



AUBURN

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COLLEGE OF ENGINEERING

Research Report, Part 2

# ANALYSIS OF TRAFFIC IMPACTS FOR RAPID BRIDGE DECK REPLACEMENT PROJECTS

*Submitted to*

The Alabama Department of Transportation

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## Highway Research Center

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*Research Supervisors*

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**ANALYSIS OF TRAFFIC IMPACTS FOR RAPID BRIDGE DECK REPLACEMENT  
PROJECTS**

**Part 2: Microsimulation Analysis of the Impact of  
Different Bridge Deck Replacement Methods on  
Travel Times and Road User Costs**

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# **ANALYSIS OF TRAFFIC IMPACTS FOR RAPID BRIDGE DECK REPLACEMENT PROJECTS**

## **Part 2: Microsimulation Analysis of the Impact of Different Bridge Deck Replacement Methods on Travel Times and Road User Costs**

### **Abstract**

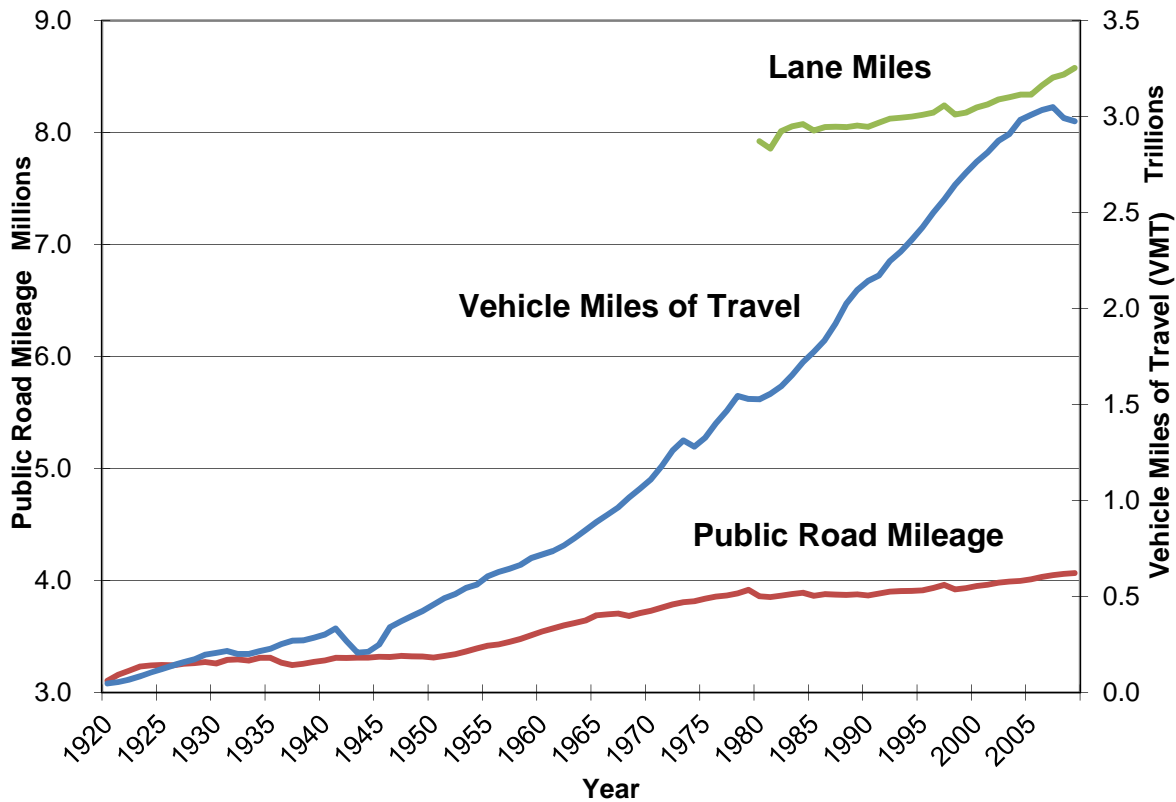
The objective of this research was to evaluate the traffic impacts of different bridge deck replacement methods using microsimulation software. Previously collected data from the test site, located on I-59 at State Route 68 near Collinsville, AL, were analyzed and traffic engineering statistics were extracted. This data were used in creating a working base model, representing the conditions during data collection, in the traffic microsimulation software package VISSIM. Alternative models were created from the base model to simulate traffic conditions, which might occur, during different bridge deck replacement methods. These alternative models provided travel time data to quantify the effects that different bridge deck replacement methods have on road users. It is recommended that alternative bridge deck replacement methods be considered for higher volume roadways, greater than 1600 vehicles per hour per lane, based on resulting travel time.

## **Chapter 1: Introduction**

### **1.1 Background**

Nearly 50% of all bridges in the United States were built between 25 and 50 years ago. This is a major concern for any state or city transportation agency since many of these bridges have a design life expectancy of 50 years. In fact, many of the current bridges in use are nearing the end of their serviceable life and are in need of repair or replacement. In addition, about 26% of all bridges in the United States are structurally deficient. Moreover, highway capacity has increased little over the last two decades (Mistry and Mangus 2006).

At the same time the U.S. has seen an increase in vehicle miles traveled (VMT), as seen in Figure 1-1. This figure also displays the growth of public roads over the same time period. From 2003 to 2009, VMT grew from 2,908,778 million miles in 2003 to 2,996,631 million miles in 2009, a 3.02% increase in VMT. The miles of public roadways, during the same period of time, increased by only 1.9% from 3,990,899 to 4,067,396 miles and in lane-miles these values are 8,315,121 miles in 2003 and 8,577,179 miles in 2009, an increase of 3.2% (Public Road Mileage 2011). Alabama followed this same growth pattern in VMT with 58,637 million VMT in 2003 to 59,303 million VMT in 2008, a 1.1% increase in VMT (BTS 2009). When coupled with aging roadways and bridges requiring reconstruction, rehabilitation, or maintenance throughout the country, this growth in VMT can cause serious but potentially avoidable delays for roadway users during road and bridge reconstruction, rehabilitation, or maintenance activities.



**Figure 1-1 Public Road Mileage and VMT 1920 to 2009 (FHWA, 2011).**

In 2009, Alabama had 15,959 bridges throughout the state. Of these, 10.6% were classified as being structurally deficient and 13.2% functionally obsolete. There are 2,720 bridges in the state that are located in urban areas (BTS 2009). With an increasing need to repair older bridges in heavy populated areas, new methods of roadway repair must be examined to prevent unnecessary commuter delay as a result of roadway repair and maintenance activities. To examine the effects of maintenance and maintenance methods on traffic it will not always be feasible to study the effects in an active construction area.

“In order to assess the potential effectiveness of a particular strategy, it must be analyzed using traffic analysis tools or methodologies” (Alexiadis 2004). Technology provides tools and techniques that allow traffic engineers the ability to analyze traffic under multiple scenarios virtually, without the event actually taking place. Microsimulation modeling is one of the tools used to analyze the effects different reconstruction, rehabilitation, or maintenance methods will have on road users.

## **1.2 Motivation for the Research**

The Alabama Department of Transportation (ALDOT) is currently studying different options for bridge deck replacement for the more highly-travelled highways in the state. In doing so ALDOT is comparing the traditional bridge deck replacement method, which leaves the entire lane of work closed until the work is completed, to a rapid bridge deck replacement method, in which the work is performed during off-peak hours (i.e., nights and weekends) and the affected lane is open during peak traffic times. In comparing these two techniques there are two different methods of applying the new concrete deck to the bridge. The first is a cast-in-place (CIP) technique where the concrete is poured on the bridge and left to cure there; the other is a pre-cast concrete that is poured and cured at a fabrication site, transported to the bridge site, and placed on the bridge. This brings the total number of deck replacement options to four: (1) traditional/pre-cast, (2) traditional/CIP, (3) rapid replacement/pre-cast, and (4) rapid replacement/CIP. It is hypothesized that the rapid bridge deck replacement methods will have less travel time delay associated with construction than a traditional deck replacement method. Also, it is hypothesized that the pre-cast concrete deck replacement methods will cause less travel time delay than the two CIP replacement methods. With less delay expected, the rapid deck replacement method's expected road user cost (RUC), on overall cost of travel for a section of roadway for each road user, will be less than that of a traditional deck replacement method. This research effort will use the traffic microsimulation software package VISSIM, developed by Planung Transport Verkehr AG (PTV), to run simulated models of traffic conditions that are expected with the different construction methods mentioned. The different models will be compared to one another based on traffic measurements of travel time, speeds, and RUC.

## **1.3 Research Objectives**

The objective of this research is to analyze the effects that different bridge deck replacement methods: (1) traditional/pre-cast, (2) traditional/CIP, (3) rapid replacement/pre-cast, and (4) rapid replacement/CIP, have on traffic conditions and road user cost of the motorist. The different deck replacement methods have two distinct sequencing and traffic control setups. A traditional bridge deck replacement method will close a single lane of travel for the entirety of the construction project and have temporary traffic control devices (TTCD) as mandated in the Manual on Uniform Traffic Control Devices (MUTCD). The alternative deck replacement method is a rapid bridge deck replacement. In this method the TTCD will be set up as specified in the MUTCD, just like a traditional bridge deck replacement. The construction sequence for this method calls for the lane to be closed during low volume hours (i.e., nights and weekends) and to reopen during peak hours or after the deck is structurally sound to drive on. This process repeats for each section of bridge decking, so for the rapid bridge deck replacement method the lane of travel is only closed when necessary and reopened when it is once again structurally sound to drive on. In Part 1 of this report, estimates were developed for the length of time for a single section of bridge to be replaced using the rapid deck replacement method depends on the concrete used. For CIP concrete the total working hours can range from 16.5 to 18.5 hours and for pre-cast concrete, the total working hours can range 9.5 to 12 hours. These times represent

the actual construction time and do not take into account end of shifts or any other work stoppage. Travel times obtained from microsimulation models are used to calculate the RUC for the four different bridge deck replacement methods, these RUC represent the expected impact to the road users at the test site. Additional RUCs are calculated to compare different lengths of traditional lane closure time.

The information obtained from this research can be used by decision makers such as traffic engineers, maintenance engineers or other individuals that make decisions regarding roadways, along with previous research, to determine which bridge deck replacement method is most suited in a location. The tasks executed to accomplish the objective of this research, to analyze the effects that different bridge deck replacement methods have on traffic conditions and road user cost of the motorist, are:

1. Conduct a literature review, documenting ways to develop an accurate traffic model and how to calculate a road user cost.
2. Develop a microsimulation model, in VISSIM, of the proposed test site for the rapid bridge deck replacement methods on I-59 at State Route 68 near Collinsville.
3. Execute and analyze the models for different bridge deck replacement methods based on VISSIM outputs.
4. Develop a congestion (variable volume) model to show effects of the associated temporary traffic control plans for the different deck replacement methods on traffic conditions for higher volumes than those at the test site.
5. Use model outputs to analyze road user costs for the candidate bridge deck replacement methods.

#### **1.4 Organization of Part 2 of This Report**

The remainder of this report is divided into four chapters that illustrate the steps taken to complete the tasks necessary to satisfy the overall objective of the research described above. *Chapter 2: Literature Review* includes relevant information found on traffic modeling of a construction site to identify the most accurate models, as well as information on establishing RUCs. *Chapter 3: Methodology* includes the steps taken to create accurate models and the steps taken to analyze the data from the models. *Chapter 4: Data Analysis* includes the analysis work of the research; in this chapter all of the model sets presented in Chapter 3 are analyzed and studied. The RUC is calculated in this chapter to provide a monetary value to the delays associated with the different models. *Chapter 5: Conclusion and Recommendations* includes a summary of the overall results from Chapter 4. Also in this chapter are recommendations on how traffic engineers or other decision makers can use the findings in this research for planning of bridge deck replacements. Recommendations for further research in this area are also presented in this chapter.

## **Chapter 2: Literature Review**

### **2.1 Introduction**

To understand the effects that traffic volumes have on delay in construction work zones it is necessary to understand transportation engineering terminology and concepts pertaining to speed, travel time, and capacity. In this chapter some of the terminology and concepts will be defined. This will provide better understanding of these terms and concepts used throughout the later chapters.

Modeling software was used to analyze the different construction methods. Smadi and Baker (2009) researched the usefulness of different traffic modeling software packages to model different traffic scenarios. Upon their recommendation the chosen software, VISSIM, was used for this research. Although VISSIM has a high learning curve, once a base model is successfully calibrated it is easy to simulate alternative alternatives, such as different lane configurations (Smadi and Baker 2009).

VISSIM is a traffic microsimulation package meaning that each vehicle is modeled independently from one another. To fully understand and use the software, a review of the literature on this topic was necessary. The most common theme found in the literature review was the need to successfully calibrate the model to replicate actual traffic patterns. Successful calibration of models ensures the models correctness and is an important step with all simulation modeling. Throughout the literature, approaches that focus on adjusting different variables in VISSIM to replicate actual traffic patterns are discussed. These different approaches will be examined throughout this chapter.

The last topic reviewed in this chapter is the calculation of road user cost (RUC). While there are multiple methods for determining RUCs, this paper focuses on RUC due to changes in delay, additional vehicle hours traveled beyond free flow conditions. In this chapter the method for determining road user cost presented in the AASHTO publication “User and Non-User Benefit Analysis for Highways”, also called the “Red Book”, along with other methods will be examined.

### **2.2 Relevant Transportation Engineering Terminology**

Volume and flow rate are two measures that can quantify the number of vehicles that pass a point on a roadway during a given time interval. *Volume* is the number of vehicles that pass a given point or section of roadway during a given time interval. *Flow rate* is the equivalent hourly rate at which vehicles traverse a given point or section of roadway during a time period less than 1 hour, usually in 15 minute intervals (HCM 2000). These measures quantify the demand on the roadway, that is, the number of vehicles that desire to use the roadway for a given time interval. It is important to distinguish differences in volume and flow rate.

Speed (and its reciprocal travel time) is an important measure of the level of service provided to a motorist. *Speed* is a rate of motion expressed as distance per unit of time, generally miles per hour (mph). *Travel Time* is the time in which a vehicle passes from one defined point to another. When characterizing the speed of a traffic stream a representative value must be used because of the broad distribution of speeds observable in the traffic stream. The *space mean speed* is calculated by dividing the length of the roadway (or segment) by the average travel time of the vehicles traversing it. This relation can be seen in Equation 2-1. The travel times in this computation include stopped delays, which are the total travel times to traverse the defined roadway section (HCM 2000).

$$S = \frac{nL}{\sum_{i=1}^n t_i} = \frac{L}{\frac{1}{n}\sum_{i=1}^n t_i} = \frac{L}{t_a} \quad (2-1)$$

Where: S = average travel speed (mph)  
 L = length of the highway segment (mi)  
 t<sub>i</sub> = travel time of the i<sup>th</sup> vehicle to traverse the section (hr)  
 n = the number of travel times observed  
 t<sub>a</sub> = average travel time over length of highway (hr)

The *Highway Capacity Manual 2000* (HCM 2000) defines *capacity* as: “the maximum sustainable flow rate at which vehicles or persons reasonably can be expected to traverse a point or uniform segment of a lane or roadway during a specified time period under given roadway, geometric, traffic, environmental, and control conditions. Capacity is usually expressed as vehicles per hour, passenger cars per hour, or persons per hour” (HCM 2000). In the HCM 2000 it is recommended that a capacity of 1600 passenger cars per hour per lane (pc/h/ln), or equivalent, be used for short-term freeway work zones, regardless of the lane closure configuration. *Passenger car equivalent* is the impact other modes of transportation (e.g., busses, trucks, and recreational vehicles) have on traffic in terms of passenger cars. This number can be adjusted based on other factors, such as:

- Intensity of Work Activity: this refers to the number of workers and amount of equipment in the work zone. It is suggested that during work activity which is more intense than normal, the 1,600 pc/h/ln can be adjusted by ±10%. The term “normal” is not defined; therefore, engineering judgment should be used to make this decision.
- Effects of Heavy Vehicles: vehicles that are over 40 ft in length and have a weight of 18,000 lbs or more. It is recommended that the heavy vehicle adjustment factor be used for this, which is shown in Equation 2-2 below.

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)} \quad (2-2)$$

Where: f<sub>HV</sub> = heavy vehicle adjustment factor  
 P<sub>T</sub> = proportion of heavy vehicles

$E_T$  = passenger car equivalent for heavy vehicles, a function of terrain  
(found in Chapter 23 of the HCM 2000)

- Presence of Ramps: if there is an entrance ramp within the taper area approaching the lane closure or within 500 ft downstream of the beginning of the full lane closure, the ramp will have a noticeable effect on the work zone capacity.

For long-term construction work zones, based on engineering judgment, the HCM2000 suggests that when a two lane freeway is reduced to 1 lane of travel then the capacity is reduced to 1550 pc/h/ln (HCM 2000). Additional adjustment can be made to both long-term and short-term work zones based on lane width. For traffic with passenger cars only, time headways increase about 10 percent going from 11 ft width lanes to lanes with 10.5 ft to 10 ft widths and by an additional 6 percent when the width is reduced to 9 ft (HCM 2000). These increases in headway translate to 9 and 14 percent drops in capacity for the narrower lane widths within the construction zone. Other factors such as adverse weather and vehicle incidents within the construction zone can have a negative effect on the capacity (HCM 2000).

Research conducted by Heaslip et al. 2009, derived a set of capacity equations for three different lane closure scenarios: 2 to 1 lane closure, 3 to 2 lane closure and 3 to 1 lane closure. The 2 to 1 lane closure will be examined as it represents the lane configuration of this research. Equation 2.3 represents the unadjusted roadway capacity; this equation has an  $R^2$  value, a statistical measure of how well a regression line approximates real data points, of 0.915. In this equation Lane1 refers to the closed lane during construction.

$$C_{unadj}^{2-1} = 1855 - 693 \times SignDist + 191 \times f_{HV-F} - 12.3 \times Rubber\% - 467 \times DistrLan1 + 829 \times DistrLan1 \times SignDist + 7.43 \times SpeedLan1_{adj} \times SignDist \quad (2-3)$$

Where:  $C_{unadj}$  = unadjusted roadway capacity for 2 to 1 travel lanes  
 SignDist = distance of upstream warning sign from beginning of work zone (mi)  
 Rubber% = 0% for no work activity and 5.6% for presence of work activity  
 DistrLan1 = percentage of traffic in lane between warning sign and work zone  
 SpeedLan1<sub>adj</sub> = unadjusted speed in lane one upstream of the warning sign (mph)

Equation 2-4 adjusts capacity based on such factors as: lighting conditions, driver population, rain, and ramp presence. These two equations can be used to estimate the roadway capacity at a work zone.



$$C_{adj} = f_l \times f_d \times f_r \times (C_{unadj} - V_R) \quad (2-4)$$

Where

- $C_{adj}$  = adjusted roadway capacity
- $f_l$  = 1.00 for daytime or 0.96 nighttime with illumination
- $f_d$  = 1.00 peak hours-weekdays, 0.93 off-peak weekdays, or 0.84 weekends
- $f_r$  = 1.00, 0.95 light rain, 0.85 moderate-to-heavy rain, or 0.75 very heavy rain
- $C_{unadj}$  = unadjusted roadway capacity
- $V_R$  = ramp volume (veh/hr) used when a ramp is within 500 ft of work zone

## 2.3 Modeling the Roadway

According to the *Traffic Analysis Toolbox Volume III: Guidelines for Applying Traffic Microsimulation Modeling Software*, there are seven tasks needed for the development of microsimulation modeling (Dowling et al. 2004). The first two steps deal with (1) creating an organized plan for the model and (2) the collection and preparation of data. The next step is (3) the development of the base model that can be easily adjusted for the different test cases. Error checking (4) to check that there are no issues with how the roadway was drawn and vehicles successfully navigate the model and (5) calibration of the model is the next two steps that should be carried out. Successful calibration occurs when the output data from the model represents the collected data; this is discussed later. Once the model is calibrated the next step, (6) analysis of the alternative model can begin. Once the alternative model is simulated and the model data are compiled and organized, it is then (7) presented in a technical document as the final step (Dowling et al. 2004).

### 2.3.1 Guidelines for Calibration

Calibration is a crucial step in any model. Once the model has been developed in the chosen program and tested for any visible roadway errors, the modeler can start the calibration process. Each microsimulation program comes with a set of user-adjustable parameters that can be set according to the roadway collected data. In the following sections the different adjustable parameters in VISSIM will be discussed along with the relevant formulas that define each function.

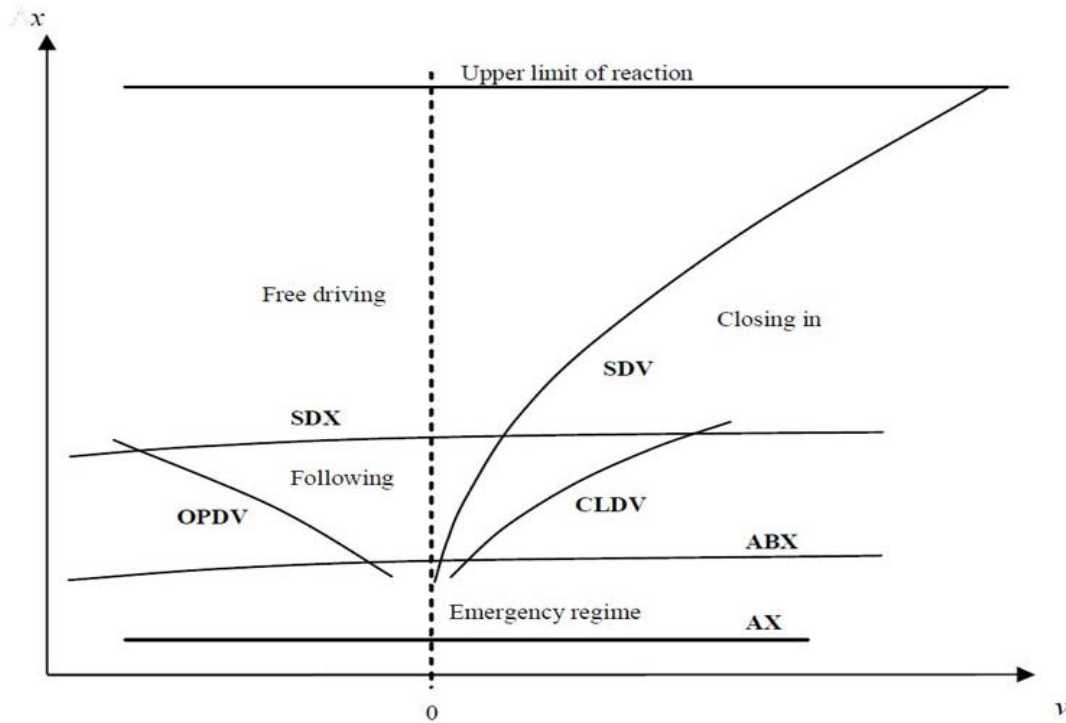
#### 2.3.1.1 Wiedemann Model

VISSIM is based on the Wiedemann “psycho-physical” car-following model and a lane-changing model, referred to as W-99. The term “psycho-physical” describes how the vehicles in a model behave. Each individual vehicle acts in a manner that simulates human driving behaviors, such as driving at different speeds, accelerating at different rates, and waiting for appropriate gaps to merge. In VISSIM the modeler also has the option to use the Wiedemann W-74 car-following model, though for freeways it is not used as it is not as detailed as the W-99 car-following model. A car-following model controls a driver’s behavior with respect to a vehicle in front of it in the same lane. A vehicle is classified as following when it is constrained by a preceding vehicle, and driving at its desired speed will result in a collision between the

vehicles. Once a vehicle is no longer constrained by a preceding vehicle it is considered to be free and travels at its desired speed (Olstam and Tapani 2004). The characteristics and behavior of individual vehicles (and drivers) affect performance measures such as speed, throughput, and queue length. One goal for the simulation modeler is to try to duplicate the field performance measures by using simulation (Chatterjee, et al. 2009). This is accomplished by adjusting two different parameters in VISSIM: (1) weaving behavior and (2) vehicle following behavior.

The Wiedmann model is based on six thresholds. The descriptions of these six thresholds from Olstam and Tapani are defined in Figure 2.1 (Olstam and Tapani 2004). These six thresholds are used in determining the following driving behaviors: following, free driving, closing in, and emergency action.

In the Wiedemann model the thresholds  $SDV$ ,  $SDX$ ,  $OPDV$  and  $ABX$ , seen in Figure 2-1, constitute the car-following model (Olstam and Tapani 2004). In order to account for variation in car throttle handling (i.e., different acceleration rates) the acceleration rate is assumed to be separate from zero at all times. When a vehicle passes either the  $SDV$  or  $ABH$  threshold it is assigned the acceleration rate  $-b_{null}$  and when passing the thresholds  $OPDV$  and  $SDX$  it is assigned the rate  $b_{null}$ .



AX: desired distance between fronts of two vehicles in a standing queue  
 ABX: Desired minimum following distance  
 SDV: Point where a driver consciously observes that he approaches a slower leading car  
 OPDV: Point where the following driver notices that he is slower than the leading car and accelerates  
 SDX: Perception threshold, maximum following distance  
 CLDV: Decreasing speed difference

**Figure 2-1 Psycho-physical car-following model by Wiedemann (Source: Olstam and Tapani 2004).**

When the vehicle is located above all thresholds in Figure 2.1 it is considered to be in free driving behavior and travels uninfluenced from all other traffic. The vehicle then uses maximum acceleration to reach its desired speed. When the desired speed is reached, inexact handling of the throttle is modeled by assigning an acceleration of  $-b_{null}$  or  $b_{null}$  to the vehicle.

When passing the *SDV* threshold, the vehicle driver notices that it is approaching a slower vehicle. The driver decelerates in order to avoid a collision. When the front to rear distance is smaller than *ABX* the following car can adopt, if necessary, the following emergency deceleration action to avoid collision with the vehicle in front (Olstam and Tapani 2004).

### 2.3.2 Lane Changing Behavior

The parameters of necessary lane changes dictate how far in advance each vehicle will be able to anticipate the off-ramp or lane drop on its current course and how aggressively that vehicle will change lanes to reach it. The two parameters look-back distance and emergency stop distance must be specified for each modeled lane closure or off ramp, in VISSIM, separately (Gomes et al. 2004). These are the three parameters associated with links: (1) Look-back distance, (2)

Emergency stop distance, and (3) Waiting time before diffusion, in VISSIM. These values can be adjusted in the link dialog box. *Look-back distance* is defined as the distance in anticipation of an event, such as a lane closure, that the driver will begin maneuvering toward the desired lane. *Emergency stop distance* is defined as the distance before the event where the driver will stop if he or she has not reached his or her desired lane. This would be located at the beginning of a taper area at a road construction site. *Waiting time before diffusion* is defined as a vehicle that has come to a halt at the emergency stop position will wait at most this amount of time for a gap to appear in the adjacent lane. After the waiting time has elapsed, the vehicle is removed from the simulation. If this happens in a model the waiting time will need to be increased so vehicles are not removed from the simulation.

### 2.3.3 VISSIM Car Following Behavior

Chatterjee et al. (2009) discussed the 10 variables that can be changed by the user in VISSIM to define vehicle following behavior. These values are located in the VISSIM software with short descriptions; the following section defines each of the variables in greater detail. The standstill distance, CC0 (ft), defines the desired distance between two consecutive vehicles at stopped condition; the default value, in VISSIM, is 4.94 ft. The desired time headway, CC1 (s), for the following vehicle, is used with CC0 to compute the safety distance:  $dx_{safe} = CC0 + CC1 * v$ , where  $v$  is the speed of the vehicle. The default value for CC1 is 0.9; higher CC1 values characterize less aggressive drivers.

CC2 (ft) defines the threshold that restricts longitudinal oscillation beyond safety distance in a following process; the default value is approximately 13 ft. CC3 characterizes the entry to the “following” mode of driving. It initiates the driver to decelerate upon recognizing a slower leading vehicle and defines the time at which the driver starts to decelerate before reaching the safety distance.

Speed oscillations after the vehicle enters the “following” mode of driving are controlled by CC4 and CC5. Smaller values of these two variables represent a more sensitive reaction of the driver to the acceleration or deceleration of the leading vehicle. CC4 is used for negative speed difference and CC5 is used for positive speed difference; the VISSIM guidelines state that these values should be of equal absolute value of opposite signs. The default values of CC4 and CC5 are  $-0.35$  and  $0.35$ , respectively. CC6 represents dependency of speed oscillation on distance in the “following” state. An increased value of CC6 results in an increase of speed oscillation as the distance to the preceding vehicle increases.

CC7 ( $\text{ft/s}^2$ ), CC8 ( $\text{ft/s}^2$ ) and CC9 ( $\text{ft/s}^2$ ) parameters control the acceleration process. CC7 controls the oscillation acceleration, or the variation in the repetition of acceleration. The CC8 variable represents the stand still acceleration. This value affects the vehicles that are stopped along the roadway and the rate at which they return to driving speed. CC9 is the acceleration at 50 mph.

This is the acceleration the vehicles use when they reach 50 mph but have not reached the desired speed.

#### **2.3.4 Parameter Adjustment in Model Calibration**

Studies have shown that the roadway capacities at work zones are not only lower than the capacities under normal operating conditions but also vary across all states in the US. This disparity means that a unique driving behavior parameter set cannot be used in all states; instead the parameter values should be chosen so as to reproduce the state-specific capacity values (Edara and Chatterjee 2010).

Correctly changing the CC-parameters is crucial to replicating work zones in VISSIM. At least four studies have investigated which of the parameters have the greatest impact on models. The research done by these four groups was to replicate work zone capacity values in a simulation model and use the W-99 driving behavior model that is included in the VISSIM software. Research by Lownes and Machemehl (2006) showed that the parameters CC0, CC1, CC2 and the lane-changing distance had a significant impact on the capacity values of models. In another study, Chitture and Benekohal (2008) showed that only parameters CC0 and CC1 had an impact on the simulated capacity values for freeways. The research by Chatterjee et al. (2009) changed the values of CC1 and CC2 in the models; this was derived from the works of Lownes and Machemehl, and Chitture and Benekohal. Their testing showed that CC0 had little impact on simulated capacity values. They also showed that the CC4/CC5 pair had unstable car-following behaviors when set greater than 3.0, and lower values had no statistically significant variation of capacity. Edara (2009) conducted research and developed a series of charts providing guidelines for changing CC1, CC2 and the safety reduction factor (SRF). These charts provide guidelines as to what these values should be based on the expected capacity and percentage of vehicles in the closed travel lane 1000 ft before the merging taper (Edara 2009).

Edara and Chatterjee (2010) developed a set of multivariate regression models to express the relationship between the driving behavior parameters, the CC values of the VISSIM model, work zone lane configuration, truck percentages, and work zone capacity. They developed equations to estimate the CC1, CC2 and SRF values for 2 to 1, 3 to 2 and 3 to 1 lane closure configurations. The SRF value refers to the reduction in safety distance to the trailing and leading vehicle on the desired lane and the safety distance to the leading vehicle in the current lane. Equations 2-5, 2-6 and 2-7 are the developed equations for the CC1, CC2 and SRF for a 2 to 1 lane configuration (Edara and Chatterjee 2010).

$$CC1 = 2.974 - 0.0009 \times Q_C + 0.0267 \times P_T + 0.0022 \times P_{CL} - 0.000029 \times Q_C \times P_T \quad (2-5)$$

Where: CC1 (s)  
 $Q_C$  = capacity (veh/hr/ln)  
 $P_T$  = truck percentage  
 $P_{CL}$  = lane distribution of the closed lane at 1000 ft upstream of the taper

$$CC2 = 82.39 - 0.0266 \times Q_C + 0.2087 \times P_T + 0.302 \times P_{CL} - 0.00009 \times Q_C \times P_T \quad (2-6)$$

Where: CC2 (ft)  
 $Q_C$  = capacity (veh/hr/ln)  
 $P_T$  = truck percentage  
 $P_{CL}$  = lane distribution of the closed lane at 1000 ft upstream of the taper

$$SRF = 00656 - 0.0002 \times Q_C + 0.0057 \times P_T + 0.0078 \times P_{CL} - 0.0000000009 \times Q_C \times P_T \quad (2-7)$$

Where: SRF  
 $Q_C$  = capacity (veh/hr/ln)  
 $P_T$  = truck percentage  
 $P_{CL}$  = lane distribution of the closed lane at 1000 ft upstream of the taper

These equations can be used to determine values for CC1, CC2 and SRF in VISSIM. These values are used to configure VISSIM during a lane closure and needed values collected during construction.

Through reviewing past research it was learned that no two roadways can be modeled by the same method. Each roadway needs to be examined during free flow conditions, no work zone, and during construction to create models that will replicate the capacities at an individual work zone. Another issue raised by Edara and Chatterjee (2010) is the use of the default truck characteristics in VISSIM. For example, the default length of a truck in VISSIM is 33.5 ft, while U.S. truck lengths can vary from 30 ft to as long as 80 ft. Two methods can be used to determine the truck type distribution, a detailed study at the work site or by using a Federal Highway Administration report from 2000 detailing the vehicle miles traveled on highways by the different classes of trucks (Edara and Chatterjee 2010).

## 2.4 Road User Cost

The main point of this research is to determine which bridge deck replacement method will have the lowest impact on the users of the system. To accomplish this, the road user cost (RUC) will be analyzed given the output from VISSIM. There are multiple ways to determine user cost. In 2010, a new version of the AASHTO “Red Book”, *User and Non-User Benefit Analysis for*

*Highways* was released (AASHTO 2010). Other methods of calculating RUC include works from individual researchers and some state DOTs have state specific methods of calculating RUCs.

To estimate user costs at a work zone, it is necessary to know the traffic flow rate, vehicle speed, and traffic capacity at the work zone, along with knowledge of vehicle occupancy and the value of time (Jiang 1999). The AASHTO Red Book uses only the vehicle speed (or travel time) and travel length to calculate a RUC. Jiang added more complex components to examine the cost due to the speed changes resulting from constant deceleration and acceleration in a work zone as well, as the effect of queues, and the overall speed reduction. Jiang’s method requires an in depth road study of before and during construction to acquire all necessary components.

In all methods used to determine the overall RUC, some values will need to be assumed for average wage/compensation and vehicle occupancy. Since the project location does not have known values for these inputs, the guidelines set in the AASHTO Red Book will be used. The Red Book states that trucks, defined as a heavy vehicle in the transport of goods and materials or in the delivery of other services other than public transport, shall receive 100% of total compensation for assigning a value of time. Contrastingly, personal vehicles shall be compensated 50% of the wage rate for determining value of time. The value assigned to each vehicle class is \$20.23 per hour for trucks and \$18.56 per hour for personal vehicles. For occupancy rates, the AASHTO Red Book recommends using 1.5 persons per vehicle for personal vehicles and 1.05 persons per vehicle for trucks (AASHTO 2010).

Equation 2-8 is used for developing the value of time (VOT). This value will be used in later equations to obtain a RUC value.

$$Value\ of\ Time\ per\ Hour\ \left(\frac{\$}{veh\ hr}\right) = Hourly\ Wage\ Rate \times Occupancy \times Percentage \tag{2-8}$$

Where:            Hourly Wage Rate = average hourly wage rate (\$)  
                       Occupancy = average occupancy per vehicle type  
                       Percentage = percentage of hourly wage rate compensation (%)

Once the VOT is obtained, the operation and ownership costs need to be calculated. Operating costs include fuel, oil, tires, and maintenance. Ownership cost includes such things as insurance, license and registration fees and taxes, vehicle depreciation and finance charges. The AASHTO Red Book contains some pre-defined values for many of these variables, as well as equations to manually calculate some of the values. Equation 2-9 is used to calculate cost of fuel, in cents per vehicle mile; Equation 2-10 is the calculation of change in fuel cost based on travel time. (AASHTO 2010)

$$C_{fuel} = 100P_{fuel}/E_{mpg} \quad (2-9)$$

Where:  $C_{fuel}$  = user cost of fuel, (\$/vehicle-mile)  
 $P_{fuel}$  = fuel price, (\$/gallon)  
 $E_{mpg}$  = fuel efficiency, (miles/gallon)

$$\Delta C(D)_{c,fuel} = (gal_{c,min})(D_0 - D_1)P_c \quad (2-10)$$

Where:  $\Delta C(D)_{c,fuel}$  = change in fuel cost as a function of delay (\$)  
 $gal_{c,min}$  = gallons per minute for vehicle class c (found in AASHTO Red Book)  
 $D_0$  = average delay before (minutes)  
 $D_1$  = average delay after (minutes)  
 $P_c$  = fuel price per gallon for vehicle class c (\$/gal)

The second set of costs needed to be looked at is the ownership costs; these are the costs associated with owning a vehicle. There are two approaches that are used in making these calculations depending upon the type of ownership cost. First there are the annual or periodic costs such as insurance payments and tag/title taxes. For these payments it is best to divide the annual cost by vehicle mile traveled (VMT) annually. Though these costs are not typically incurred on a per-mile basis, such an approach renders accurately the impact on total ownership costs in these categories. For costs that are incurred once in the lifetime of a vehicle (purchase price) it is recommended to use an amortized approach for calculating a periodic value then turning it into a per-vehicle mile cost. Equation 2-11 is the approach to determine the annual amortized value of a vehicle (AASHTO 2010).

$$PMT = P_{veh} \frac{r[(1+r)^L + SV]}{(1+r)^L - 1} \quad (2-11)$$

Where:  $PMT$  = annual amortized value (\$)  
 $P_{veh}$  = capital value (price) of the vehicle (\$)  
 $r$  = interest rate (per year)  
 $L$  = expected life of the vehicle (in years)  
 $SV$  = salvage value (\$)

Equation 2-12 converts the acquired PMT from Equation 4 to  $PMT_{min}$ , a value that represents the overall PMT in terms of cents per minute of delay. To calculate change in  $PMT_{min}$  for a section of delay, multiply the  $PMT_{min}$  by the change in time, in minutes, caused by delay. (AASHTO 2010)



$$PMT_{min} = \frac{(PMT \times 100)}{(365 \times 24 \times 60)} \quad (2-12)$$

Where:  $PMT_{min}$  = \$ per minute of delay  
 $PMT$  = annual amortized value (\$)

Inventory costs of cargo need to be applied to trucks that are delayed in the construction zone as well. To calculate the inventory costs on a vehicle per-mile basis, an hourly interest rate must be computed along with the amount of time it takes the vehicle to travel a mile. Equation 2-13 is used to calculate inventory costs, per vehicle-mile, based on speed. To calculate for a change of speed in Equation 2-13, change  $1/s$  to  $(1/s_0 - 1/S_1)$ .

$$I(S) = 100 \times \frac{r}{8760} \times \frac{1}{S} \times P_{cargo} \quad (2-13)$$

Where:  $I(S)$  = inventory costs (\$ per vehicle-mile) as a function of speed  
 $r$  = interest rate, per annum  
 $P_{cargo}$  = value of the cargo (\$)  
 $S$  = speed of the vehicle (mph)

Equation 2-14 represents this and to calculate the change in inventory cost based on delay, multiply the calculated value by the change in time.

$$I(D) = 100 \times \frac{r}{8760 \times 60} \times P_{cargo} \quad (2-14)$$

Where:  $I(D)$  = inventory costs per vehicle-minute of delay (\$ per minute)  
 $r$  = interest rate, per annum  
 $P_{cargo}$  = value of cargo (\$)

The preceding equations are used together to calculate an overall RUC for a particular roadway using the three calculated components: value of time, ownership costs and operating costs. Once the VOT, ownership and operating costs are calculated and they are in the same units they can be added together to acquire the total RUC, as seen in Equation 2.8.

## 2.5 Lessons Learned

Through the review of the literature done for this study, methods for creating reasonably accurate simulation models have been learned. The emphasis on model calibration throughout all literature indicates that this step in modeling is crucial for creating a successful model. Without an accurate model the model output data will not be accurate. The development of a RUC value is dependent on the accuracy of the model's output.

Using what was learned from the literature will allow for the creation of accurate simulation models of the project site. The output from these models will then be applied to the development of RUC values and provide a method of analyzing the different bridge deck replacement methods.

## **Chapter 3: Methodology**

### **3.1 Introduction**

This project is focused on determining the user cost for different bridge deck replacement methods for the Alabama Department of Transportation (ALDOT). To accomplish this task, the microsimulation software VISSIM was used to simulate the section of roadway where the construction will take place. Individual vehicle speed, vehicle length, volume, and time of day were collected on I-59 in Collinsville, AL. The collected data were used to construct and calibrate the base model so that any alternative would be reasonably accurate.

This chapter is divided into the following five sections. Section 3.2 presents the data collection methods and collected data traffic statistics, along with the use of these data within VISSIM. Section 3.3 details the method of creating the base model in VISSIM and the steps used to ensure its correctness. Section 3.4 contains the methods of creating the different model scenarios used in this research, including: lane bias models, alternative lane configuration models, and the variable volume models. Section 3.5 presents the method used to calculate the road user cost (RUC) from the output data from VISSIM, and Section 3.6 is a summary of the chapter.

### 3.2 Data Collection and Preparation

The data were collected by using three remote traffic microwave sensor (RTMS) G4 units, which are non-intrusive microwave radar units set up using roadside trailers. Data for each direction of traffic were collected for one week. The northbound (NB) data were collected from Friday March 18, 2011 to Friday March 25, 2011, and the southbound (SB) data were collected from March 25, 2011 to Friday April 1, 2011. The collection units were installed in the northbound direction approximately at mileposts 202.1 NB, 204.4 NB, and 204.6 NB while the southbound collectors were installed approximately at mileposts 207.0 SB, 205.8 SB, and 205.3 SB on I-59. These locations are the approximate positions of critical locations of the expected temporary traffic control plan: the advance warning area, beginning of taper and the beginning of the buffer zone. The approximate locations of these devices are shown in Figure 3.1, and Table 3.1 summarizes the collector locations, while Figure 3.2 shows the RTMS unit locations with respect to the activity area of the bridge deck replacement project.

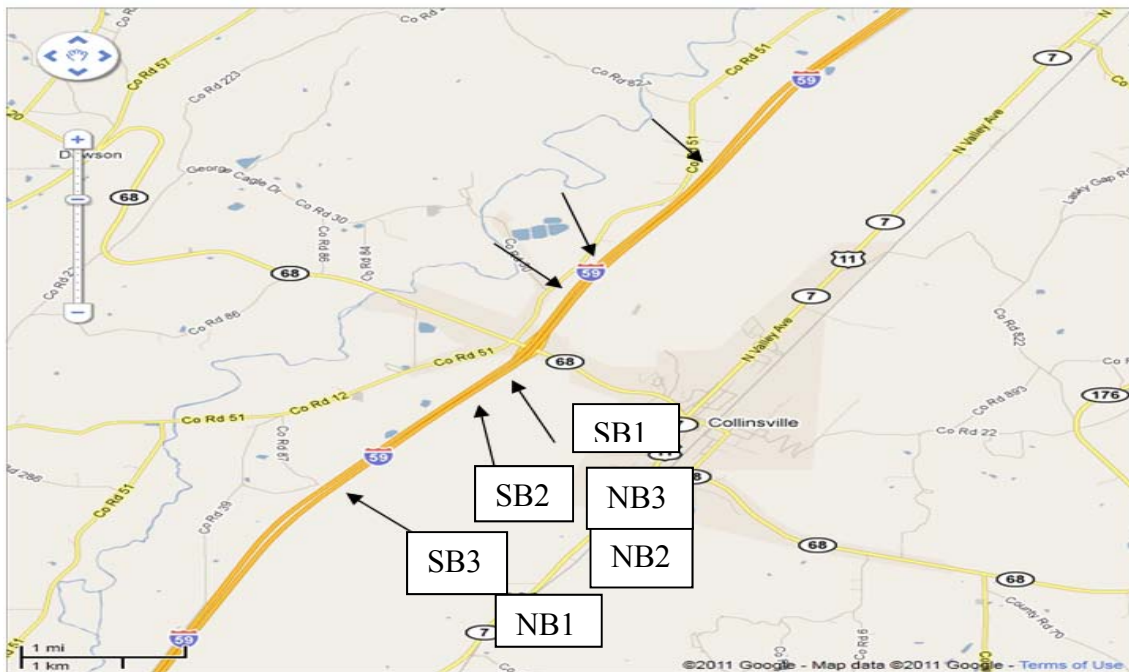
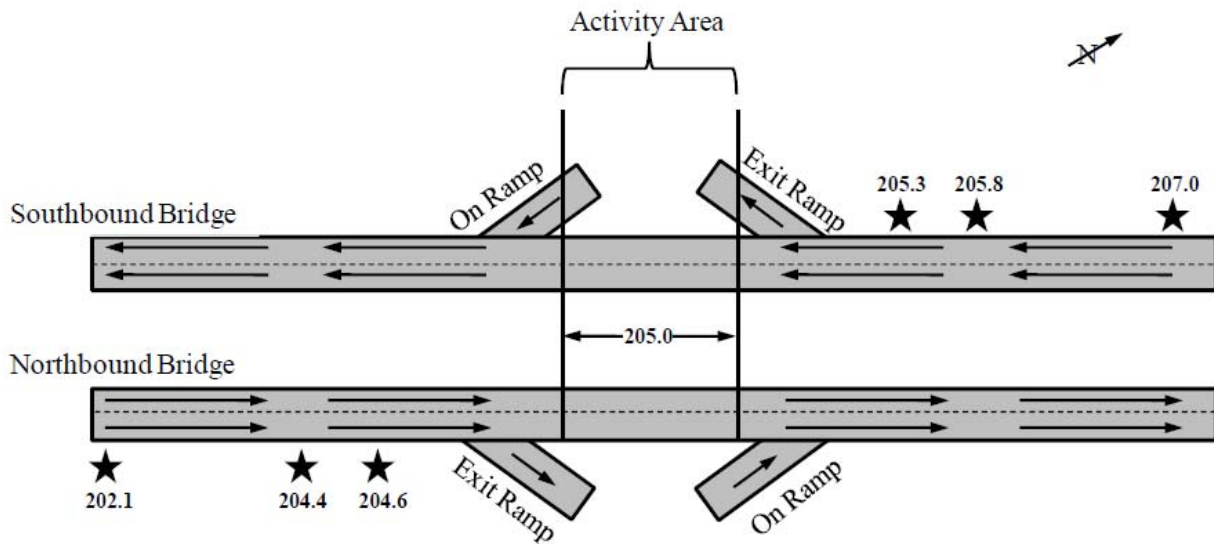


Figure 3-1 RTMS Unit Locations.



\*Not to scale

**Figure 3-2 RTMS Unit and Activity Area Locations.**

**Table 3.1 Collector Names and Locations**

Collector Name	Collector Location (Milepost)
NB1	202.1 NB
NB2	204.4 NB
NB3	204.6 NB
SB1	207.0 SB
SB2	205.8 SB
SB3	205.3 SB

The data were delivered as text files in which observations were ordered by time. The data were imported into Microsoft Excel and sorted based on day of the week using pivot tables so they could be modeled. The RTMS units collected multiple data variables during the collection period including: travel time, speed, vehicle length, and other values not relevant to this study. The units collected data from Friday afternoon to Friday afternoon the following week, the first and last full days of collection were to define the time period for the study (Saturday to Thursday). The posted speed limit at the test site is 70 mph. A summary of the weekly data from collectors NB1 and SB1 is presented in Table 3.2. The RTMS units identified classification by vehicle length. A passenger vehicle is any vehicle that was less than 30 ft and truck is any vehicle 30 ft or greater; recreational vehicles were not accounted for in this research. These values are defined in the AASHTO publication “A Policy on Geometric Design of Highways and Streets” (AASHTO 2004). Peak Hour Factor is a measure of variability of demand during the peak hour. It is the ratio of the of volume during the peak hour to the maximum rate of flow during a 15 minute time period within the peak hour.

**Table 3.2 Weekly Traffic Analysis Data for collectors NB1 and SB1**

	Collector NB1	Collector SB1
Average Speed (mph)	71.8	75.1
Standard Deviation (mph)	5.1	9.2
85 <sup>th</sup> percentile speed (mph)	77	85
15 <sup>th</sup> percentile speed (mph)	67	68
Pace Speed (mph)	69 to 78 mph	68 to 77 mph
Peak Hour	Sunday 1:30PM to 2:30PM	Thursday 3:45PM to 4:45PM
Peak Hour Volume (veh)	485	525
Peak Hour Factor	0.86	0.93
Heavy Vehicle Percentage	25	19

### **3.3 Model Development and Error Checking**

VISSIM allows users to use background images to have a precise drawing of the roadway system being modeled; the modeling for this project started with a map (courtesy of Google Maps) of the project area, Collinsville, AL, being imported into VISSIM. The image was scaled from within the program and the interstate was drawn according to the map. In VISSIM, roadways are comprised of links and link connectors. Links in VISSIM are used to model stretches of uninterrupted roadway. Link connectors in VISSIM connect two or more links and can be used to model different lane closure configurations. To model this roadway two links were used and one connector in each direction of travel. The initial link starts two miles before the merging taper begins, and the second link consists of one mile of roadway after the activity area. The links are connected by a link connector to form a model of 17,840 ft.

For the initial development and testing of the model, the traffic volumes used were based on data obtained from the ALDOT website (ALDOT 2010). These volumes allowed initial viewing of the model before data collection was complete. The projected speed in VISSIM was assigned the range 65 mph to 75 mph; this provides a 10 mph range around the posted speed limit of 70 mph on this section of roadway, with median speed at 70 mph. The temporary traffic control plan was not finalized prior to development of this model; therefore, the critical locations in the plan were set up in the model based on a typical temporary traffic control plan obtained from ALDOT and guidelines found in the Manual on Uniform Traffic Control Devices (MUTCD). The ALDOT typical plan calls for two speed changes prior to the construction site: one placed 1820 ft from the taper and another 500 ft before the taper. The speeds assigned to the model were 60 mph at the first reduction and 50 mph at the second reduction. This configuration is displayed in Figure 3.3. Based on Figure 6C-4 of the MUTCD, the taper length was assigned 840 ft, a function of posted speed and offset width, and the buffer zone assigned a length of 730 ft, based on stopping sight distance at 70 mph (FHWA 2009). A length of 430 ft was assigned to the work area, 226 ft is the length of the bridge, and the rest allows for equipment and material storage. This gives a

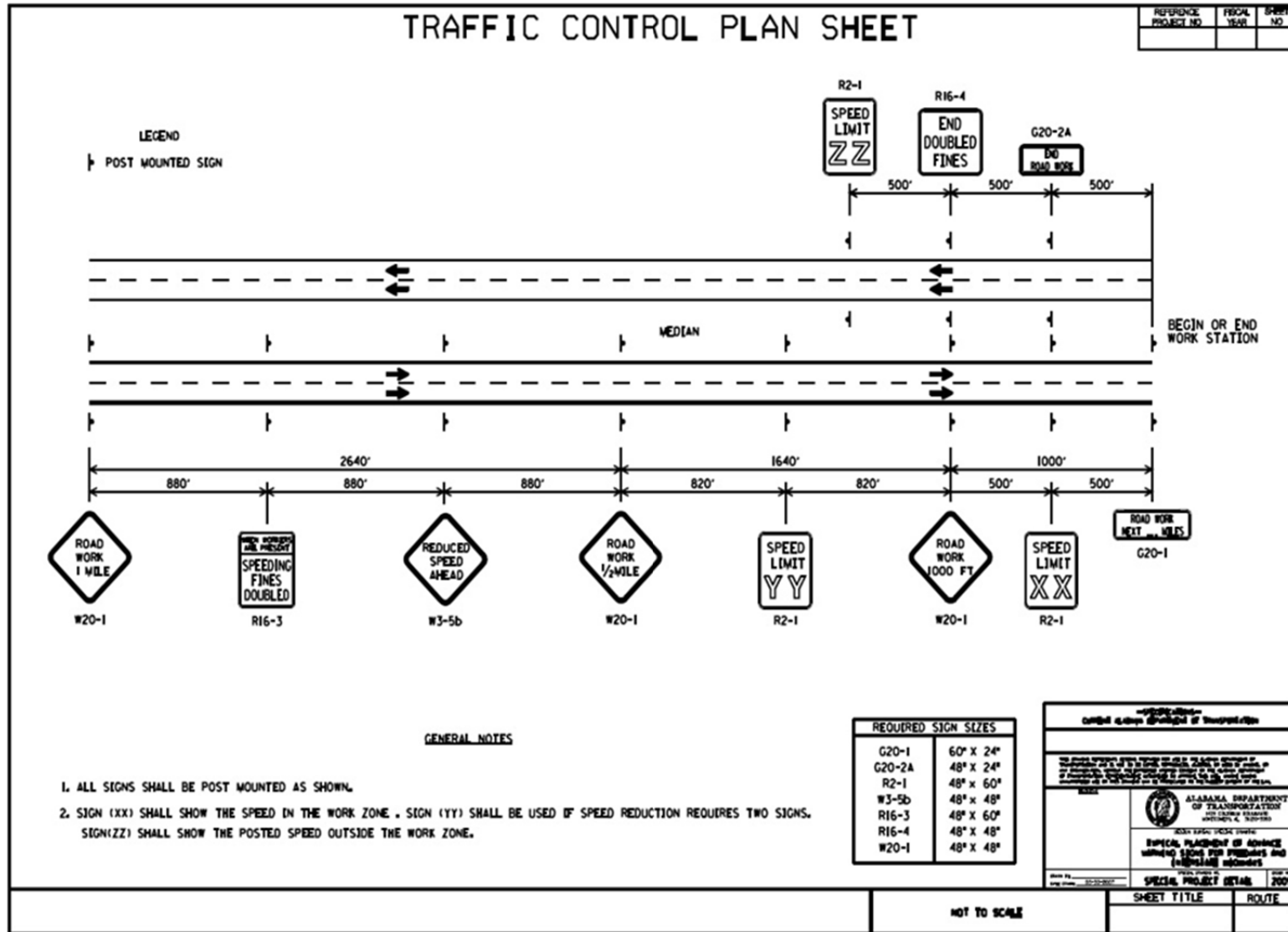
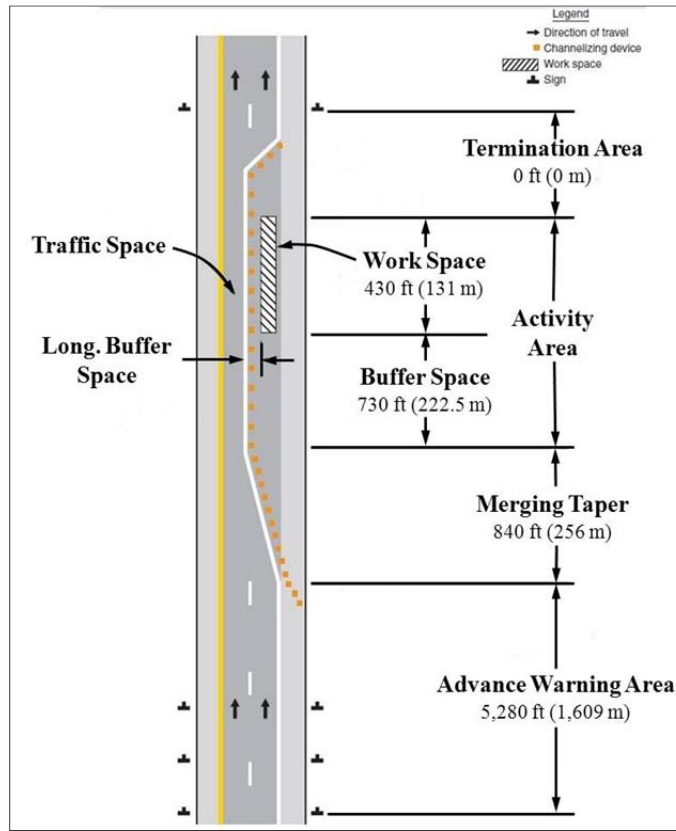


Figure 3-3 Traffic Control Plan Sheet (Source: ALDOT, Taunton 2010).

total length of 2000 ft from beginning of the taper to the end of the activity area. This layout can be seen in Figure 3.4.



**Figure 3-4 Lane Closure Modeled in VISSIM.**

### 3.3.1 Peak Day Transportation Statistics and Vehicle Composition

The peak day data was used to model each direction of travel. Northbound traffic had its peak day on Saturday whereas the southbound peak day was a Thursday. Both directions of travel had peak hour volumes less than 550 vehicles per hour, as shown in Table 3.3. Both directions of travel had similar traffic statistics, with the southbound direction having an average speed 2.4 mph less than the northbound average speed. Table 3.3 shows the breakdown of the daily traffic statistics for the northbound and southbound travel lanes, these values were used to create the models.

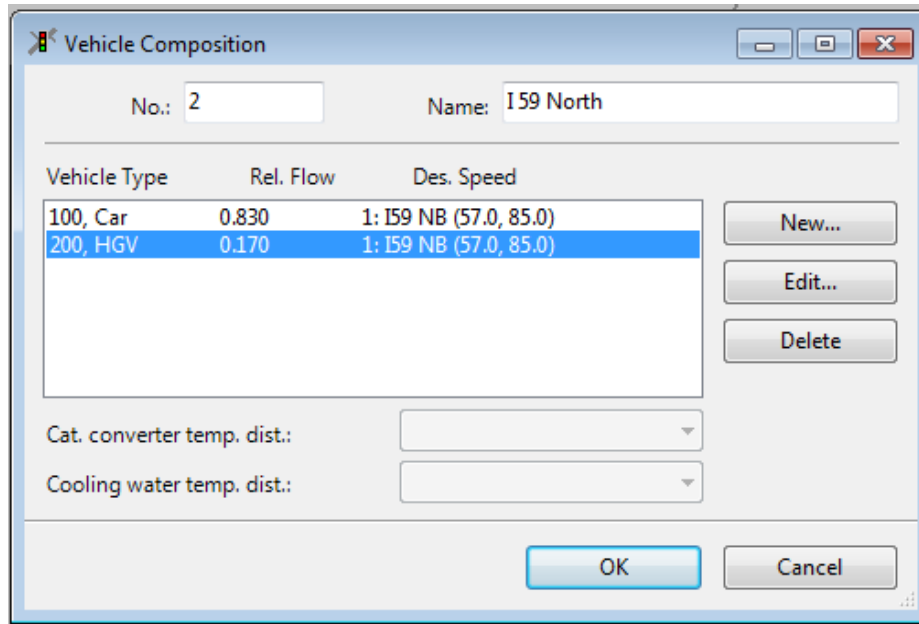


**Table 3.3 Peak Day Traffic Statistics Data for collectors NB1 and SB1**

	Northbound	Southbound
Day	Saturday (03/19/11)	Thursday (03/31/11)
Average Speed (mph)	73.3	70.9
Standard Deviation (mph)	4.4	4.0
85 <sup>th</sup> percentile speed (mph)	77	76
15 <sup>th</sup> percentile speed (mph)	66	67
Pace Speed (mph)	69-78	68-77
Peak Hour	13:30-14:30	15:45-16:45
Peak Hour Volume	474	525
Peak Hour Factor	0.95	0.93
Heavy Vehicle Percentage	17	23

Three aspects of the traffic data were used to create the models: vehicle composition, speed distribution, and hourly volumes. In VISSIM each of these three variables has a unique graphical user interface (GUI) to input the necessary values. Initially there are default values for all of these variables in VISSIM.

The default vehicle composition in VISSIM is 2% heavy vehicles and 98% passenger cars. This was changed by going into the traffic menu in VISSIM and selecting Vehicle Composition. Figure 3.5 represents the northbound vehicle composition GUI in VISSIM. The vehicle composition consists of two vehicle types in VISSIM; type 100, cars and type 200, heavy goods vehicles (HGV). HGV is any truck with a gross combination mass over 7,716 lbs. These are assigned the respective traffic composition values. The southbound model was set up in the same manner with a different assigned name and relevant traffic compositions.



**Figure 3-5 NB1 Vehicle Composition in VISSIM.**

### 3.3.2 Speed Distributions

Speed distributions were then established to provide the modeled roadway the same distribution that was observed in the study area. The daily traffic was grouped by speed in Microsoft Excel and the percentages were calculated in 5 mph increments. VISSIM has a lower bound of 0.01 or 1% for the speed distribution. Because of this it was necessary to combine the first speed groups to have a value greater than 1%. Tables 3.4 and 3.5 show the northbound and southbound calculated speed distributions used in VISSIM.

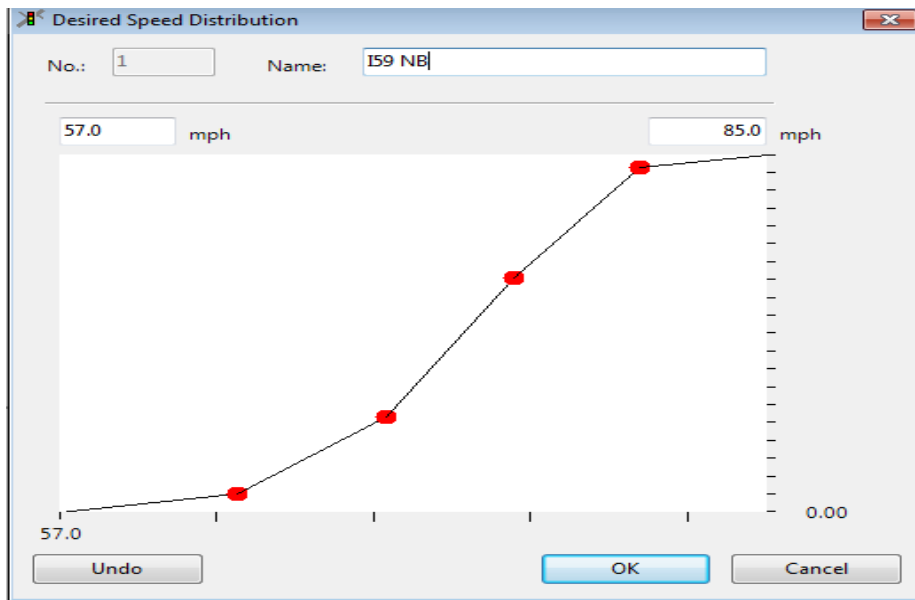
**Table 3.4 NB1 Speed Distributions**

Speed (mph)	Percentage (%)
57-65	4.81
66-70	22.80
71-75	38.07
77-80	32.16
81-85	2.16

**Table 3.5 SB1 Speed**

Speed (mph)	Percentage (%)
59-65	11.12
66-70	35.24
71-75	43.2
76-80	9.75
81-83	0.69

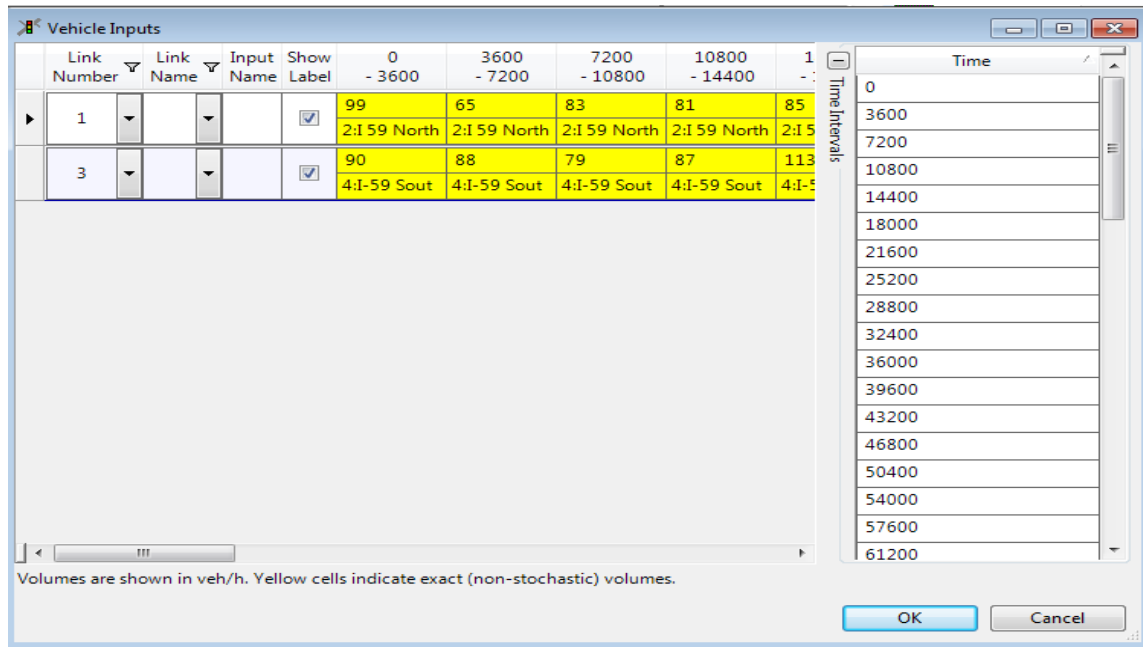
The speed distributions are also input in VISSIM in a GUI. The Desired Speed Distribution for VISSIM is found in the Base Data menu under Desired Speed. To enter the speed distribution the upper and lower bound of the speed range must be input. The distribution is inserted by selecting a location along the speed curve and moving the resulting red marker to the correct speed value and ratio. As an example, the final product for the northbound distribution can be seen in Figure 3.6.



**Figure 3-6 NB1 Speed Distribution in VISSIM.**

### 3.3.3 Hourly Volumes

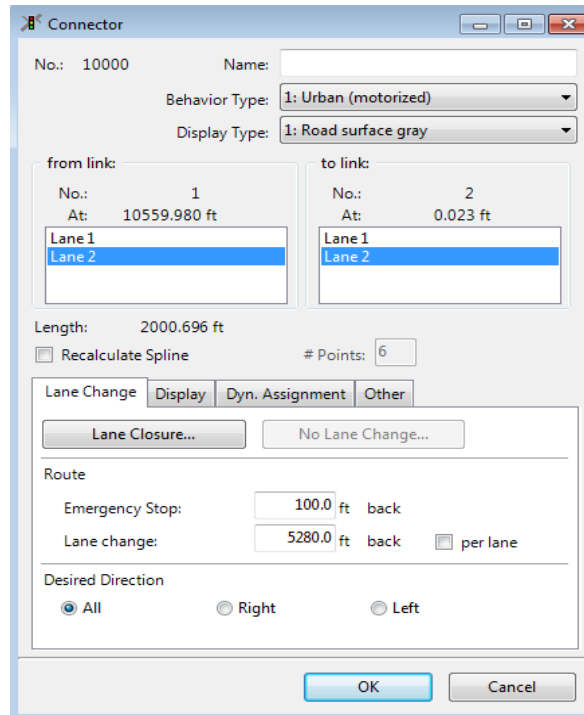
To enter the hourly volumes collected into VISSIM the corresponding hours were converted into seconds, so 12:00AM represents the period 0-3600 seconds. This is due to VISSIM using seconds as the unit of time for the volume distributions. By default VISSIM has a single hour for vehicle input, so all additional values were keyed into the software. Each volume entry represents the number of vehicles that will enter the model during that given time, so in this model in the first hour (0–3600 seconds) there will be 99 vehicles entering the northbound direction of travel and 90 entering the southbound, as seen in Figure 3.6. These values are taken from the collected traffic data at the project site. VISSIM also has the ability to use stochastic values for vehicle volumes, this is used when the modeler wants to model random vehicle volumes in the model, for the models in this project the values are static. This and the GUI for inserting volume data into VISSIM can be seen in Figure 3.7.



**Figure 3-7 NB1 Vehicle Inputs in VISSIM.**

After the traffic inputs were defined, the data collection component of VISSIM was placed into the models. A simple travel time collection system was set up on both the northbound and southbound models. The start of the travel time collection was placed where the first advance warning traffic sign would be placed, 1 mile before the beginning of the taper and ended 0.5 miles after the end of the construction zone; this provides a total of 9,920 ft for travel time collection. Additional data collection points were used to determine spot speeds, which were used to calibrate the model.

To change the free flow model into the alternative models (discussed in depth in following sections), the required temporary traffic control devices were added. The speed changes were located in accordance to the acquired traffic control plan obtained from ALDOT; the first speed reduction was set at 60 mph and the second to 50 mph, each speed had a distribution of  $\pm 5$  mph with the posted speed as the 85<sup>th</sup> percentile speed. Figure 3.8 shows the connector GUI for connectors in VISSIM. The GUI shows information about the connector such as: connections to VISSIM links, length of connector, lane closure configuration, emergency stop and lane change values.



**Figure 3-8 Connector screen in VISSIM.**

Each lane can be closed in the Lane Closure dialog box, and emergency stop and lane change values can be assigned. These two values only work when there is a defined travel route on the system for vehicles to follow. Travel routes are defined by the modeler as paths vehicles will travel on the roadway. The lane change value is also dependent on the length of the defined route, so a short route before the lane closure can lead to cars merging while inside the construction zone, vehicles would enter the buffer zone and then merge. Alternatively, a single lane can be left unconnected along the connector to achieve the same results, while using a defined travel route.

### **3.4 Models Used for Analysis**

Models used in this section are based on the previously constructed model in Section 3.2. This provides consistency throughout all models, so that the locations and length of all models are equal. All models were simulated using VISSIM's Multirun function, with the following parameters: (1) starting random seed 5, (2) random seed increment 15, and (3) number of simulations 15. The random seed affects when vehicles enter the model and at what speed, within the defined parameters. The random seed increment raises the random seed value by that amount. Simulating each model 15 times provides 99% confidence that the output from VISSIM will be within 2.0 units of output (in this case the output units is seconds) of actual values (Dowling et al. 2004). For example, if the model output had an average travel time of 100 seconds, then the expected travel time at the site would be in the range of 98 to 102 seconds, with 99% confidence that these values are accurate from the model.

Documentation provided by PTV America, the developers of VISSIM, indicated that models should be simulated longer than the defined vehicle input times. This ensures that all vehicles make it through the model. The model run time is found in the simulation parameters menu. All hour-long models were set to run for 5,400 simulation seconds and the 24 hour models were run for 87,000 simulation seconds, providing adequate time for vehicles to clear the model. These values were decided upon after simulating models multiple times to find the necessary run time length for all vehicles to clear the model under free flow conditions (VISSIM Online Help).

### **3.4.1 Lane Bias Modeling**

An initial test was done to determine if the output from VISSIM changed based on which lane was closed (left or right). The models are based on the northbound lane geometry and the data collected on the 19<sup>th</sup> of March 2011 (Saturday). The models represent a single hour of travel time with volume levels of 1,000 veh/hr and 2,200 veh/hr. These values were selected based on expected capacity. At 1,000 veh/hr the traffic volume has doubled the peak hour volume at the test site but has not reached the point where congestion is expected, while 2,200 veh/hr is nearing capacity for a one lane roadway. By comparing the two sets of data it can be determined if VISSIM has a lane bias and if models representing right and left lane closures will be needed. The results from this set of models heavily influenced the rest of the models. If there was shown to be a lane bias then models with a closed lane would have to be modeled with a right and left lane closure configuration. The results from these models can be found in *Chapter 4: Data Analysis*.

### **3.4.2 Different Lane Configuration Modeling**

This model set was created to examine the different lane configurations from the different bridge deck replacement methods. Three models were used to simulate the northbound and southbound direction of travel for the following conditions: Free Flow “free flow scenario”, Closed Lane with TTCO “closed lane scenario” and Free Flow with Reduced Speeds “reduced speed scenario”. To replace a single section of bridge decking using a cast-in-place (CIP) rapid replacement method, after the traffic control devices are in place, it would take between 16.5 and 18.5 hours of total construction hours to complete. With the pre-cast rapid bridge deck replacement method the total construction time would take between 9.5 and 12 hours of work. These time values do not take into account any work stoppage, such as shifts ending, so it was decided to simulate each model for a 24 hour period of time. Table 3.6 shows the hourly volumes collected at the test site and used in these models.

**Table 3.6 Hourly Volumes Used to Model Northbound and Southbound Travel Lanes**

Hour	Northbound (veh/hr)	Southbound (veh/hr)
1:00:00	99	90
2:00:00	65	88
3:00:00	83	79
4:00:00	81	87
5:00:00	85	113
6:00:00	98	176
7:00:00	148	240
8:00:00	225	331
9:00:00	327	383
10:00:00	379	402
11:00:00	397	367
12:00:00	421	420
13:00:00	407	359
14:00:00	443	370
15:00:00	456	459
16:00:00	462	485
17:00:00	415	513
18:00:00	396	477
19:00:00	374	308
20:00:00	340	256
21:00:00	282	196
22:00:00	236	173
23:00:00	196	148
24:00:00	117	115

The free flow model represents the collected data at the test site in Collinsville, AL. The closed lane with TTCO represents a traditional construction site and the free flow with reduced speed is a model representing the delay when both lanes are open to traffic but at a reduced speed. Each of these models was modeled for a 24 hour period of time, based on the data collected from the test site.

### **3.4.3 Variable Volume Modeling**

To examine a wide range of possible traffic volumes the northbound lane was used to model different volumes for a single hour. The model is first simulation at 500 veh/hr for an hour incrementing by 100 veh/hr until 2200 veh/hr is reached. The volume range was selected based on of the peak hour at the test site and a 2400 pc/hr/ln capacity on freeways as suggested in the *Highway Capacity Manual* (HCM 2000). Any volume less than 500 veh/hr would not provide

any form of congestion so they were not modeled; with volumes above 2200 veh/hr congestion would occur.

Each of the three model types: free flow, closed lane, and reduced speed are used for variable volume modeling. This provides a comparison between each of the models according to the modeled volume. It is expected that at some hourly volume the difference between the two different bridge deck replacement methods will become significant and allow decision makers to choose one method over the other.

### 3.5 Road User Cost Analysis

The travel time values acquired from the different models are applied using the guidelines in the AASHTO publication “User and Non-User Benefit Analysis for Highways”, also called the “Red Book.” This provides a RUC for both the passenger cars and trucks in the model. These methods will be applied to the three 24 hour models and the variable volume models. The data acquired will complement the travel time data and give a monetary value to the different models in terms of RUC.

To develop the RUC for each of the models, the values found in *Chapter 5: User Benefit Analysis Models* in the “Red Book” were used. Table 3.7 is a list of the values used in developing the RUC calculations. As mentioned in *Chapter 2: Literature Review* of this report, these values are needed to calculate RUC: Value of Time (VOT), ownership cost and operating cost.

**Table 3.7 Assumed Values for RUC Calculation from “Red Book”**

	Passenger Vehicle	Truck
Finance Rate	0.10	0.10
Percentage of Hourly Wage	50%	100%
Average Hourly Wage(\$)	18.56	20.23
Average Vehicle Occupancy	1.3	1.05
Other Operating Cost per Mile(\$/mile)	0.04	0.05
Vehicle Life (years)	10	8
Vehicle Cost (\$)	20,000	60,000
Salvage Value at End of Life (\$)	2,000	5,000
Insurance per Year (\$)	1,000	1,500
Cargo Value(\$)		200,000
Miles per Year	15,000	50,000

The VOT is calculated by multiplying the percentage of hourly wage, average hourly wage, and travel time in hours. Calculating the ownership cost takes into account the value of the vehicle as well as costs associated with insurance. Additionally the cargo value is taken into account when



doing this calculation for trucks. Using the values from Table 3.6 the finance rate, vehicle cost, vehicle life, and salvage value are used to calculate the amortized vehicle cost per hour using Equation 3-1. To calculate the amortized insurance cost per hour the insurance cost is divided by 8760 (to convert from years to hours); the cargo cost per hour is calculated by multiplying the cargo cost by the finance rate then dividing by 8760.

$$A = \frac{i * \frac{V * (1 + i)^n - F}{(1 + i)^n - 1}}{8760} \quad (3-1)$$

Where:      A = amortized vehicle cost per hour  
               i = finance rate  
               V = vehicle cost (\$)  
               n = vehicle life (years)  
               F = Salvage Value (\$)

The three components of the ownership cost are added together to obtain a total ownership cost per hour value, as seen in Section 2.3. As with obtaining the VOT portion of the RUC, the travel time obtained from VISSIM is converted into hours and multiplied with the total ownership cost to obtain the cost for this project.

The final portion of calculating a RUC is determining the operating cost, which includes values associated with driving a vehicle such as: fuel, oil and tires. The “Red Book” contains reference charts with values used for calculating operating costs. The fuel costs found in the “Red Book” are based on fuel prices from 2000, so new fuel price components need to be calculated for this project. The fuel prices used are based on the U.S. national average as of August 16, 2011, the cost of a gallon of gasoline was \$3.604 per gallon and \$3.835 per gallon for diesel fuel (US EIA, 2011).

To calculate the amount of used fuel based on the travel time, Table 5-6 in the “Red Book” entitled “Fuel Consumption (Gallons) per Minute of Delay by Vehicle Type” is used. This can be seen as Table 3.8.

**Table 3.8 Fuel Consumption (Gallons) per Minute of Delay by Vehicle Type  
(Source: AASHTO 2010)**

Free Flow Speed (mph)	Small Car	Big Car	SUV	2-Axle SU	3-Axle SU	Combo
20	0.011	0.022	0.023	0.074	0.102	0.198
25	0.013	0.026	0.027	0.097	0.133	0.242
30	0.015	0.03	0.032	0.122	0.167	0.284
35	0.018	0.034	0.037	0.149	0.203	0.327
40	0.021	0.038	0.043	0.177	0.241	0.369
45	0.025	0.043	0.049	0.206	0.28	0.411
50	0.028	0.048	0.057	0.235	0.321	0.453
55	0.032	0.054	0.065	0.266	0.362	0.495
60	0.037	0.06	0.073	0.297	0.404	0.537
65	0.042	0.066	0.083	0.328	0.447	0.578
70	0.047	0.073	0.094	0.36	0.49	0.62
75	0.053	0.08	0.105	0.392	0.534	0.661

This table is divided into multiple vehicle classifications. Without a full analysis of vehicle types at the construction site, it was decided to use the values for “small car” and “combo” vehicles, the values for the “combo” vehicles were applied to the truck category while “small car” used for all passenger cars. The chart uses travel speeds to determine the fuel consumption rate. The chart has speeds in increments of 5 mph starting at 20 mph and ending at 75 mph. To obtain the fuel consumption rate for the models, interpolation was done to obtain the corresponding value.

The fuel consumption rate, found through interpolation, was multiplied by the cost of fuel and the travel time (in minutes) for passenger cars and heavy vehicles, as seen in Equation 3.2. This provides one component of the operating cost.

$$Cost\ of\ Fuel\ Component = Fuel\ Consumption\ Rate \times Travel\ Time \times Fuel\ Cost \quad (3-2)$$

Where:            Fuel Consumption Rate = rate of fuel consumption of a vehicle  
                       Travel Time = travel time (s)  
                       Fuel Cost = cost of fuel (\$)

The other component of operating cost comes directly from the “Red Book”. This component of the operating cost represents other aspects of operating a vehicle, such as: oil, tires, and maintenance. Passenger cars incur a \$0.04 cost per mile while trucks have a cost \$0.05 per mile. Multiplying these values by the length of travel provides the rest of the operating cost for the

project area. This obtained value is added to the fuel component to obtain the total operating cost.

The RUC average for passenger cars and trucks is obtained by adding the VOT, ownership cost and operating cost for their respective vehicle class. These values can then be multiplied by the number of vehicles in each class to obtain the RUC for each vehicle type, passenger cars and trucks, as seen in Equations 3-3 and 3-4. Adding the results from these two equations will provide the overall RUC.

$$\begin{aligned} \text{Total Passenger Car RUC} &= \\ \text{Average Passenger Car RUC} &\times \text{Number of Passenger Cars} \end{aligned} \tag{3-3}$$

Where:       Average Passenger Car RUC = average RUC (\$)  
              Number of Passenger Cars = number of vehicles classified as passenger cars

$$\begin{aligned} \text{Total Truck RUC} &= \text{Average Truck RUC} \times \text{Number of Trucks} \end{aligned} \tag{3-4}$$

Where:       Average Truck RUC = average RUC (\$)  
              Number of Trucks = number of vehicles classified as trucks

### **3.6 Summary of Methodology**

The models created in this chapter will be used to simulate the different situations for the different bridge deck replacement methods. The data they produce will be discussed in *Chapter 4: Data Analysis*. The analysis of these data will be used to show which method of bridge deck replacement is the best choice based on travel time and RUC. These results will aid any decision maker as to which bridge deck replacement method will be best suited for the particular construction project, based on travel time, congestion, and the RUC, they will manage.

## Chapter 4: Data Analysis

### 4.1 Introduction

In this chapter the results from the different simulations will be discussed and analyzed. All simulations were simulated using the parameters defined in *Chapter 3: Methodology*. Three sets of simulations were modeled to obtain the necessary data for analysis. The initial simulations were used to test for a lane bias in VISSIM, the second set of simulations are based on the collected field data, and the last set of simulations is variable volume modeling. The final analysis based on the simulation results is the road user cost analysis in which the data from the models will be applied and used to determine the road user cost difference between the three different scenarios: Free Flow, Closed Lane with TTCD and Free Flow with Reduced Speeds.

### 4.2 Lane Bias Analysis

An initial test was performed to determine if the output from VISSIM changed based on which lane was closed (left or right). By comparing the two sets of data it can be determined if VISSIM has a lane bias and if models representing right and left lane closures will be needed.

Table 4.1 shows the results of 15 iterations of lane closure analysis for a volume of 1,000 veh/hr; this simulates the effects of having 1,000 vehicles enter the study area in one hour. This volume was selected because it represents an approximate mid-range volume, if capacity on a single lane of roadway is 2,400 veh/hr, per the *Highway Capacity Manual*. The averages over the 15 iterations show that when closing the left lane, the average travel time is estimated to be 118.80 seconds, and 118.77 seconds was found to be the average travel time for the right lane. This is a 0.03 second time difference between closing the left or the right lane for 1,000 veh/hr. A t-test to determine the significance of the difference in means was conducted on these values and a p-value of 0.818 was obtained showing that the difference in these values is not statistically significant, at  $\alpha = 0.05$ .

**Table 4.1 Travel Times for Different Lane Closures 1000 veh/hr**

Iteration	Left Lane Closed		Right Lane Closed	
	Travel Time (sec)	Standard Deviation (sec)	Travel Time (sec)	Standard Deviation (sec)
1	118.89	5.12	118.77	5.01
2	119.09	5.15	119.04	5.07
3	118.68	4.86	118.67	5.03
4	118.69	5.12	118.62	5.02
5	118.81	5.06	118.80	5.08
6	119.20	5.06	119.28	5.18
7	118.14	4.78	118.05	4.83
8	118.90	5.04	118.86	4.98
9	118.82	4.98	118.60	4.94
10	119.01	5.31	119.24	5.25
11	118.86	5.04	118.93	5.07
12	118.36	5.24	118.35	5.18
13	118.61	4.93	118.51	4.84
14	118.44	5.20	118.43	5.19
15	119.53	5.38	119.43	5.24
Total Average	118.80	5.10	118.77	5.07

Table 4.2 shows the effects that lane closures have on a volume of 2200 veh/hr, as volumes approach capacity, in VISSIM. Closing the left lane gives an average travel time of 212.87 seconds while closing the right lane gives an average travel time of 208.18 seconds, a difference of 4.69 seconds. A t-test was conducted, which produced a p-value of 0.845, at  $\alpha = 0.05$ , indicating that the values are not statistically different.

**Table 4.2 Travel Times for Different Lane Closures 2200 veh/hr**

Iteration	Left Lane Closed		Right Lane Closed	
	Travel Time (sec)	Standard Deviation (sec)	Travel Time (sec)	Standard Deviation (sec)
1	147.85	45.36	154.48	47.13
2	280.38	202.08	322.69	194.12
3	148.33	29.72	157.62	28.35
4	179.08	79.96	204.83	106.16
5	165.17	63.72	155.68	56.80
6	147.24	50.30	150.44	45.29
7	260.31	140.36	169.52	58.80
8	314.06	191.77	290.27	219.01
9	337.93	279.75	266.58	188.27
10	207.33	94.00	309.72	246.46
11	233.00	198.54	175.38	60.52
12	234.86	155.70	221.43	133.29
13	248.42	199.09	153.28	30.10
14	147.87	36.86	133.94	14.48
15	142.52	18.50	258.37	171.66
Total Average	212.87	156.00	208.18	144.97

These values show that there is no lane bias in VISSIM, that closing one lane will not produce statistically different values than closing the other lane at volumes of up to 2200 veh/hr. This also provides a valid reason to continue all other models with a single lane closure and not simulate a second model for the opposite closure.

### **4.3 Northbound and Southbound Analysis of Traffic at Project Site**

Based on estimated lane closure and work times, it was decided to examine a 24 hour period of time for the models. The reasoning behind the time is the rapid bridge deck replacement method will result in a lane closure of 9.5 to 19.5 hours, without taking into account any necessary work stoppage. Therefore, modeling a worst case of a 24 hour period was done for each lane closure scenario. The volumes for this set of models represent the hourly volumes, for the peak day, found at the test site. This creates models that replicate the peak day events at the test site. Each direction of travel has three corresponding models: free flow, closed lane with temporary traffic control devices (TTCDD), and free flow with reduced speed. The model entitled “Closed lane” with TTCDD represents a typical road construction layout, with a lane closure and reduced speed zone, as outlined in Section 3.2. The model entitled “Reduced speed” represents the period of time when no construction is taking place and both lanes are open with work zone speed reductions in place. These models use the collected hourly vehicle volumes and traffic statistics collected at the test site, based on the peak day in each direction of travel.

Table 4.3 shows the average travel time for free flow conditions in both directions of travel. The travel time and the average speed are over the 15 iterations executed in VISSIM. The speed was calculated for comparison with the field-collected data. In both northbound and southbound cases the difference in speeds was within the expected 2.0 mph range: averaging the 15 model iterations provides a 99% confidence level that model values will be within 2.0 units of the collected data. This validated that the model was calibrated adequately.

**Table 4.3 Model Data for Free Flow Conditions**

Direction	Travel Time (sec)	Standard Deviation (sec)	Model Average Speed (ft/sec)	Model Average Speed (mph)	Field Collected Average Speed (mph)	Speed Difference (mph)
Northbound	94.03	6.88	105.53	71.95	73.20	-1.25
Southbound	97.12	6.22	102.14	69.64	70.89	-1.25

The two alternative models examined are a lane closure model, where a single lane is closed and temporary traffic control devices are in place to slow and channel the traffic into the open lane, and a reduced speed model where both lanes are open for vehicle travel but at the same reduced speeds as the closed lane model. Both northbound and southbound lanes were modeled with these two model setups: this provided two sets of data to examine and compare.

Table 4.4 provides the travel times for the northbound models for the 15 iterations of the model in VISSIM. The average travel time for each lane configuration can be seen in the final row of the table; it can be seen there is a 0.82 second difference in average travel time over a 24-hour period between the closed lane and reduced speed configurations.

**Table 4.4 Northbound Travel Times**

Iteration	Closed Lane Configuration		Reduced Speed Configuration	
	Travel Time (sec)	Standard Deviation (sec)	Travel Time (sec)	Standard Deviation (sec)
1	116.55	5.83	115.67	6.16
2	116.44	5.66	115.61	5.95
3	116.41	5.69	115.59	6.03
4	116.46	5.71	115.65	6.06
5	116.29	5.72	115.45	6.06
6	116.36	5.73	115.53	6.06
7	116.41	5.69	115.64	6.01
8	116.52	5.71	115.72	6.05
9	116.39	5.77	115.56	6.08
10	116.37	5.71	115.53	6.07
11	116.36	5.77	115.61	6.08
12	116.41	5.75	115.58	6.07
13	116.30	5.64	115.48	5.97
14	116.36	5.81	115.56	6.16
15	116.43	5.69	115.59	6.05
Average	116.40	5.73	115.58	6.06

Table 4.5 provides the model data for the southbound travel lanes. Comparing the average travel time for the two scenarios one can see there is a 0.54 second difference between the two methods.



**Table 4.5 Southbound Travel Times**

Iteration	Closed Lane Configuration		Reduced Speed Configuration	
	Travel Time (sec)	Standard Deviation (sec)	Travel Time (sec)	Standard Deviation (sec)
1	118.90	5.48	118.38	5.68
2	118.88	5.50	118.37	5.73
3	118.90	5.49	118.34	5.72
4	118.80	5.57	118.28	5.75
5	118.83	5.52	118.31	5.72
6	119.14	5.71	118.57	5.78
7	119.04	5.55	118.50	5.75
8	118.92	5.56	118.38	5.72
9	118.99	5.63	118.44	5.81
10	118.85	5.51	118.34	5.71
11	118.95	5.46	118.39	5.64
12	118.94	5.49	118.35	5.70
13	118.82	5.60	118.30	5.85
14	118.96	5.65	118.38	5.83
15	118.96	5.57	118.42	5.75
Average	118.92	5.55	118.38	5.74

Both the northbound and southbound models have an average travel time difference, over a 24-hour period of time, of less than 1 second. The total delay difference can be calculated for the 24-hour modeled time period for each direction. Table 4.6 provides a breakdown of the travel time difference between the closed lane and reduced speed configurations, for each direction of travel.

**Table 4.6 Calculated Delay Difference Between the Two Model Methods**

	Northbound	Southbound
Daily Volume (veh)	6532	6635
Average Delay (sec)	0.82	0.54
Total Delay (sec)	5356.24	3582.9
Total Delay (min)	89.27	59.72

Table 4.6 shows the total travel time difference between the closed lane model and the reduced speed model. The results show that over a 24-hour period of time the northbound lane will experience nearly 1.5 hours of travel delay aggregated for the entire daily traffic volume and the southbound lane will experience close to an hour of delay. These values are not large enough to easily conclude if one deck replacement method will perform better than the other, based on the travel time differences.

A statistical analysis was conducted on each pair of travel times: northbound closed lane, reduced models and the two southbound models. The travel time values acquired from the 15 iterations in VISSIM were compared according to the travel direction, first using an F-test to determine if the difference in variance in travel times between the scenarios was statistically significant. Both the northbound and southbound data sets had a p-value greater than 0.05 for the F-test: 0.46 for northbound and 0.29 for southbound data, showing that the difference in variances of the data sets is not statistically significant. A t-test was then performed on the data sets; the test resulted in a p-value less than 0.001 for both directions of travel, at  $\alpha = 0.05$ . The test revealed that the average travel times were in fact statistically different.

The average travel times, while being statistically different, are not practically different in that the difference in average travel times would not favor one bridge deck replacement method over another. Being that the peak hour volume in the model (based on the field data) is less than 600 vehicles, another method was needed to find a traffic volume point where the travel time differences are statistically and practically different. The method implemented is variable volume modeling, in which the traffic volumes in the models are increased until a difference in travel times for the closed lane and reduced speed models become different, both statistically and practically.

#### **4.4 Variable Volume Analysis**

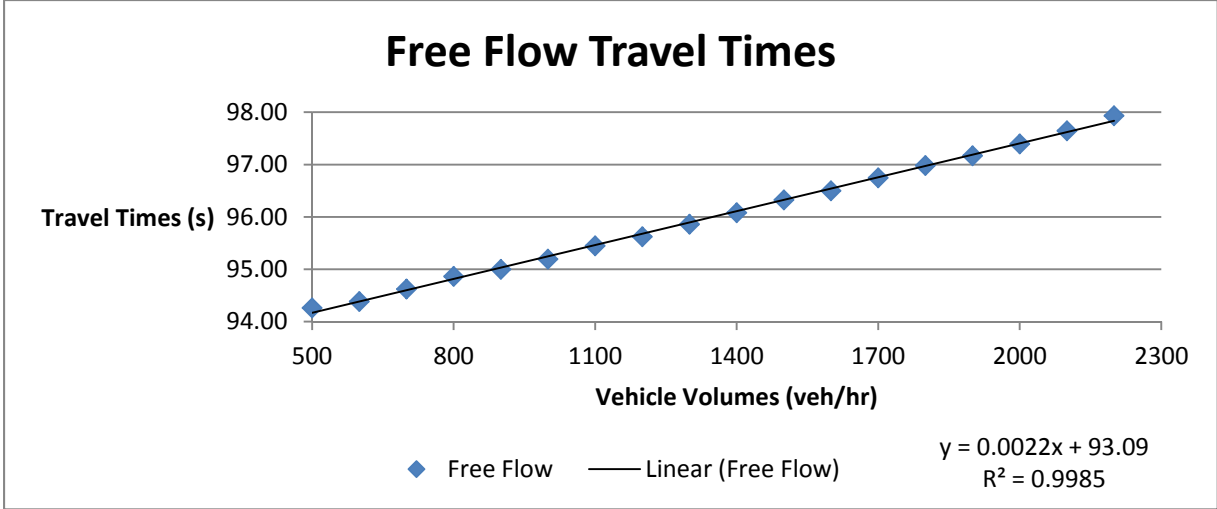
To examine a wide range of possible traffic volumes the northbound lane was used to model different volumes for a single hour. The model is first simulated at 500 veh/hr for an hour then increasing by 100 veh/hr until 2200 veh/hr is reached. The three model configurations of free flow, lane closure, and reduced speed were modeled for all 18 volumes.

Table 4.7 shows the average travel times for the three model configurations for all 18 different volumes. The values found in the tables show that under both free flow conditions (normal speeds, posted 70 mph speed limit) and reduced speeds the average travel times increase by nearly 4 seconds per vehicle from volumes of 500 veh/hr to 2200 veh/hr. The model for a closed lane scenario shows a steady increase in travel times until 1800 veh/hr when the travel times begin to increase at a larger rate for each additional 100 veh/hr, resulting in an increase in average travel time of approximately 119 seconds per vehicle from volumes of 500 veh/hr to 2200 veh/hr.

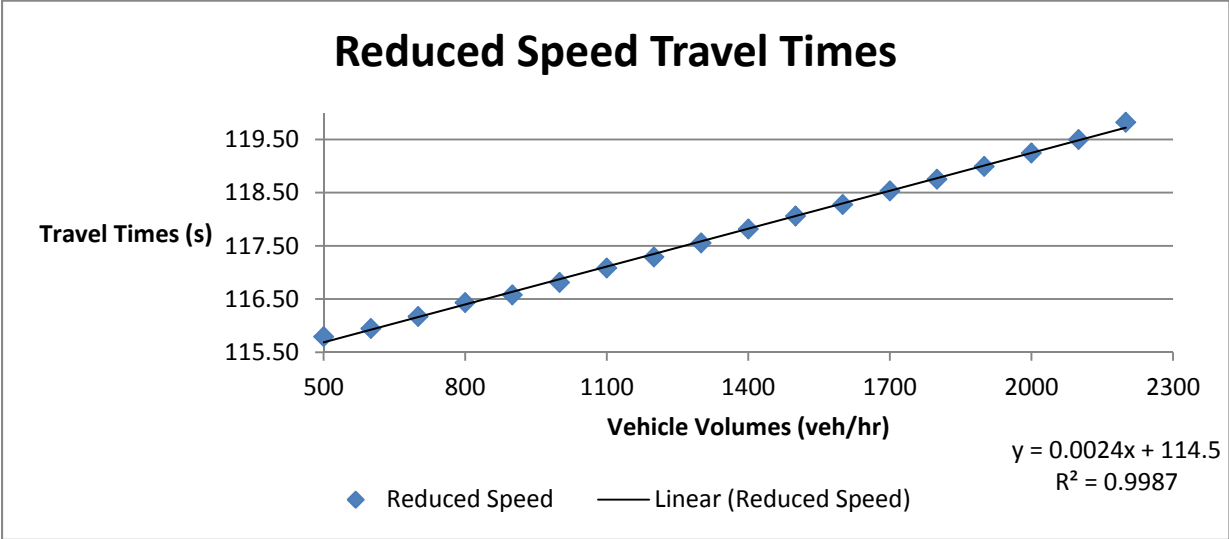
**Table 4.7 Average Travel Times for 18 Different Volumes**

	Free Flow	Lane Closure	Reduced Speed
Volume (veh/hr)	Average Travel Time (s)		
500	94.26	116.97	115.79
600	94.38	117.27	115.94
700	94.62	117.75	116.17
800	94.86	118.14	116.43
900	95.00	118.45	116.58
1000	95.20	118.77	116.81
1100	95.45	119.16	117.08
1200	95.62	119.54	117.29
1300	95.86	119.84	117.55
1400	96.08	120.26	117.82
1500	96.33	120.74	118.06
1600	96.50	121.05	118.28
1700	96.75	121.73	118.53
1800	96.98	122.47	118.75
1900	97.17	124.54	118.99
2000	97.39	129.46	119.24
2100	97.64	137.13	119.50
2200	97.93	235.63	119.82

Figures 4.1 and 4.2, respectively, show the average travel times for free flow and reduced speed conditions with fitted linear trend lines. The slopes in the lines are similar for both models; this is to be expected as the only difference is a slower speed in the reduced speed model. Both of these data sets were fitted with a linear equation with an  $R^2$  value of more than 0.99, meaning that the linear regression equations are acceptable predictors to find any travel time based on vehicle volumes.

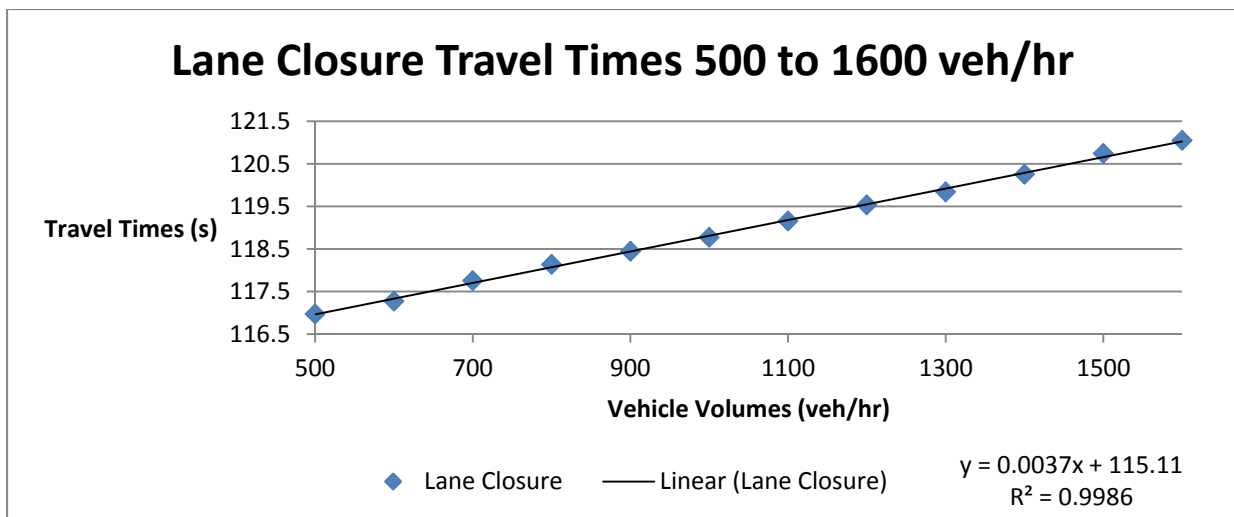


**Figure 4-1 Free Flow Travel Times with Fitted Trend Line.**

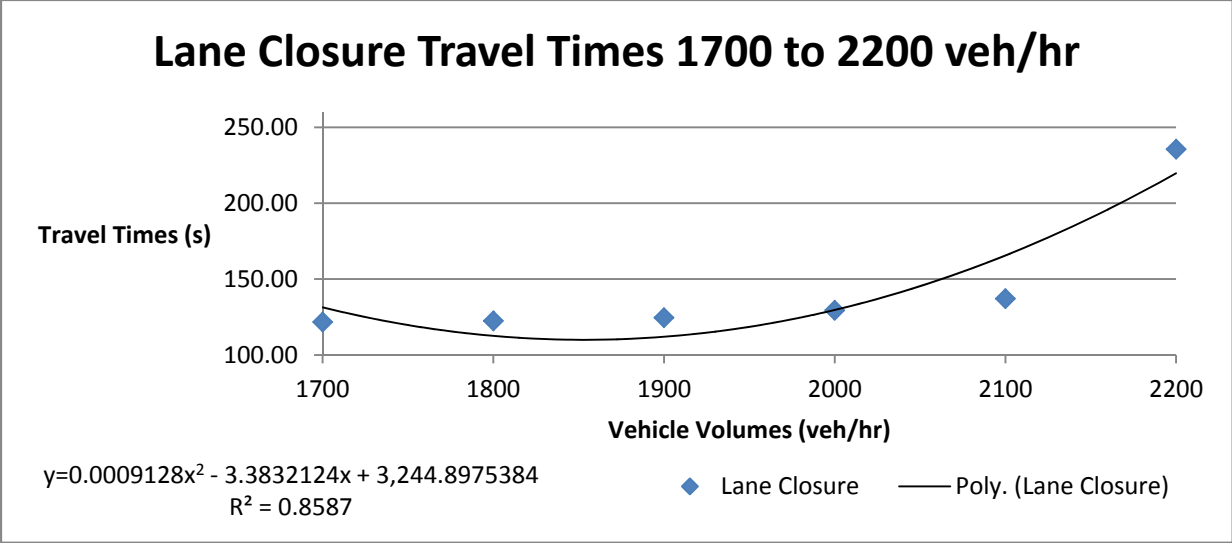


**Figure 4-2 Reduced Speed Travel Times with Fitted Trend Line.**

Fitting data from the closed lane simulation required dividing the data into two different sets for mathematical modeling purposes to achieve the best  $R^2$  value, as the best fitting single equation was a fourth order polynomial (equation) or greater. Using a fourth order polynomial, while technically acceptable, is rather complex for the phenomenon being modeled. Different breakpoints were tested until the best overall fit for all data (as measured by  $R^2$ ) was obtained. Setting the breakpoint of data between the 1500 veh/hr and 1600 veh/hr resulted in the second data set having an  $R^2$  value of 0.77, while setting the breakpoint after 1700 veh/hr decreased the number of data points for the second data set resulting in a higher  $R^2$  value but with fewer data points to do calculations. Therefore the first set of used data consists of volumes of 1600 veh/hr and less, and the second set is volumes of 1700 veh/hr and larger. Figure 4.3 shows the first set of closed lane data; this portion of the data is similar to both the free flow and reduced speed models. In this range of volumes, a linear fit was sufficient. The set of data for volumes of 1700 veh/hr and larger can be seen in Figure 4.4; this set of data is best fit with a second order polynomial (quadratic) equation or higher, the second best fit was with a exponential equation with an  $R^2$  value of 0.58.



**Figure 4-3 Lane Closure Travel Times 500 to 1600 veh/hr with Fitted Trend Line.**



**Figure 4-4 Lane Closure Travel Times 1700 to 2200 veh/hr with Fitted Curve.**

The data from the four data sets provides a method of estimating any travel time based on volume for the three different lane configurations modeled. Though this is useful, it does not provide an objective indication regarding a volume at which one bridge deck replacement method is better suited than another on the basis of traffic impacts and RUCs. For this a series of t-tests were applied to the travel times for each volume pair for the closed lane and reduced speed scenarios; this will provide a numerical value to determine if the difference in travel times between the scenarios is statistically significant. The values of the *t*-statistic were calculated using the data in Table 4.7 for the average travel times and Table 4.8 for the standard deviations of the 18 different models; the *t*-values are shown in Table 4.9.

**Table 4.8 Standard Deviation for Travel Times for Different Volumes**

	Free Flow	Lane Closure	Reduced Speed
Volume (veh/hr)	Standard Deviation (s)		
500	6.92	5.59	6.02
600	6.80	5.45	5.92
700	6.76	5.34	5.86
800	6.71	5.22	5.76
900	6.65	5.13	5.70
1000	6.59	5.07	5.62
1100	6.50	4.93	5.48
1200	6.42	4.97	5.40
1300	6.32	4.89	5.28
1400	6.31	4.95	5.22
1500	6.27	5.16	5.14
1600	6.21	4.94	5.05
1700	6.22	5.20	5.03
1800	6.21	6.00	5.00
1900	6.22	8.55	4.94
2000	6.14	16.18	4.89
2100	6.13	27.52	4.85
2200	6.08	173.89	4.75

The standard deviations for both the free flow and reduced speed scenarios both decrease as the traffic volume increases. This is expected as more vehicles are present on the roadway and with no lane closure the vehicles begin to travel across a narrower range of speeds. The closed lane scenario shows that between 1800 and 1900 veh/hr, the standard deviation begins to increase greatly; this is due to the vehicles having to stop in a queue that has formed in the closed lane.

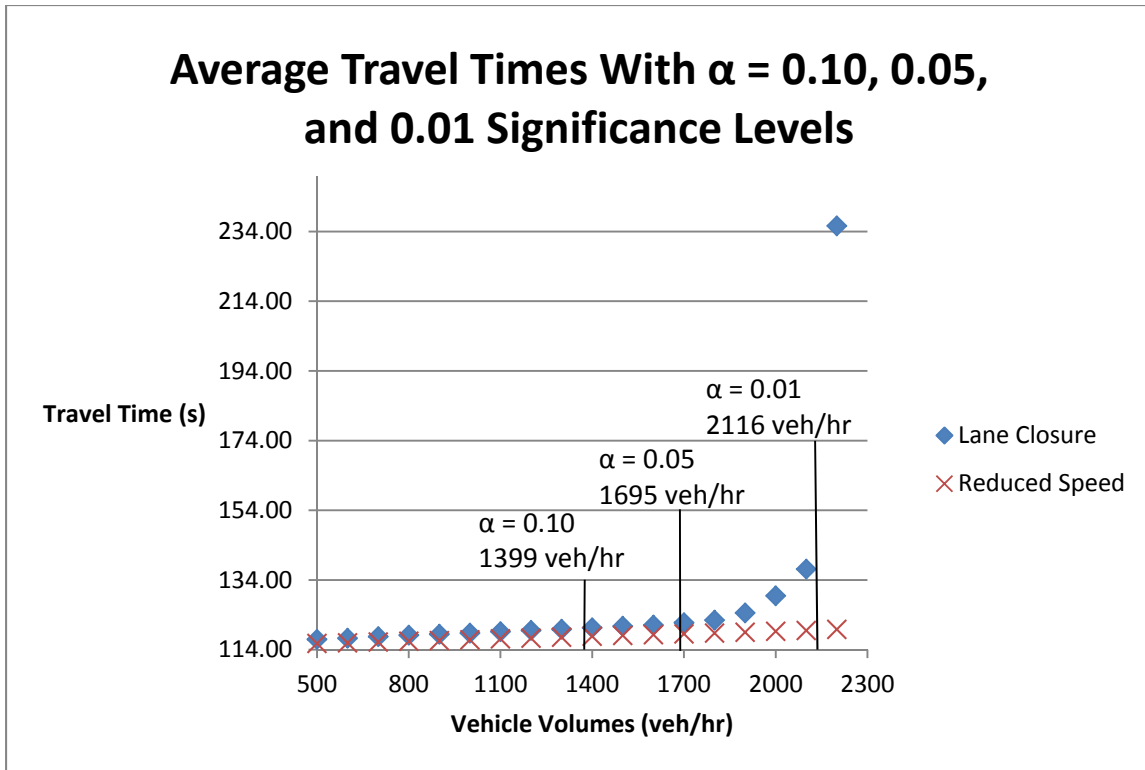
The values of the t-statistic were calculated to compare the travel time means of the closed lane and reduced speed models, the values can be seen in Table 4.9. These values can provide guidance for determining an acceptable value of travel time differences.

**Table 4.9 t-values Comparing Closed Lane and Reduced Speed Models**

Volume (veh/hr)	t-Values
500	0.557
600	0.640
700	0.772
800	0.849
900	0.944
1000	1.003
1100	1.090
1200	1.184
1300	1.230
1400	1.314
1500	1.425
1600	1.524
1700	1.709
1800	1.845
1900	2.176
2000	2.340
2100	2.445
2200	2.578

Three other t-values were calculated using interpolation from Table 4.9 – those associated with significance levels of  $\alpha = 0.10$ ,  $0.05$ , and  $0.01$ . These values will be used to identify volumes at several values of probability of incorrectly characterizing the travel time differences as statistically significant. Using a table of critical values for the t-statistic, with 28 degrees of freedom, the t-values for the  $\alpha = 0.10$ ,  $0.05$ , and  $0.01$  significance levels were found to be 1.313, 1.701 and 2.467, respectively. By linear interpolation, which is an approximation for the nonlinear change in t-values, the volumes that can be associated with these significance levels are 1399 veh/hr for  $\alpha = 0.10$ , 1695 veh/hr for  $\alpha = 0.05$  and 2116 veh/hr for  $\alpha = 0.01$ . Figure 4.5 shows these three calculated significance levels placed on the travel time curves of both the closed lane and reduced speed models. This chart can be viewed as a quick guide to the findings in this section.





**Figure 4-5 Average Travel Times with Significance Levels.**

#### 4.5 Road User Cost Analysis

The road user cost was calculated using the methods discussed in *Chapter 3: Methodology*. The input values needed that were not provided in the AASHTO publication “*User and Non-User Benefit Analysis for Highways*” also called the “Red Book” are: travel time, study length, and current fuel prices. The travel times have been discussed in previous sections in this chapter and vary with each model type; the study length used is 9920 ft. (1.87 miles). The used value for the cost of a gallon of gasoline was \$3.604 per gallon and \$3.835 per gallon for diesel fuel; these numbers were obtained on August 16, 2011(EIA 2011) and can fluctuate based on oil prices. Two sets of RUC values will be discussed in this section based on the travel time values found in Sections 4.2 and 4.3 of this chapter.

##### 4.5.1 Northbound and Southbound Road User Costs

The travel time data obtained from the three simulations of free flow, closed lane with TTCD, and free flow with reduced speed, based on the collected traffic data at the study site, was used to calculate the RUC for both the northbound and southbound travel lanes; these values can be found in Tables 4.3, 4.4 and 4.5. The travel times were converted to equivalent speed values to interpolate the fuel consumption per minute of delay based on Table 3.7 in the previous chapter; the interpolated values are presented in Table 4.10. These values are used in calculating the vehicle operating component of the RUC. Using the travel times, interpolated fuel consumption rate and the predefined values found in the “Red Book”, the RUC was calculated.

**Table 4.10 Interpolated Fuel Consumption (Gallons) per Minute of Delay by Vehicle Type**

	Northbound		Southbound	
	Passenger Cars	Trucks	Passenger Cars	Trucks
Free Flow	0.049	0.636	0.047	0.617
Closed Lane	0.035	0.521	0.034	0.511
Reduced Speed	0.036	0.525	0.034	0.513

The value of time (VOT) is based on the travel time (in hours) being multiplied with the value of time per hour of each vehicle type, \$13.92 for passenger vehicles and \$21.24 for trucks. The calculated VOT for each direction of travel and vehicle type are found in Table 4.11.

The ownership cost is again based on travel time in hours being multiplied by a value of dollars per hour, \$0.471/hr for passenger cars and \$3.688/hr for trucks. The calculated ownership cost for both directions of travel and vehicle type is seen in Table 4.11.

The final component needed to calculate the RUC is the operating cost. There are two components in this value. First the interpolated values in Table 4.10, fuel costs, and travel time in minutes are multiplied together. Second other operating costs are calculated by multiplying the length (in miles) of the study area by: \$0.04 for passenger cars and \$0.05 for trucks. These calculated values are found in Table 4.11.

**Table 4.11 RUC Components**

Value of Time for Each Vehicle Type (\$)				
	Northbound		Southbound	
	Passenger Cars	Trucks	Passenger Cars	Trucks
Free Flow	0.363	0.555	0.376	0.573
Closed Lane	0.45	0.687	0.46	0.702
Reduced Speed	0.447	0.682	0.458	0.699
Ownership Cost for Each Vehicle Type (\$)				
Free Flow	0.012	0.096	0.013	0.1
Closed Lane	0.015	0.119	0.016	0.122
Reduced Speed	0.015	0.118	0.016	0.121
Operating Cost for Each Vehicle Type (\$)				
Free Flow	0.354	3.915	0.347	3.924
Closed Lane	0.321	3.971	0.317	3.976
Reduced Speed	0.322	3.969	0.318	3.975

The values for each component in Tables 4.11 are then added together to obtain the road user cost per vehicle. These values are multiplied by the vehicle distributions (found in Table 3.3) to

obtain the RUC per vehicle type; this can be seen in Table 4.12. These values are added to obtain the total RUC for the 24 hour period in each direction, seen in Table 4.13.

**Table 4.12 RUC for Each Vehicle Type (\$)**

	Northbound		Southbound	
	Passenger Cars	Trucks	Passenger Cars	Trucks
Free Flow	3955.29	5070.55	3757.88	7014.82
Closed Lane	4261.04	5304.52	4049.03	7324.62
Reduced Speed	4249.41	5296.30	4041.79	7317.15

**Table 4.13 Total RUC for Each Travel Direction (\$)**

	Northbound	Southbound
Free Flow	9025.84	10772.70
Closed Lane	9565.56	11373.65
Reduced Speed	9545.70	11358.94

The higher RUC in the southbound lane is expected due to a slightly higher volume and a higher percentage of heavy vehicles. Comparing the RUC for the Closed Lane and Reduced Speed Scenarios shows a difference of less than \$20 for a 24 hour period; such a difference does not provide evidence to claim one deck replacement method would benefit road users over the other.

#### **4.5.2 Variable Volume Road User Cost**

The steps taken to calculate the variable volume road user cost data is the same as the method used in the previous section of this chapter. The travel time data found in Table 4.7 is used to calculate the RUC for each of the hourly volumes. The total RUC for the variable volume models can be seen in Table 4.14: the VOT, ownership and operating cost tables can be found in the appendix.

**Table 4.14 Total RUC for Variable Volume Models**

	Free Flow	Lane Closure	Reduced Speed
Volume (veh/hr)	Road User Cost (\$)		
500	691.30	733.26	731.07
600	829.79	880.58	877.63
700	968.61	1028.59	1024.49
800	1107.59	1176.67	1171.62
900	1246.42	1324.79	1318.55
1000	1385.54	1473.19	1465.93
1100	1524.97	1622.07	1613.62
1200	1664.25	1771.23	1761.24
1300	1803.92	1920.29	1909.28
1400	1943.65	2070.17	2057.50
1500	2083.64	2220.75	2205.84
1600	2223.42	2370.64	2354.17
1700	2363.83	2523.05	2502.92
1800	2504.45	2676.43	2651.61
1900	2644.89	2841.31	2800.61
2000	2785.76	3032.74	2949.89
2100	2926.99	3254.91	3099.34
2200	3068.72	4415.25	3249.58

The values in Table 4.14 were graphed to obtain equations to predict RUC values based on the vehicle volumes. It was found that the free flow and reduced speed scenarios were linear. The following equations represent these two scenarios:

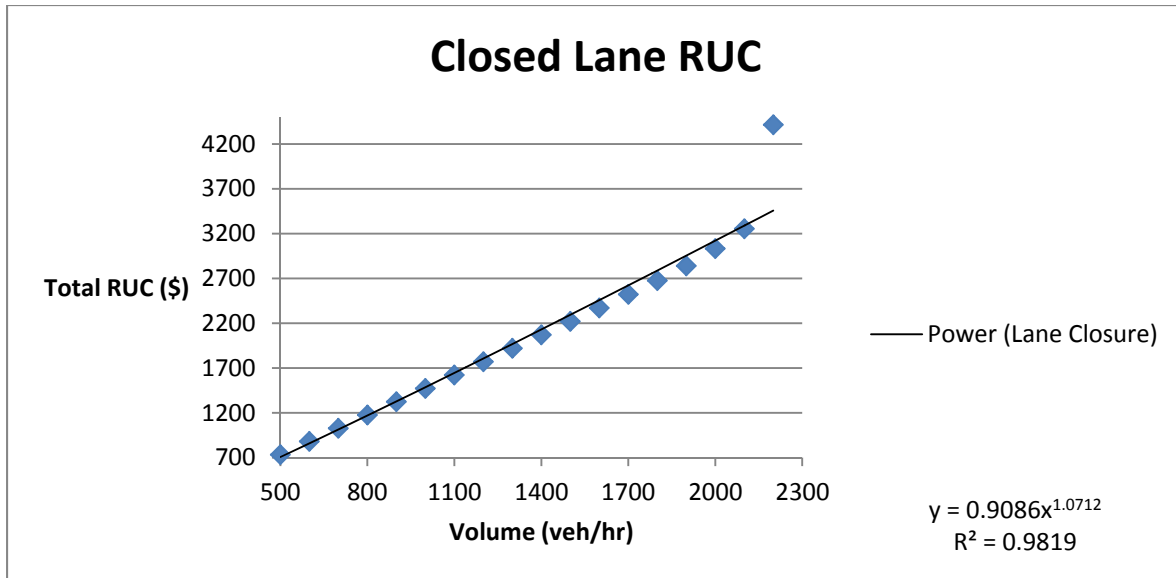
$$RUC(\$) = 1.398(\text{Volume}) - 11.341$$

Equation 4.1

$$RUC(\$) = 1.4812(\text{Volume}) - 13.827$$

Equation 4.2

The closed lane scenario can be graphed with a single power equation curve as seen in Figure 4.6. Other curves were fitted to the data, all with high  $R^2$  values (greater than 0.90). The power function was chosen due to it having higher  $R^2$  values than linear or quadratic models. The equations for the three scenarios can be used to calculate the RUC based on traffic volumes for a single hour.



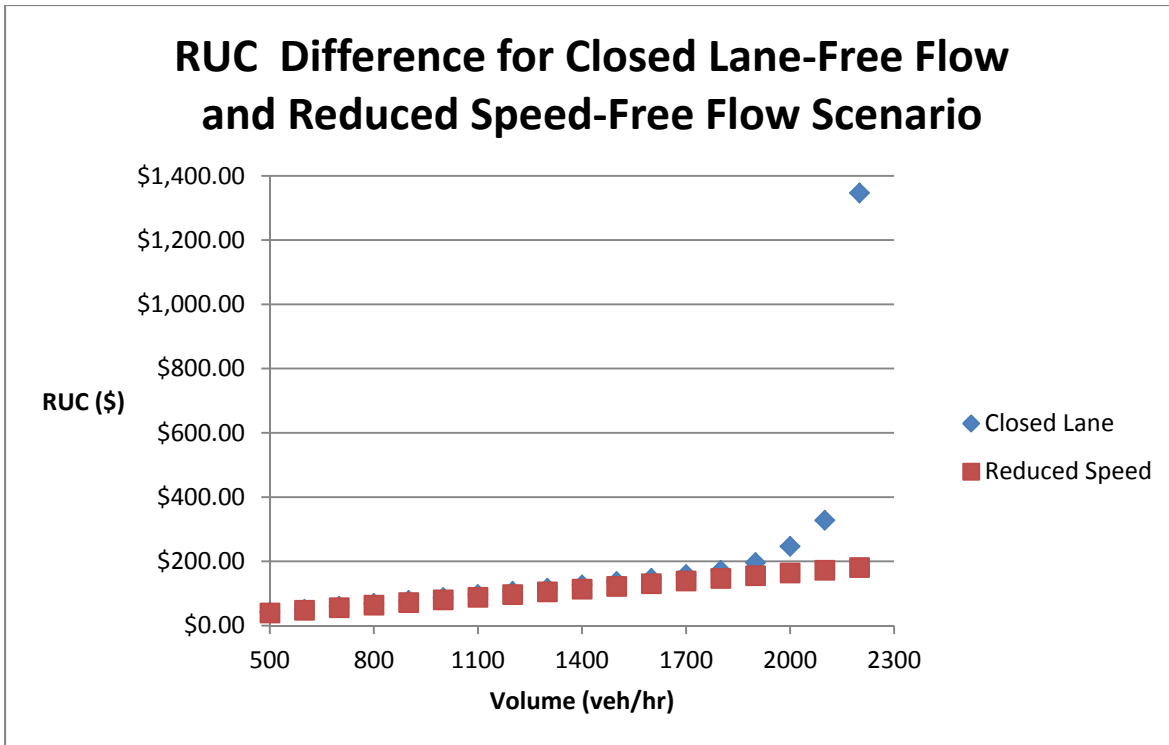
**Figure 4-6 Lane Closure RUC with Fitted Curve.**

Three volumes were calculated based on the t-statistics to find the  $\alpha = 0.10, 0.05,$  and  $0.01$  significance levels for the travel time data. Using these volumes with the acquired RUC equations the road user cost values can be calculated for each of the three scenarios. These values can be seen in Table 4.15.

**Table 4.15 RUC for the Significance Levels of  $\alpha = 0.10, 0.05,$  and  $0.01$**

	Free Flow	Lane Closure	Reduced Speed
Volume (veh/hr)	Road User Cost (\$)		
$\alpha = 0.10$ (1399 veh/hr)	1944.46	2128.99	2058.37
$\alpha = 0.05$ (1695 veh/hr)	2358.27	2614.93	2496.81
$\alpha = 0.01$ (2116 veh/hr)	2946.83	3316.39	3120.39

Comparing RUC from the free flow scenarios to the other two scenarios will provide the incurred RUC for the closed lane and reduced speed scenarios per hour. Figure 4.7 shows the difference between the free flow scenario and the closed lane scenario, as well as, the difference between the free flow scenario and the reduced scenario. This shows the RUC difference per hour at the indicated volumes.



**Figure 4-7 RUC Differences (Closed Lane & Free Flow), (Reduced Speed & Free Flow).**

### 4.5.3 Total Project RUC

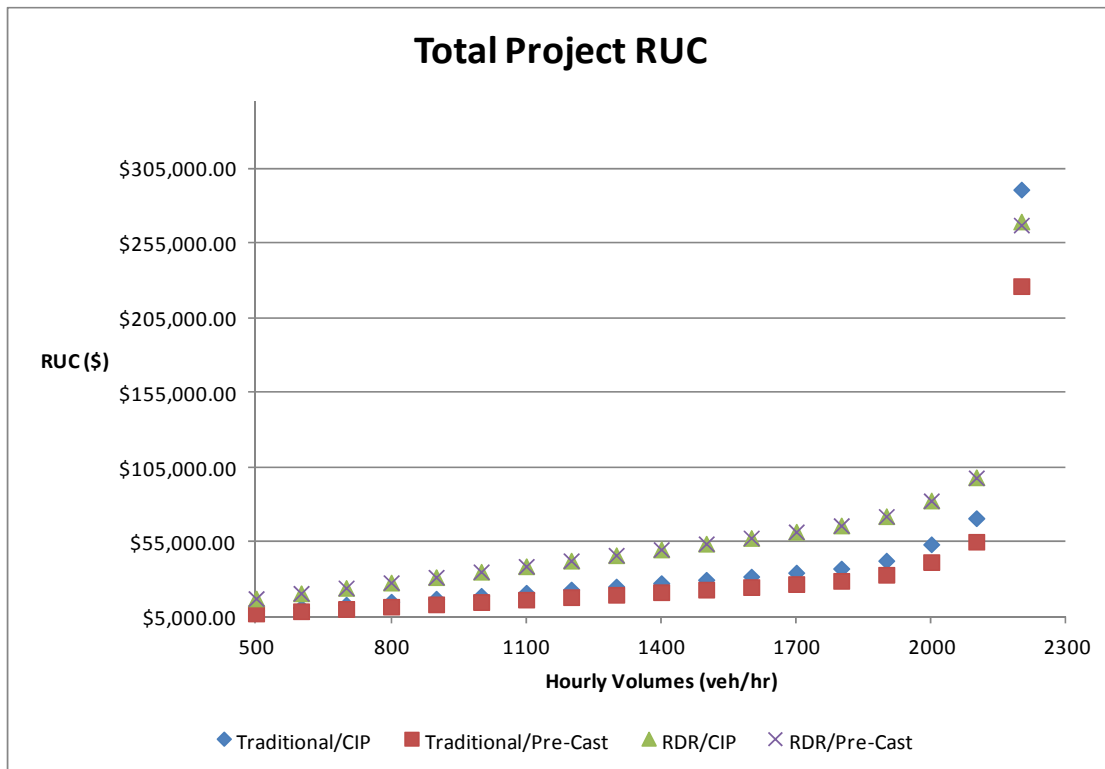
Using the hourly RUC calculated for the variable volume models the total project RUC was calculated. Using the project schedule developed in Chapter 4 of Part 1 of this report, the four different construction methods have the following single lane replacement schedule: (1) traditional/CIP has 108 hours of weekday lane closure, (2) traditional/pre-cast has 84 hours of weekday lane closure, (3) RDR/CIP has 83 hours of weekend closure and 127 hours of weekday reduced speed, (4) RDR/pre-cast has 82 hours of weekend closure and 128 hours of weekday reduced speed. Using the calculated RUC's for each hourly volume obtained in the variable volume models the total project RUC can be calculated by doubling the schedule of each single lane construction method.

Table 4.16 shows the total project RUC, assuming a constant hourly volume through the length of the project on a 225 ft bridge. This constant hourly traffic volume is not likely to occur on any roadway but by looking at the calculated RUC values for each hourly volume an estimate can be made regarding the expected RUC at a similar construction site.

**Table 4.16 Total Project RUC for Each Hourly Volume (\$)**

Hourly Volume	Traditional/CIP	Traditional/Pre-Cast	RDR/CIP	RDR/Pre-Cast
500	\$9,064.27	\$7,049.99	\$17,068.66	\$17,064.28
600	\$10,972.23	\$8,533.95	\$20,584.02	\$20,578.10
700	\$12,956.46	\$10,077.25	\$24,150.91	\$24,142.70
800	\$14,921.31	\$11,605.46	\$27,730.42	\$27,720.32
900	\$16,926.70	\$13,165.21	\$31,329.25	\$31,316.78
1000	\$18,934.22	\$14,726.61	\$34,970.56	\$34,956.03
1100	\$20,975.03	\$16,313.92	\$38,636.45	\$38,619.54
1200	\$23,107.22	\$17,972.28	\$42,394.51	\$42,374.54
1300	\$25,137.93	\$19,551.72	\$46,080.94	\$46,058.90
1400	\$27,328.61	\$21,255.58	\$49,921.94	\$49,896.61
1500	\$29,615.14	\$23,034.00	\$53,796.56	\$53,766.73
1600	\$31,798.49	\$24,732.16	\$57,647.05	\$57,614.11
1700	\$34,389.63	\$26,747.49	\$61,756.62	\$61,716.37
1800	\$37,148.01	\$28,892.90	\$65,926.11	\$65,876.46
1900	\$42,426.68	\$32,998.53	\$72,159.00	\$72,077.61
2000	\$53,346.39	\$41,491.64	\$82,685.24	\$82,519.54
2100	\$70,830.97	\$55,090.75	\$98,212.24	\$97,901.10
2200	\$290,849.69	\$226,216.43	\$269,461.14	\$267,129.80

Table 4.16 shows that RDR methods do not provide a benefit to RUC. The RDR methods have a larger impact on road users in all but one instance, the traditional/CIP method at 2200 veh/hr. This is due to the five days of reduced speed between the weekends of construction for the RDR methods. As seen in section 4.4.2 the RUC for each hourly volume for the closed lane and reduced speed scenario are within \$100 for volumes 500 veh/hr to 1900 veh/hr and increase after that point. Because of this the total project RUC is determined by the total hours of change in traffic conditions. Both RDR methods have 108 hours of traffic impact where the traditional methods have 108 hours for CIP and 84 hours for pre-cast. Figure 4.8 shows the RUC for each of the four construction methods at the test site, based on estimated projected durations developed in Part 1 of this report.



**Figure 4-8 Total Project RUC.**

To look at longer possible traditional closures and compare them to the RDR methods Table 4.17 has the calculated RUC for the different replacement methods. Table 4.17 shows the RUC for multiple weeks of lane closure using both traditional deck replacement methods. It is assumed a single week for each method will require the same lane closure time as mentioned previously. This approach shows the differences in the two traditional deck replacement methods; expanding it for multiple weeks provides numerical data representing the RUC if the bridge deck replacement took multiple weeks to finish.



**Table 4.17 Different Traditional Deck Replacement Lengths of Time and RDR Methods**

	Two Weeks		Three Weeks		Four Weeks		Five Weeks			
Volume	CIP	Pre-Cast	CIP	Pre-Cast	CIP	Pre-Cast	CIP	Pre-Cast	RDR/CIP	RDR/Pre-Cast
500	\$9,064	\$7,050	\$13,596	\$10,575	\$18,129	\$14,100	\$22,661	\$17,625	\$8,534	\$8,532
600	\$10,972	\$8,534	\$16,458	\$12,801	\$21,944	\$17,068	\$27,431	\$21,335	\$10,292	\$10,289
700	\$12,956	\$10,077	\$19,435	\$15,116	\$25,913	\$20,154	\$32,391	\$25,193	\$12,075	\$12,071
800	\$14,921	\$11,605	\$22,382	\$17,408	\$29,843	\$23,211	\$37,303	\$29,014	\$13,865	\$13,860
900	\$16,927	\$13,165	\$25,390	\$19,748	\$33,853	\$26,330	\$42,317	\$32,913	\$15,665	\$15,658
1000	\$18,934	\$14,727	\$28,401	\$22,090	\$37,868	\$29,453	\$47,336	\$36,817	\$17,485	\$17,478
1100	\$20,975	\$16,314	\$31,463	\$24,471	\$41,950	\$32,628	\$52,438	\$40,785	\$19,318	\$19,310
1200	\$23,107	\$17,972	\$34,661	\$26,958	\$46,214	\$35,945	\$57,768	\$44,931	\$21,197	\$21,187
1300	\$25,138	\$19,552	\$37,707	\$29,328	\$50,276	\$39,103	\$62,845	\$48,879	\$23,040	\$23,029
1400	\$27,329	\$21,256	\$40,993	\$31,883	\$54,657	\$42,511	\$68,322	\$53,139	\$24,961	\$24,948
1500	\$29,615	\$23,034	\$44,423	\$34,551	\$59,230	\$46,068	\$74,038	\$57,585	\$26,898	\$26,883
1600	\$31,798	\$24,732	\$47,698	\$37,098	\$63,597	\$49,464	\$79,496	\$61,830	\$28,824	\$28,807
1700	\$34,390	\$26,747	\$51,584	\$40,121	\$68,779	\$53,495	\$85,974	\$66,869	\$30,878	\$30,858
1800	\$37,148	\$28,893	\$55,722	\$43,339	\$74,296	\$57,786	\$92,870	\$72,232	\$32,963	\$32,938
1900	\$42,427	\$32,999	\$63,640	\$49,498	\$84,853	\$65,997	\$106,067	\$82,496	\$36,080	\$36,039
2000	\$53,346	\$41,492	\$80,020	\$62,237	\$106,693	\$82,983	\$133,366	\$103,729	\$41,343	\$41,260
2100	\$70,831	\$55,091	\$106,246	\$82,636	\$141,662	\$110,182	\$177,077	\$137,727	\$49,106	\$48,951
2200	\$290,850	\$226,216	\$436,275	\$339,325	\$581,699	\$452,433	\$727,124	\$565,541	\$134,731	\$133,565

#### **4.6 Summary of Analysis Results**

The charts and graphs provided in this chapter provide a method of comparing the traffic impacts and resultant RUCs of the different deck replacement methods. The results showed little difference in the effects of bridge deck replacement methods on traffic, over a 24 hour period of time. Comparing the effects the different methods have on increasing hourly traffic volumes provides data that can be used to determine which method is better suited for a particular location. The total project RUC showed that weekend RDR methods have a greater impact on road users than weekday traditional deck replacement methods, for a 225 ft bridge.

The variable volume modeling procedure provides travel time data for 18 different hourly volumes for the test site. The charts and graphs from this data provide a quantified method of determining the preferred construction staging method based on hourly volumes at a construction site. These values also provide a guideline for the construction timing selected for a construction project. The significance levels obtained from the variable volume travel times provide a method of comparing whether the two different deck replacement methods are statistically the same or not. Finding the hourly volumes for the significance levels of  $\alpha = 0.10$ , 0.05, and 0.01, for the probability of incorrectly characterizing the travel time differences as statistically significant, provides a decision maker with three volumes for quick reference to where the different deck replacement methods would be.

The road user cost calculations enhance the values obtained in the VISSIM models. Being that the travel time values are a major contributor to the RUC values, it was expected that the RUC values follow the same trends as the travel times from all of the different scenarios modeled. The values obtained from the variable volume models provide RUC values for 18 different hourly volumes; these values can be used in determining the deck replacement method to reduce RUC.

The total project RUC showed that under optimal conditions at the test site the traditional/pre-cast deck replacement method had the lowest impact on RUC. As seen in Figure 4.17 the total RUC is dependent on length of time the construction is to occur. If a traditional deck replacement method takes longer than two weeks, for a single lane, the RDR methods have a lower impact on the RUC; this is assuming a bridge length of 225 ft.

## **Chapter 5: Conclusion and Recommendations**

Several traffic microsimulation models were developed to examine the effects that different bridge deck replacement methods have on traffic impacts and road user costs. The analysis of these models provided a statistical comparison of the effects of different lane configurations in a work zone. The lane configurations consist of three different scenarios: free flow (no change in configuration), closed lane configuration (where temporary traffic control devices (TTCD) are in place and the speed limit is reduced), and reduced speed (where speeds are reduced and no TTCD is in place). The overall travel time in each of the simulations was used to create a road user cost (RUC) analysis that produces an economic cost from the travel times of each scenario modeled. A literature review was completed to establish knowledge of microsimulation modeling, particularly in freeway work zones, and development of RUC analysis. This obtained knowledge allowed for correct and efficient modeling techniques used in the development of the RUC. The following sections present the conclusion and findings in this project as well as recommendations regarding the findings and any future research into this area.

### **5.1 Conclusion**

The initial model sets showed that for the study area on Interstate 59 at State Route 68 in Collinsville, Alabama, where the peak hour volume is less than 550 vehicles in each direction, the difference in travel times between closing a lane with TTCD and having only a reduced speed limit is statistically significant but is not significant from a practical perspective. With a delay difference of less than one second per vehicle it was not possible to deem one bridge deck replacement method better than another, from a traffic impact and RUC perspective. The variable volume models were then used to establish a point at which it could be deemed that one bridge deck replacement method is more suited than the other with respect to traffic impacts.

The variable volume models showed that the reduced speed scenario behaved similarly to normal roadway conditions. The average travel times for the free flow and reduced speed scenarios at 500 veh/hr were 94.26 seconds and 115.79 seconds, respectively. These values increased approximately linearly to 97.93 seconds for the free flow scenario and 119.82 seconds for the reduced speed scenario at 2200 veh/hr. The growth for the closed lane scenario produced a linear growth pattern from 500 to 1600 veh/hr. From 1700 veh/hr to 2200 veh/hr, the growth in travel times is best modeled as a 2<sup>nd</sup> order polynomial with growth from 121.73 seconds to 235.63 seconds. Statistical analysis was performed to compare the reduced speed and closed lane scenarios. The travel times at which the differences are statistically significant at significance levels of  $\alpha=0.10$ , 0.05, and 0.01 were calculated and found to be at volumes of 1399, 1695 and 2116 veh/hr respectively. These three volumes can serve as a guideline as to which bridge deck replacement method to use based on the vehicle volumes at a particular location and the desired level of significance in the difference between travel times for each scenario.

The road user cost showed the same trends as the travel time data obtained from all of the different simulations. This was to be expected as the calculation of RUCs use travel time as the main input variable. The values obtained from the RUC calculations, based on the variable volume modeling approach, provide a dollar value for road user cost based on the vehicle volumes during a simulated hour.

Calculating the total RUC for a project showed that the total RUC is dependent on the total length of time that traffic is not traveling under normal conditions. While traditional bridge deck replacement methods close a single lane of traffic for the entirety of that lane's replacement, the total hourly impact to road users is less than that of the RDR methods and therefore impacts the road user less.

## **5.2 Recommendations for Use of this Research**

The results from this research provide an objective decision-support tool to help with determining when rapid bridge deck replacement should be used and when it would not be a benefit to the motorist. The RUC calculations provide another method of determining the most appropriate bridge deck replacement method. Using the tables and figures in Section 4.4.3 the effects the different bridge deck replacement methods have on RUC can be seen. These tables and figures provide monetary values based on each deck replacement method for the test site, a 225 ft bridge, longer bridges will have higher RUC's than those seen in this report. Figure 4.8 and Table 4.16 provides a way to compare the impact of each deck replacement method has on road users based on total RUC. Table 4.17 shows the effects different lengths of lane closures have on RUC. The RDR method used in this report is for weekend lane closure, weeknight lane closures using RDR may have lower total RUC.

## **5.3 Recommendations for Future Research**

Upon the completion of this study, multiple ideas to further the research into this topic were developed. These issues pertain to the differences between urban and rural interstates, the impact of traffic volumes, and roadway geometry. In many cases, performing vehicle studies at different locations is recommended.

Rural interstates are typically 4-lane divided roadways; this was the case with the study area in this research. Applying the site-specific data to a 6 or 8-lane divided interstate, commonly found in urban settings, is not feasible and would not represent the lane closure configurations correctly. This study addressed this issue, as much as possible within the study scope, with the variable volume analysis approach. Another potential issue is the proximity of interchanges to the construction area; this also effects construction in urban settings more than rural. Close proximity of interchanges affects the weaving patterns before and after a potential construction zone, which could potentially lead to longer travel times for commuters.

Traffic volumes on a roadway will impact the travel times, as seen in the variable volume modeling. The base model for this research was created using traffic data from a relatively low volume Interstate roadway; the peak hour volume for both directions of travel was approximately 525 vehicles. Drivers in higher volume traffic will have different driving behaviors which could affect the base travel times and speeds.

The geometry of a bridge can have an impact on travel times during a construction project. The length of the bridge in the study area for this project was 225 ft. with little horizontal curvature and little vertical curvature; this geometry does not represent all bridges. Longer bridges will likely have different effects on travel times; this research does not provide any data to support what effects different bridge lengths have on travel time. The curvature, both vertical and horizontal, of the bridge would possibly impact the travel times during construction.

Obtaining more in-depth vehicle data, such as gap time and accurate vehicle type counts, during a construction event would provide useful data in completing a more accurate model representation of the traffic conditions during construction. These collected data would be used to adjust some of the more refined calibration parameters in VISSIM. Modeling these data would provide an accurate model of vehicles during a construction event.

All of these recommendations require studying the impact of construction on commuters at different and varying locations. Different lane configurations and geometries could impact commuters in different ways during construction. These differences should be studied before deciding on the best bridge deck replacement method at a particular location.

Validation of these models should be done to verify the accuracy of the three different scenarios. To validate these models new data would need to be collected. To validate the “Free Flow” scenario a new set of data at the test site under normal operating conditions is needed. To validate the “Closed Lane” and “Reduced Speed” scenarios data during those lane configurations will need to be collected. The new vehicle volume data can be entered into the existing models and new sets of model data can be compared to the field collected data. If the model data is within the tolerable limits decided upon then the models can be considered valid and any data they produce is accurate.

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**Appendix A:  
Supporting Road User Cost Data**

**Table A-1 VOT for Each Vehicle Type for Each Scenario (\$)**

Volume	Free Flow		Lane Closure		Reduced Speed	
	Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck
500	0.364	0.556	0.452	0.690	0.448	0.683
600	0.365	0.557	0.453	0.692	0.448	0.684
700	0.366	0.558	0.455	0.695	0.449	0.685
800	0.367	0.560	0.457	0.697	0.450	0.687
900	0.367	0.561	0.458	0.699	0.451	0.688
1000	0.368	0.562	0.459	0.701	0.452	0.689
1100	0.369	0.563	0.461	0.703	0.453	0.691
1200	0.370	0.564	0.462	0.705	0.454	0.692
1300	0.371	0.566	0.463	0.707	0.455	0.694
1400	0.372	0.567	0.465	0.710	0.456	0.695
1500	0.372	0.568	0.467	0.712	0.457	0.697
1600	0.373	0.569	0.468	0.714	0.457	0.698
1700	0.374	0.571	0.471	0.718	0.458	0.699
1800	0.375	0.572	0.474	0.723	0.459	0.701
1900	0.376	0.573	0.482	0.735	0.460	0.702
2000	0.377	0.575	0.501	0.764	0.461	0.704
2100	0.378	0.576	0.530	0.809	0.462	0.705
2200	0.379	0.578	0.911	1.390	0.463	0.707



Volume	Free Flow		Lane Closure		Reduced Speed	
	Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck
500	0.012	0.097	0.015	0.120	0.015	0.119
600	0.012	0.097	0.015	0.120	0.015	0.119
700	0.012	0.097	0.015	0.121	0.015	0.119
800	0.012	0.097	0.015	0.121	0.015	0.119
900	0.012	0.097	0.016	0.121	0.015	0.119
1000	0.012	0.098	0.016	0.122	0.015	0.120
1100	0.012	0.098	0.016	0.122	0.015	0.120
1200	0.013	0.098	0.016	0.122	0.015	0.120
1300	0.013	0.098	0.016	0.123	0.015	0.120
1400	0.013	0.098	0.016	0.123	0.015	0.121
1500	0.013	0.099	0.016	0.124	0.015	0.121
1600	0.013	0.099	0.016	0.124	0.015	0.121
1700	0.013	0.099	0.016	0.125	0.016	0.121
1800	0.013	0.099	0.016	0.125	0.016	0.122
1900	0.013	0.100	0.016	0.128	0.016	0.122
2000	0.013	0.100	0.017	0.133	0.016	0.122
2100	0.013	0.100	0.018	0.140	0.016	0.122
2200	0.013	0.100	0.031	0.241	0.016	0.123

**Table A-3 Operating Cost for Each Vehicle Type for Each Scenario (\$)**

Volume	Free Flow		Lane Closure		Reduced Speed	
	Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck
500	0.353	3.916	0.320	3.972	0.321	3.970
600	0.353	3.916	0.319	3.973	0.321	3.970
700	0.352	3.917	0.319	3.974	0.321	3.970
800	0.352	3.918	0.318	3.975	0.321	3.971
900	0.352	3.918	0.318	3.975	0.320	3.971
1000	0.351	3.919	0.317	3.976	0.320	3.972
1100	0.351	3.920	0.317	3.977	0.320	3.972
1200	0.350	3.920	0.316	3.977	0.319	3.973
1300	0.350	3.921	0.316	3.978	0.319	3.973
1400	0.349	3.921	0.315	3.979	0.319	3.974
1500	0.349	3.922	0.315	3.980	0.318	3.974
1600	0.348	3.923	0.314	3.981	0.318	3.975
1700	0.348	3.923	0.313	3.982	0.318	3.975
1800	0.347	3.924	0.312	3.984	0.317	3.976
1900	0.347	3.924	0.310	3.988	0.317	3.976
2000	0.347	3.925	0.307	3.998	0.317	3.977
2100	0.347	3.925	0.302	4.015	0.316	3.977
2200	0.346	3.926	0.280	4.207	0.316	3.978

**Table A-4 Road User Cost for Each Vehicle Type for Each Scenario (\$)**

Volume	Free Flow		Lane Closure		Reduced Speed	
	Passenger Car	Truck	Passenger Car	Truck	Passenger Car	Truck
500	302.95	388.35	326.78	406.48	325.50	405.57
600	363.65	466.14	392.53	488.05	390.80	486.83
700	424.50	544.11	458.68	569.91	456.28	568.21
800	485.43	622.16	524.88	651.79	521.92	649.70
900	546.29	700.13	591.09	733.70	587.44	731.11
1000	607.28	778.26	657.47	815.72	653.22	812.71
1100	668.42	856.55	724.14	897.94	719.18	894.43
1200	729.48	934.76	790.96	980.27	785.11	976.13
1300	790.73	1013.18	857.73	1062.57	851.28	1058.00
1400	852.01	1091.64	924.97	1145.20	917.56	1139.95
1500	913.41	1170.23	992.63	1228.12	983.90	1221.94
1600	974.71	1248.71	1059.88	1310.75	1050.23	1303.93
1700	1036.40	1327.43	1128.61	1394.44	1116.82	1386.10
1800	1098.29	1406.16	1197.91	1478.52	1183.37	1468.24
1900	1160.07	1484.82	1274.62	1566.69	1250.10	1550.50
2000	1222.11	1563.65	1368.47	1664.26	1317.00	1632.89
2100	1284.35	1642.64	1482.68	1772.23	1384.00	1715.35
2200	1346.89	1721.83	2231.48	2183.77	1451.45	1798.13