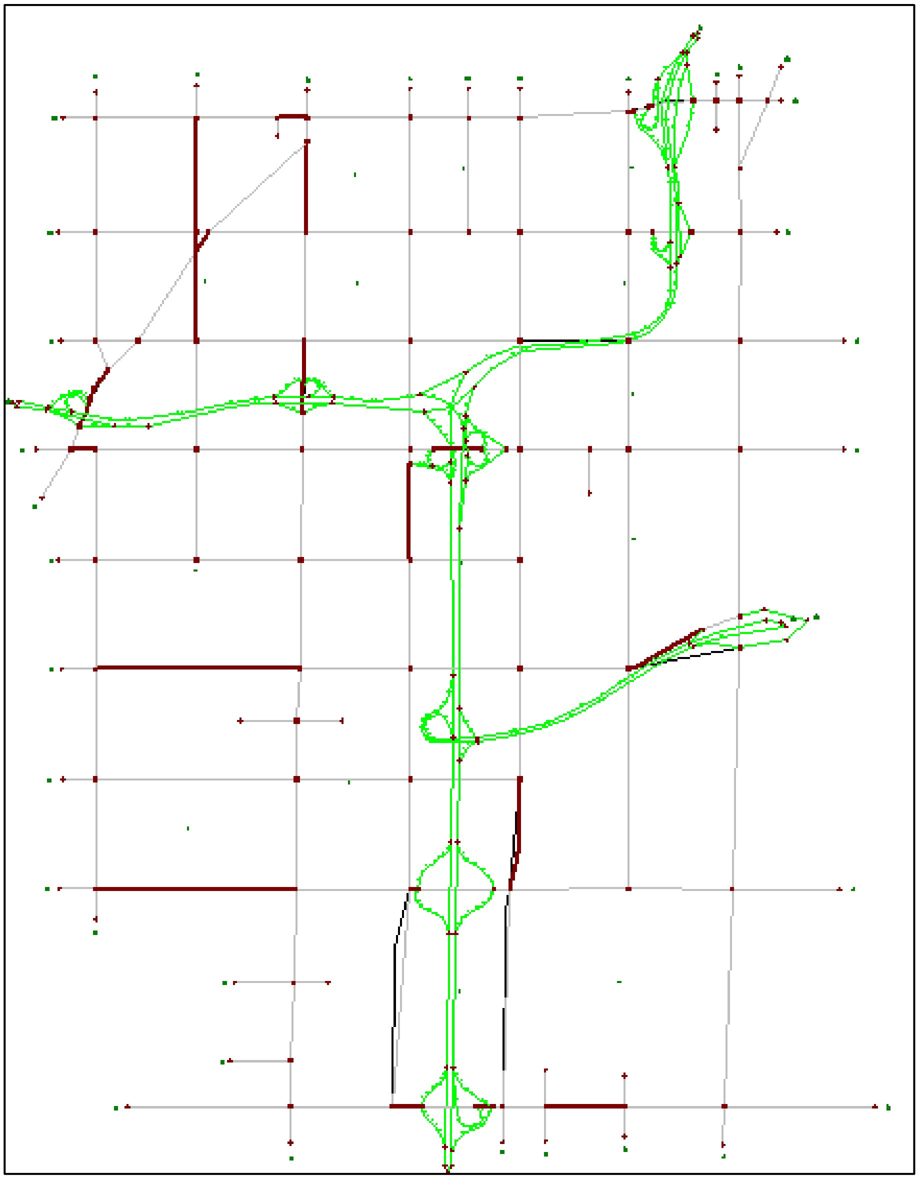


Figure 26: Bi-Directional Traffic Count with Navteq Pattern Travel Time

The limited link coverage of the Navteq data limits is usefulness for the OD estimation in the Mitchell network. The Navteq data has potential in alternative corridors where the travel time coverage better matches the bi-directional traffic count locations. The other problem with the Navteq travel time data for OD estimation, discussed previously in the travel time section, is the Navteq travel time does not correctly capture intersection delay. The procedure Navteq uses to determine travel time needs to be improved inorder for the this OD estimation procedure to be viable.

As noted in the travel time section, the Mitchell interchange was under construction when the floating car runs collected the travel time data. Because the volumes used in the network were taken when the interchange was not under construction, the non-construction Mitchell interchange network was used for the OD estimation tests. The differences in the volume and travel time data sources is not ideal, but is adequate for establishing applicability of the OD estimation procedure.

5.7.2: Implement OD Algorithm in QRS II

This section will discuss how to implement the OD algorithm utilizing travel time with QRS II and coding needs in the GNE network. The first coding requirement is the link traffic counts. The observed peak hour counts for the model are entered in the link attribute fields "ground count A to B" and "ground count B to A". If the counts are directional they should be entered by link direction. If the counts for the link are two-way volumes they should be split 50/50 (or split by engineering judgment) and entered into each of the ground count attribute fields. In addition, in the link attribute dialog box there is an approach code attribute, which specifies the type of count for the link. If the traffic count is directional, a "d" should be entered; if bidirectional a "b" should be entered.

Travel time is also an input variable in the link attributes of the GNE network. Travel time attribute is entered for both directions of the link (attribute "Travel Time A to B (minutes)" and "Travel Time B to A (Minutes")). If travel time is unknown for a link, the field should remain empty. If the travel time attribute is empty the bi-directional volume splitting procedure will not be performed for the link during the OD estimation. However, QRS II will calculate a travel time for the traffic assignment. If travel time is entered and the count is bidirectional, the travel time will be used to derive a directional split for the traffic count. The input travel time in the link attributes is also used for the first traffic assignment equilibrium iteration, but after the first equilibrium assignment, QRS II calculates its own travel time.

5.7.3: Testing OD Refinement Using Travel Time

The proposed OD estimation procedure was tested using the floating car travel time data, full network traffic count coverage, and a seed OD table derived from the Gravity Model. The estimation was completed using the whole table least squares method with a trip table weight of one and equal weighting for each link direction traffic count. The whole table least squares minimization procedure is the traditional method, which seeks to minimize the distance between link traffic assignment and traffic counts, and between OD trip pairs in the seed and final OD matrix. Sixty traffic equilibrium iterations were completed for the OD estimation. For more information about this method and the input variables, see QRS II 8 Reference Manual.

Three model runs were done to compare the performance of the OD estimation procedure. The first run used the floating car travel time to set bi-directional split factors for the real set of bi-directional links shown in Figure 25. The second run used the real set of bi-directional links and nine additional links segments on 27th Street with known directional splits that were converted to bi-directional classification, these nine link segments are shown in Figure 27. The third run used the real set of bi-directional links and five additional link segments on Bolivar Avenue and 20th Street that were converted to bi-directional classification, these links are shown in Figure 28.

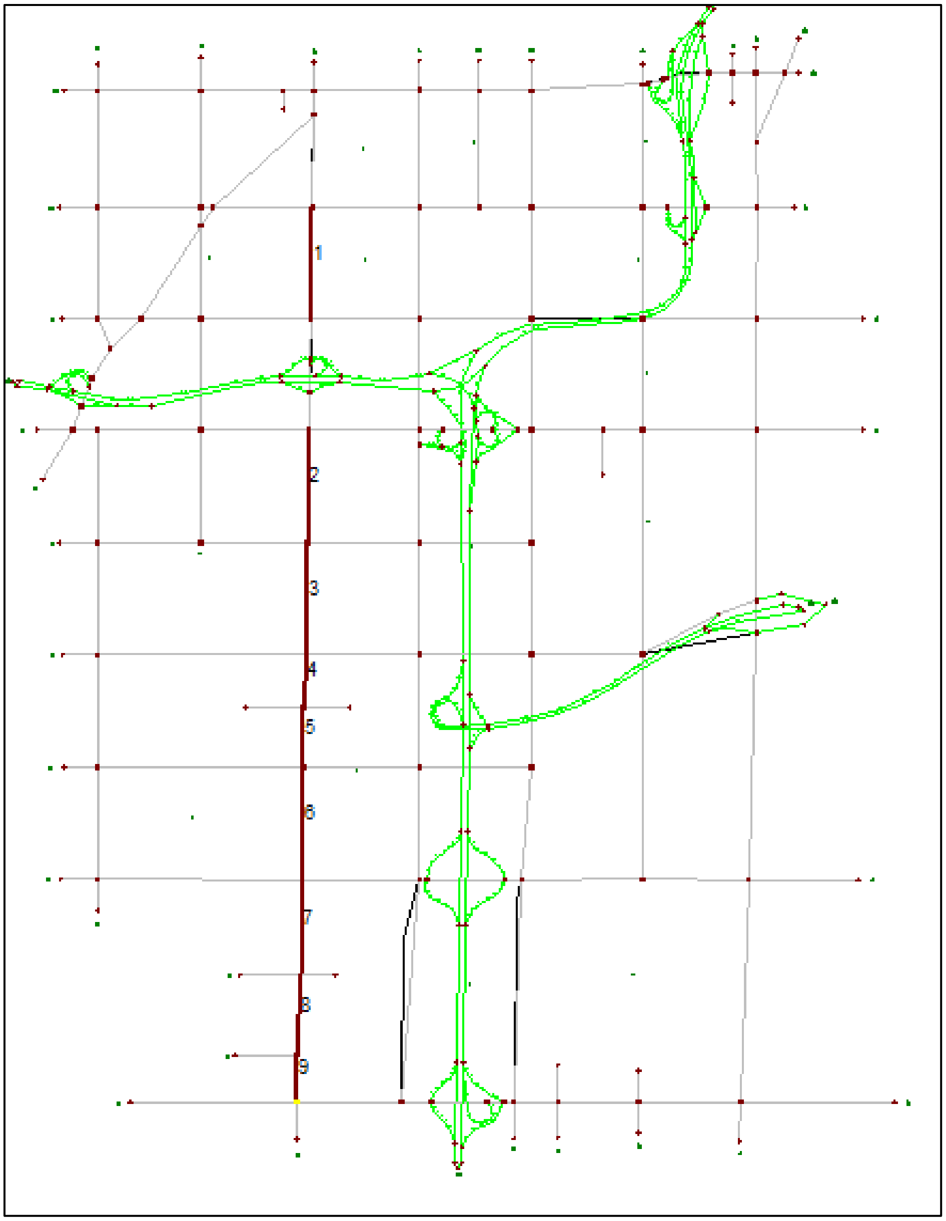


Figure 27: Links with Directional Traffic Counts Set to Bi-Directional for Second Model Run

The directional traffic counts on links changed to bi-directional had their traffic counts set with an initial 50/50 directional split. Therefore, if the directional split procedure fails for one of these links during equilibrium iteration the directional split will revert to 0.5.

For model run 2, the nine link segments along 27th Street evaluated with bi-directional traffic counts all reported average relative error and the λ variable for the travel time-volume function within acceptable ranges (Table 6). The average relative error needs to be less than 0.3 and λ needs to be between 0.05 and 1 for the function to be acceptable.

Link ID	Average Relative Error		λ Variable	
	A to B	B to A	A to B	B to A
1	0.06	0.02	0.37	0.37
2	0.21	0.21	0.42	0.27
3	0.16	0.10	0.43	0.45
4	0.06	0.10	0.42	0.25
5	0.14	0.10	0.40	0.30
6	0.23	0.03	0.22	0.42
7	0.20	0.18	0.29	0.22
8	0.29	0.17	0.28	0.31
9	0.23	0.28	0.33	0.17

Table 6: Average Relative Error and λ For Select Links

After the volume-travel time function is established for a link, the observed input travel time needs to be greater than the free travel time of the link function. If the input travel time is less than the free travel time for the function, the direction split procedure is aborted with the directional split factor reverted to the original input value. In the case of the nine link segments on 27th Street, the free travel time on four links was greater than the input travel time on at least one of the two link directions. For these four links the original 0.5 directional split was used to set target link volumes, these occurrences are highlighted in Table 7. The table also includes free travel time, adjusted free travel time, and QRS II output travel time for the nine links.

While developing the volume-travel time function from the 81 modeled travel time-volume data points, if the modeled free travel time is greater than the modeled travel times, the free travel time is adjusted to be 0.01 seconds less than the lowest modeled travel time. It was anticipated these occurrences would occur early in the equilibrium assignment when the assigned volume is changing by greater increments, which has potential to create major variations in the traffic signal green splits. For the nine link segments, the initial free travel time was greater than the modeled travel time for eleven out of eighteen link directions. This suggests that after sixty equilibrium assignment iterations the assigned volumes for these links were not converging.

A comparison of the input link traffic counts along 27th Street to the assigned volumes using both the travel time to set the directional split factor and using the target split directly in the OD estimation procedure are shown in Table 8. The highlighted directional splits values under column "Assignment Using TT for Splits", indicates links, which reverted to a 0.5 directional split on the 60th equilibrium assignment iteration.

Link ID	Facility	Free Travel Time		Adjusted Free Travel Time		Travel Time			
		A to B	B to A	A to B	B to A	Flt Car A to B	Flt Car B to A	Output A to B	Output B to A
1	27th Street	0.990	0.970	0.990	N/A	0.94	0.91	1.04	1.02
2	27th Street	1.000	1.000	0.940	N/A	1.02	1.53	1.07	1.11
3	27th Street	0.990	0.930	0.960	N/A	1.29	0.77	1.05	1.03
4	27th Street	0.600	0.610	0.600	N/A	0.42	0.51	0.65	0.67
5	27th Street	0.640	0.640	0.610	N/A	1.12	0.64	0.68	0.69
6	27th Street	1.000	0.990	0.940	N/A	1.11	1.17	1.05	1.07
7	27th Street	0.890	0.890	0.880	N/A	0.91	1.29	0.93	1.00
8	27th Street	0.840	0.800	0.700	0.780	0.79	1.13	0.79	0.83
9	27th Street	0.600	0.640	0.560	0.560	0.89	0.38	0.63	0.64

Table 7: Input and Output Travel Time for Test Links in Model Run 2

Link ID	Facility	Traffic Count		Assigned Volume after OD Table Refinement				Directional Split		
		A to B	B to A	Using TT to Split A to B	Using TT to Split B to A	Using Observed Directional Splits A to B	Using Observed Directional Splits B to A	Traffic Count	Assignment Using TT for Splits	Assignment Using Observed Splits
1	27th Street	1500	1104	1205	1000	1213	884	0.58	0.55	0.58
2	27th Street	1452	1782	1402	1540	1529	1300	0.45	0.48	0.54
3	27th Street	1441	1443	1305	1468	1312	1372	0.50	0.47	0.49
4	27th Street	1487	1372	1283	1275	1251	1201	0.52	0.50	0.51
5	27th Street	1487	1372	1323	1121	1234	1133	0.52	0.54	0.52
6	27th Street	1401	1197	1184	1187	1199	991	0.54	0.50	0.55
7	27th Street	1187	1023	1045	1554	1310	1108	0.54	0.40	0.54
8	27th Street	1187	1023	971	1237	1131	969	0.54	0.44	0.54
9	27th Street	1187	1023	1067	973	1083	908	0.54	0.52	0.54

Table 8: Comparison of Assigned Volumes in Model Run 2 After OD Table Estimation Using the Observed Directional Splits Verses Using Travel Time to Set Splits

The results for model run 2 indicate that overall the floating car travel time was ineffective in setting target directional splits for 27th Street links. For the sixtieth equilibrium iteration, four of the nine links used the original directional splits because the input travel time from the floating car runs was less than the free flow travel time calculated by QRS II. Assigned volumes for link ID 2, 5, and 6 evaluated close to the target directional split. The other two links that successfully evaluated (link ID 7 and 8); set directional splits with the heavy volumes in the B to A direction while the target counts were heavier in the A to B direction. For both of these location the input travel time in the B to A direction was much higher than A to B, and therefore calculated as a much higher volume during the directional split procedure. These two links indicate the directional split procedure is effective, but the travel time for this data set does not represent the traffic counts.

Due to possible discrepancies with the travel time another location in the Mitchell network was chosen to test a third model run of the OD estimation procedure. The model run takes the real bi-directional links with the addition of another five links along 20th Street and Bolivar Avenue that were changed to bi-directional for this analysis.

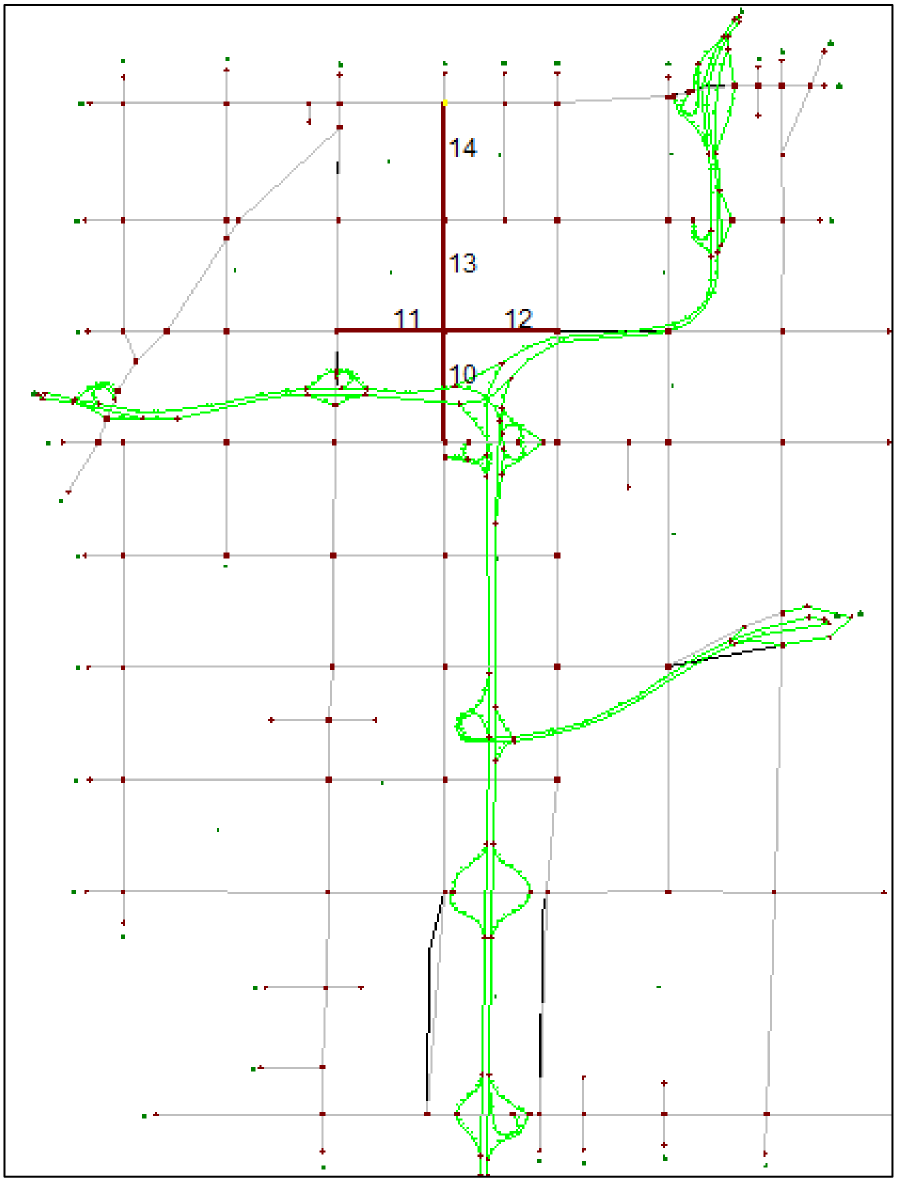


Figure 28: Links with Directional Traffic Counts Set to Bi-Directional for Model Run 3

Out of the five links analyzed in model run 3, one of the links did not meet acceptable average relative error, while another was close to the 0.3 threshold. These links with average relative error and λ variable are summarized in Table 9.

Link ID	Average Relative Error		λ Variable	
	A to B	B to A	A to B	B to A
10	0.29	0.08	0.18	0.73
11	0.07	0.22	0.75	0.33
12	0.32	0.13	0.12	0.86
13	0.14	0.07	0.74	0.29
14	0.08	0.23	0.52	0.48

Table 9: Average Relative Error and λ Variable for Select Links in Model Run 3

Model run 3 had two of the ten-link directions needing a modeled free travel time adjustment to derive the volume-travel time function. Link 13 has an input travel time less than the free flow travel time calculated by QRS II. So, two out of the five links fail to evaluate direction split, link 12 because of average relative error and link 13 because of the input travel time (see Table 10).

Link 10 and 11 both evaluated close to the default directional split of 0.5. Link ID 14 had similar results to the assigned volume using predetermined split factors, however the resultant split factor was much greater than the target split factor (See Table 11).

Link ID	Facility	Free Travel Time		Adjusted Free Travel Time		Travel Time			
		A to B	B to A	A to B	B to A	Flt Car A to B	Flt Car B to A	Output A to B	Output B to A
10	20th Street	1.160	1.050	1.120	N/A	1.83	1.81	1.20	1.09
11	Bolivar Avenue	1.030	1.070	N/A	N/A	1.58	1.96	1.11	1.14
12	Bolivar Avenue	1.060	1.060	N/A	N/A	1.57	1.29	1.15	1.12
13	20th Street	1.050	1.080	N/A	N/A	0.98	0.89	1.08	1.09
14	20th Street	1.160	1.160	1.140	N/A	1.47	1.36	1.17	1.25

Table 10: Input and Output Travel Times for Test Links in Model Run 3

Link ID	Facility	Traffic Count		Assigned Volume after OD Table Refinement				Directional Split		
		A to B	B to A	Using TT to Split A to B	Using TT Split B to A	Using Observed Directional Splits A to B	Using Observed Directional Splits B to A	Traffic Count	Assignment Using TT for Splits	Assignment Using Observed Splits
10	20th Street	560	404	602	547	753	439	0.58	0.52	0.63
11	Bolivar Avenue	201	150	340	317	332	390	0.57	0.52	0.46
12	Bolivar Avenue	126	199	69	309	57	399	0.39	0.18	0.13
13	20th Street	581	310	508	418	601	337	0.65	0.55	0.64
14	20th Street	344	251	568	210	590	230	0.58	0.73	0.72

Table 11: Comparison of Assigned Volumes in Model Run 3 After OD Table Estimation Using the Observed Directional Splits Verses Using Travel Time to Set Splits

Overall, the procedure using travel time to obtain directional split factors for the OD estimation shows potential, but in the preliminary testing did not meet expectations. The lambda and average relative error was acceptable for most of the test links as shown in Figure 29 and Figure 30.

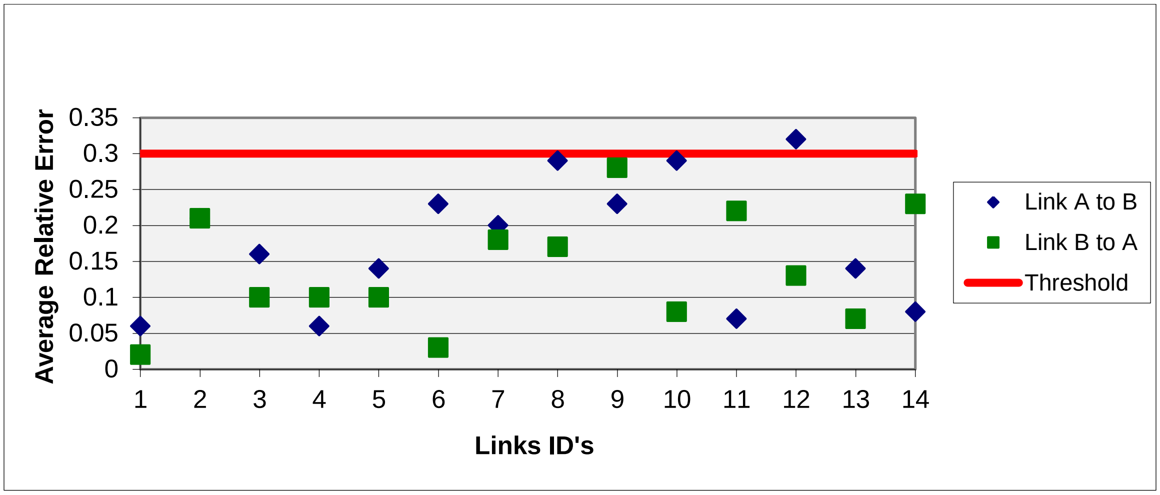


Figure 29: The Average Relative Error for the 14 Test Links

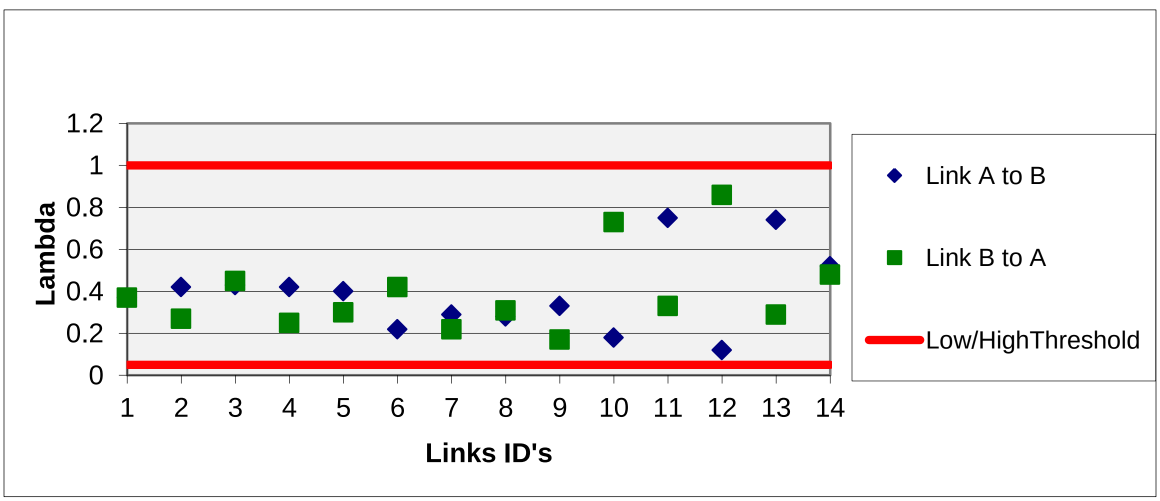


Figure 30: The evaluated Lambda variable for the 14 Test Links

The major cause for errors with the volume-travel time function was due to the input floating car travel time being smaller than the free flow travel time calculated by QRS II. Either these errors are due to the floating car data not representing the traffic counts, possibly because of the construction in the area, the errors are because the Mitchell network was not properly calibrated, or the macroscopic model is unable to capture intersection specific delay deviations. The free travel time to input comparison of the 14 test links are shown in Figure 31, and Figure 32.

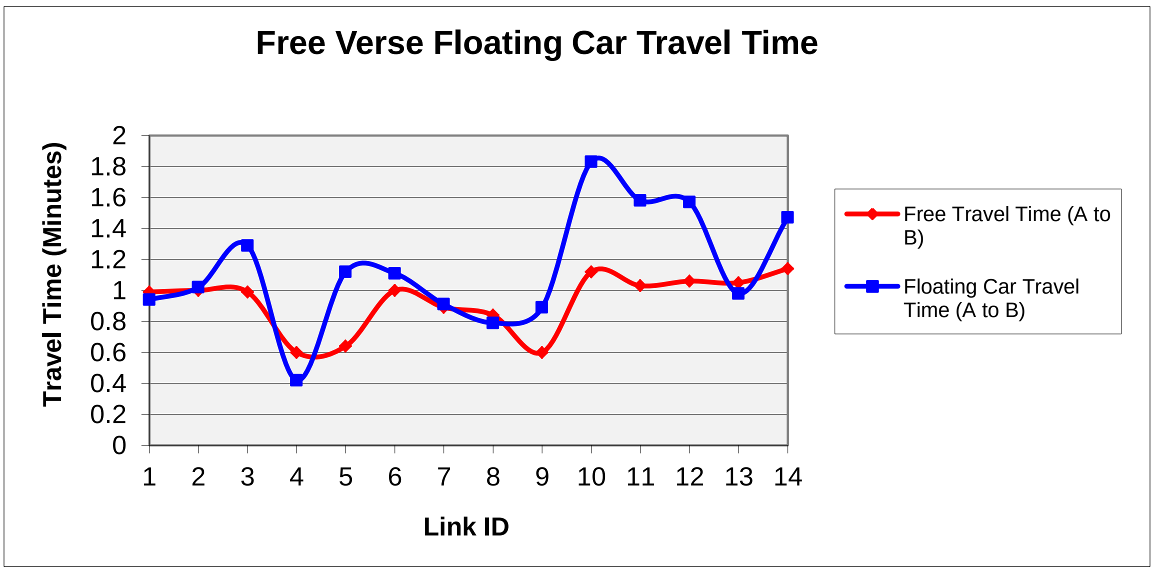


Figure 31: Free Verse Floating Car Travel Time in the A to B Direction

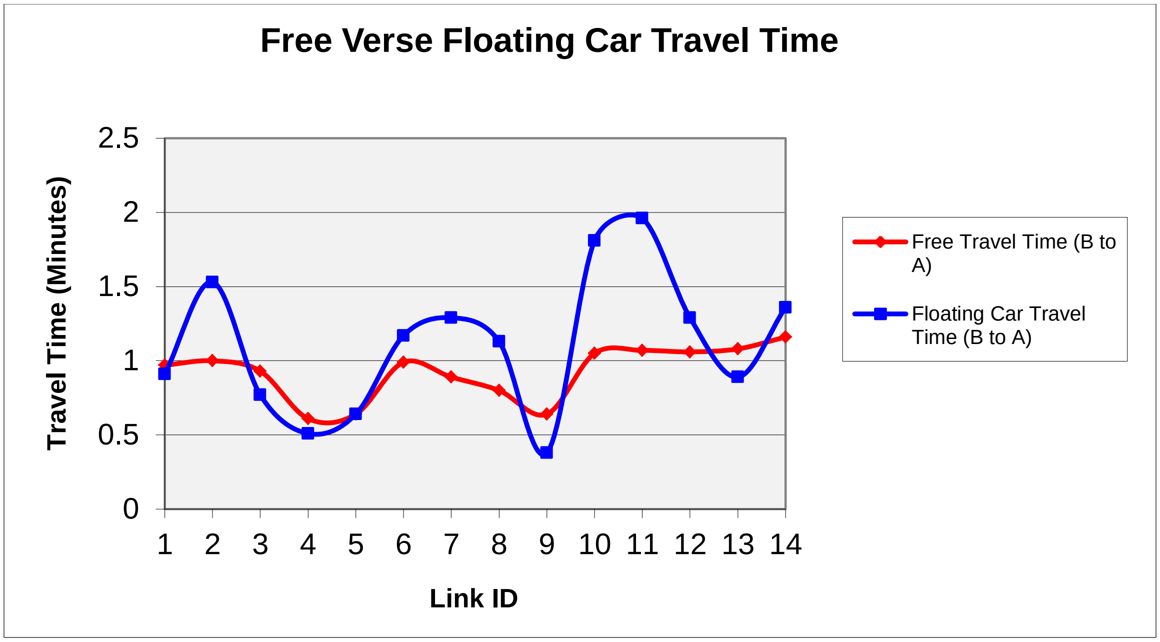


Figure 32: Free Verse Floating Car Travel Time in the B to A Direction

The travel time comparison between the floating car runs and the QRS II output travel time shows that for most arterials the floating car runs had larger travel times when summed over the whole corridor. Given that overall arterial travel times were higher for the floating car runs, but link specific were lower for many links in this test suggests that major delay is accumulated on some intersections, but other intersections receive little delay due to progression from coordinated traffic signals. QRS II captures the overall travel time effects of progression, but spreads the travel time improvements of progression across all intersections in the corridor. The effects from progression may have been exacerbated because the floating car runs mostly captured only the through movement at the intersections.

The outcome of the OD estimation procedure using travel time to set directional split factors was mixed. Figure 33 shows the directional split results of the 14 test links. The figure compares the observed directional split from traffic counts to the assigned using travel time and assigned using the observed split. It shows 7 out of the 14 test links having the directional split fail on the last equilibrium traffic assignment. While function that did not fail had directional splits setting the opposite direction as major as compared to the observed traffic counts. This situation happened for links 6, 7, 8, 9. However, in these cases the travel

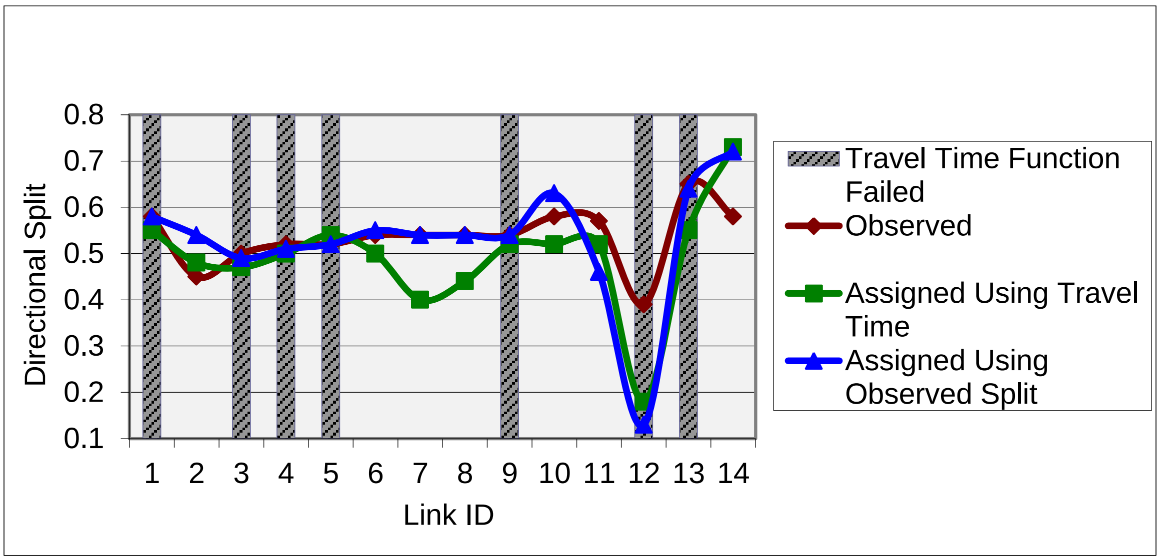


Figure 33: Comparison of Directional Splits for the 14 Test Links

More research is needed on the OD estimation technique using travel time to split bi-directional traffic counts. This analysis showed that the technique is viable if the travel time is representative of the target traffic counts. However, the analysis indicates shortcomings in the travel time collection and macroscopic modeling capabilities at individual intersections leading to discrepancies in approach delay and link travel time.

CHAPTER 6: SUMMARY AND RECOMMENDATIONS

This thesis presented findings on the implementation of actuated traffic signals in the macroscopic framework and the use of travel time in the OD estimation procedure. Both objectives of the thesis show applicability using a mid-sized network in Milwaukee, WI.

6.1: Summary Actuation in QRS II

The implementation of the actuated procedure in QRS II, which is a modification of a HCM 2010 procedure for calculating actuated signal green splits, showed adequate convergence and reasonable green splits for uncoordinated actuated signals. The sensitivity of the actuation procedure was presented by modifying the minimum and maximum green times, passage times, and detector lengths. The analysis also indicated proper convergence of the equilibrium traffic assignment with actuated traffic signals.

The actuation procedure in QRS II has the following assumptions and simplifications as compared to the HCM procedure:

- Simultaneous gap out
- Passage time and detector lengths are defined for left and thru traffic at the network level
- Limits lane groups to: exclusive protected lefts, through movements, and left exclusive permitted
- Protected and permitted lefts are determined based on opposing through and left turn volumes
- One left exclusive lane that is protected is assumed to be protected-permitted
- Two left exclusive lanes that are protected are assumed protected only.

- No coordinated-actuated signals
- No pedestrians

6.2 Summary OD Estimation Using Travel Time

A methodology for estimating an OD matrix using travel time in conjunction with traffic counts and pattern OD matrix was presented. The OD estimation methodology uses travel time to derive directional split factors for bi-directional traffic counts to set directional target volumes for the OD estimation routine. Travel time data from floating car runs and Navteq (private sector vendor) were compared for use in the OD estimation routine. It was found that the Navteq data in its current form is not adequate for the OD estimation procedure because Navteq does not capture delay from intersections. The floating car data was also found to have some errors due to the collection technique and from the time of collection because of construction in the area, but was determined to be adequate for testing applicability of the OD estimation procedure.

The OD estimation procedure was analyzed using the floating car travel time and showed that travel time data has potential to be used in the OD estimation framework. However, further research is needed to overcome deviations in signalized approach delay calculated in the macroscopic models compared to real life traffic conditions. In addition, for this OD estimation procedure to be effective for large networks, large travel time databases are needed. The most viable source for the travel time data is from private sector vendors. Further research is needed to analyze alternative private sector travel time vendors for applicability of their data in the OD estimation procedure.

6.3: Future Research on Actuated Signal in Macroscopic Models

Future work is needed on the actuated signal module in QRS II to improve and further validate the existing implementation. The following work is needed:

- Compare QRS II green split outputs on existing actuated signals to observed field data. This would help validate the assumptions made while implemented the HCM procedure into QRS II.
- Develop a technique to incorporate actuated-coordinate functionality into the macroscopic model.

6.4: Future Research on OD Estimation Using Travel Time

Future work is needed in order for the OD estimation technique using travel time to be effective in large scale networks. The following areas need further research:

- **Travel Time Collection**: For the OD estimation procedure to be effective travel time on links segments, need to be the weighted average of all lane groups in an approach. The floating car runs for the Mitchell network measured the delay for the through movement for the majority of the intersection approaches. Creating a floating car course, which encompassed all approach turning movements in proportion to the turning volumes, would be the most accurate way for establishing average link travel time. However, collecting floating car data in this manner would be very time consuming and expensive. The private sector travel time data is the most feasible input data for the OD estimation algorithm because of its extensive speed sampling and network coverage. However, the private sector data (Navteq) needs to capture intersection delay for them to be a viable travel time source. Other private sector travel time vendors may have the travel time data needed for this procedure.

- **Approach Delay**: The QRS II planning model uses the HCM procedure with signalized intersection algorithms (set phase green splits) to calculate approach delay. The effects of coordinated signals in a macroscopic model are best modeled over an entire corridor not with delay of an individual link. Further research is needed to overcome the effects of progression on coordinated signals in macroscopic models. In addition, further research is needed to better replicate signal-timing plans in the macroscopic model. The actuation module presented in this thesis presents finding on adding actuated signals to the macroscopic models, which allows the models to better replicate real life traffic control strategies.
- **New Procedure:** Develop a procedure that would be able to overcome the lack of consistency between HCM calculated travel time and Navteq travel time, so Navteq data could be used effectively in OD estimation.

CHAPTER 7: CONCLUSION

The objective of this thesis was to improve the calibration of macroscopic models through two separate research topics. The first topic was to develop and test a methodology for implementing actuated signals within the macroscopic modeling framework. The second research topic was to utilize private sector travel time data to aid OD table estimation. This research was done in collaboration with Dr. Horowitz, who was responsible for programming both the actuation module and the OD estimation using travel time into QRS II macroscopic modeling framework.

The actuation research topic started with the need to calculate a representative fixed time cycle length and green phase durations for actuated signals in a macroscopic transportation model. The technique for representing actuated signals needed to utilize basic actuated timing variables and be compatible with macroscopic model calculations. The research used an existing procedure in the Highway Capacity Manual that calculates average phase durations and cycle lengths for the actuated signals. Unfortunately, the HCM procedure was not completely compatible with the macroscopic modeling framework. Simplification of the HCM procedure were initially prototyped using a basic 8-phase intersection example in a C++ programming environment independent of the QRS II program. The initial prototype indicated the HCM procedure could utilize input variables available in the macroscopic model with the addition of minimum and maximum green time, passage time, red time, yellow time, and detector lengths to calculate an average intersection cycle length and phase durations. Further assumptions for a complete implementation into QRS II were collaborated with Dr.

Horowitz. Analysis presented in the thesis showed the actuation module in QRS II is capable of representing the sensitivity of actuated input variables and shows adequate convergence during equilibrium traffic assignment. Although further research is needed on the topic of actuation in macroscopic models, this analysis suggests actuated timing plans can be used in macroscopic models to calculate average delay.

The research on origin-destination table estimation using travel time was initiated because of a need to derive directional specific target volumes for input into the OD minimization framework. The directional specific targets provides more information on links with bi-directional traffic counts during the OD estimation, which would allow the model to better represent real life traffic patterns. This research topic was also pursued because of the availability of private sector travel time data. This research used data from Navteq because it was available from the Wisconsin Department of Transportation.

The OD table estimation procedure used the reversed BPR curve to relate volume as a function of travel time. During the development of the volume-travel time relationship it was found that the function could not be developed with standard variables used for all links because the volume-travel time relationship is dependent on the interaction of vehicles on the adjacent and opposing approaches of an intersection. Because of this a modeling procedure was developed that perturbed the approach volumes creating 81 volume-travel time data points. A non-linear regression procedure was adopted from ALGIB source code to estimate variables of the reversed BPR function to best represent the 81 data points for each link direction. These link direction functions were used to estimate the directional split factor of the links during the OD estimation. The results from the OD table estimation using travel time showed potential, but had some fatal flaws partially due to poor input travel time. Further research is needed using a new network with updated traffic counts, and travel time. Although during the OD table estimation the results indicated difficulties in replicating the directional factors using travel time there are important lessons to take away from the research. The lessons learned which can be applied to later research include:

- A volume-travel time function or relationship needs to consider impacts of volume on adjacent and opposing approaches.
- Navteq travel time data has very good network coverage. However, its applicability in urban signalized arterials is limited because the database does not account for intersection delay. Further advancements in data collection need to take place by Navteq for the data to be viable in this OD estimation framework. Travel time data from alternative private sector vendors should be explored for applicability in this OD estimation framework.
- A well-calibrated macroscopic model can be capable of replicating field measured travel time along a corridor. However, it is difficult for a macroscopic model to match link segment specific travel time. This research showed that impacts from coordinated traffic signals can cause great variations between field measured travel time and modeled travel time based on HCM calculations. This is because HCM applied travel time improvements generically to intersections;

while real life coordinated signals will cause vehicles to have minimal delay at some intersections and major delay at others.

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