MODELING FREEWAY LEVEL-OF-SERVICE BASED ON TRAVEL TIME AND TRAVEL TIME RELIABILITY

by

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ABSTRACT

MD. SHAH IMRAN. Modeling freeway level-of-service based on travel time and travel time reliability. (Under the direction of DR. SRINIVAS S. PULUGURTHA)

The assessment of operational performance is a challenge for the local transportation agencies and Metropolitan Planning Organizations (MPOs) due to the dynamic nature in traffic movement. As demand approaches to the capacity of a roadway (or of the intersections along the road), extreme traffic congestion sets in. When vehicles are fully stopped for periods of time, this is colloquially known as a traffic jam or traffic snarl-up. A qualitative classification of traffic is often done in the form of a six letter A–F level-of-service (LOS) scale defined in the Highway Capacity Manual (HCM), a document widely used by engineers and planners in the U.S. (or used as a basis for national guidelines worldwide). LOS is the chief measure of "quality of service" which describes operational conditions within a traffic stream and is different for different facilities. These levels are used by transportation engineers and planners as shorthand to describe traffic levels to the lay public.

However, the current LOS criteria is based on density parameters along with some speed information for freeway sections, and service flow rate with speed information for highways, which do not always convey message to the general road users. In addition, unlike flow rate and speed information, density is not directly collected or readily available from field.

Travel time is one of the most important metrics used by the practitioners for decision making processes. It is also easily understood by road users and helps them

choose their routes to reach their destinations quickly. Real time continuous data collection is possible through the use of roadside Bluetooth detectors, on-road sensors, traffic cameras, or other technologies. Hence, if travel time related parameters can be established to denote the service level of freeways, the LOS would be readily available and the variation can easily be recorded over time. Further, travel time reliability has become an important concept for modern and urban transportation system managers. It is defined as “the consistency or dependability in travel times, as measured from day to day and/or across different times of the day” and a measure of the service provided by a transportation network.

This dissertation correlates the travel time and travel time reliability with density based LOS thresholds and identifies a more convenient and easily understood and usable LOS criteria based on such measures. A microscopic simulation model is developed, calibrated, and validated using the real world data. The calibrated parameters are used in several hypothetical microscopic simulation models representing different sections of freeway section types (i.e., basic freeway section, weaving section, and merging/diverging area) in order to develop a meaningful density – travel time and density – travel time reliability relationships and corresponding LOS criteria. Prior to developing the relationships and LOS criteria, the microscopic simulation based density values are compared with the HCM based densities in order to indicate the validity of the models.

The dissertation finds a strong correlation between HCM based densities and densities from VISSIM, which further validates the notion that a calibrated microscopic simulation model can be effectively used to represent general traffic behavior. The density – travel time per mile relationship shows a non-linear (exponential) relationship for all the freeway section types, which further questions the generic speed assumptions made by HCM for different LOS profiles. A polynomial relationship was observed between density – travel time reliability indices. It was found that average travel time per mile threshold values for respective LOS letters increase as the speed limit decreases until the condition comes close to saturation where the speed limit on the freeway does not have any influence on the operation. It can also be noted that as the posted limit decreases, the percent difference between the two respective adjacent travel time per mile threshold values also decreases.

The dissertation also finds that the average travel time reliability LOS threshold values for their respective LOS letters decrease for all freeway section types as the speed limit decreases. For PTI based thresholds, the 95th percentile travel time decreases as the speed limit decreases but the 5th percentile travel times remain relatively similar. In case of BTI based thresholds, the 95th and 50th percentile travel time values become closer as the speed limit decreases. For all freeway section types, the percent difference between two respective adjacent PTI LOS threshold values remains relatively similar with slight increase or decrease as the speed limit decreases. However the percent difference between two adjacent BTI threshold values tends to increase as the speed limit decreases. The dissertation also showed that based on the observation period (number of data points), the LOS estimation can differ significantly.

Overall, this dissertation provides important insights on a more convenient and easily understandable approach to define freeway LOS and provides baselines for future researchers to investigate and develop the method further.

DEDICATION

I dedicate my dissertation work to my family and many friends. A special feeling of gratitude to my loving parents, Md. Shahjahan Miah and Shahnaj Parvin whose words of encouragement and push for tenacity still ring in my ears. My sister Noor Jannat has never left my side and is very special to me. I will always appreciate all they have done for me.

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LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ANOVA	analysis of variance
BTI	buffer time index
C–D	collector–distributor
CTR	Chicago Test bed for Reliable Routing
DOT	Department of Transportation
DOW	day of week
DTM	Design Traffic Memorandum
FDOT	Florida Department of Transportation
FFS	free flow speed
FHWA	Federal Highway Administration
HCM	Highway Capacity Manual
HCS	Highway Capacity Software
ITS	Intelligent Transportation System
LOS	level-of-service
MOE	measures of effectiveness
MPO	Metropolitan Planning Organization
MrSID	Multi-resolution Seamless Image Database
O-D	origin-destination
ODOT	Oregon Department of Transportation
pc/mi/ln	passenger cars per mile per lane
PI	Policy Index

PTI	planning time index
RTP	Regional Transportation Plan
TOD	time-of-the-day
TRB	Transportation Research Board
TTI	travel time index
v/c	demand flow rate to capacity
VMT	vehicle miles traveled
VOR	value of reliability

CHAPTER 1: INTRODUCTION

1.1. Introduction

The transportation system is one of the national economic pillars providing mobility to the community. It has also been recognized as one of the sectors to evaluate the national developmental achievements. Therefore, many studies have addressed the issue with performance improvement of transportation network as a primary goal.

From 1950, results from many travel demand studies have been applied extensively in transportation system analyses. Most of these studies conducted based on the assumption that transportation infrastructure, environment, and human factors are harmoniously interconnected so that the transportation system can run and operate with complete control and in good condition (Susilawati et al., 2010). However, transportation system is a complex and dynamic system. Many factors can adversely affect transportation network performance. These include travel demand as well as short term or long term incidents, which lead to higher travel time variability and related consequences to the community (Nicholson et al., 2003). At the same time, just-in-time services of mobility, especially for goods transportation, has forced practitioners toward considering transportation system reliability as a measure for a robust transportation system. Therefore, in the last two decades, transportation system reliability has become a strong interest for research (Bell and Iida, 2003).

There are several definitions of the transportation system reliability depending on the measurement context. Transportation system reliability focuses on the probability of successful completion of a trip at a specified time (travel time reliability), based on the remaining connectivity between an origin–destination (O–D) pair (connectivity reliability), and/or at certain level of the link capacity (capacity reliability) (Bell and Iida, 2003). As the need for more reliable transportation system has forced transportation planners to study several aspects of transportation system reliability, the travel time reliability has been commonly accepted as the key indicator in assessing the transportation system performance. It is also an important factor in route choice analysis. The better understanding of travel time reliability and variability might help relieve the congestion problems and lower the impact of different type of incidents (Recker et al., 2005). Therefore, the more reliable the transportation system is, the more stable is its performance ensuring more predictable trip times and improved activity schedules. Reliable travel time also has a significant effect on improving the freight industry's performances on delivering goods (Recker et al., 2005).

According to Lomax et al. (2003), the travel time reliability is the level of consistency of transportation services for a mode, trip, route, or corridor for a time period. The travel time reliability also refers to the idea of traffic performance, particularly from the viewpoint of road users. Previous research investigated the road user's disutility of being late and arriving early due to the unreliable travel time and developed disutility model based on the logit choice model by applying the amount of the extra travel cost needed for the extra travel time (Small et al., 1999; Noland and Polak, 2002; Bates et al., 2001).

Many recent studies have investigated the effect of unreliable travel time on traveler's behavior by examining traveler behavior and their response to the provision of travel information. According to Susilawati et al. (2010), provision of accurate and reliable information can improve the transportation network reliability. They also concluded that Stochastic User Equilibrium (SUE) model can generate the route choice behavior of road users based on the different levels of information provision. According to Tannabe et al. (2007), the greater variance of travel time of selected links makes the links less attractive to the road users. Bogers and Lint (2007) also found that provision of traveler information has a significant impact on road user's decision and users would choose the route with minimal travel time variance based on their experience. Tannabe et al. (2007) also identified that road users change their routes to reduce the uncertainty of travel time.

The Highway Capacity Manual (HCM) historically has been among the most important reference guides used by transportation professionals, which is useful for planning, design, preliminary engineering, and operations analysis (TRB, 2010). It provides analytical concepts for characterizing traffic flow, capacity, quality-of-service (QOS), and level-of-service (LOS), which are useful measures for the transportation planners seeking a systematic basis for evaluating elements of the surface transportation system – particularly highways, but also other modes. The HCM provides guidance on analyzing facilities, segments, and points for uninterrupted-flow roadways, such as freeways and multilane highways. It distinguishes between capacity, defined as the hourly flow rate, and other performance measures, such as density, speed, delay, number of stops, queue length, and volume-to-capacity ratio (Kittelson and Vandehey, 2013).

Though travel time and travel time reliability are increasingly recognized as important mobility performance measures, the HCM lacks a method to address these mobility performance measures for specific types of facilities such as freeways, multilane highways, and urban corridors. The HCM provides the LOS on freeways based on density in terms of passenger cars per mile per lane (pc/mi/ln) on the freeway segments. However, traffic movement has dynamic characteristics and density calculation is not readily available from field, which makes it difficult to track the performance of the freeways based on the current method. Therefore, travel time and/or travel time reliability based performance measurement is a great concept to replace the existing method.

1.2. Background and Problem Statement

1.2.1. Definition of Travel Time Reliability

The travel time reliability better quantifies the benefits of traffic management and operation activities than simple averages. “Reliability” itself may have little meaning to the traveling public. Road users have their own perception of travel time reliability based on their own daily experiences. However, there is a lack of knowledge regarding the nature of useful reliability information, the procedure to express them, the effect of such information on the travelers’ route choice, and system performance in terms of recurring and non-recurring highway congestion.

According to the Florida Department of Transportation (FDOT) (2000), travel time reliability is “the consistency or dependability in travel times during a specific period of time under stated conditions”. This consistency depends on travel time threshold resulting from the impact of the influencing factors. The threshold is used as a measurement of travel time reliability and typically represents the addition of the extra time (or cushion time or buffer time) to the average travel time to ensure on-time arrival of the travelers. As such, “reliable” segments are those on which travel time threshold is equal to or lower than the extra time added to the average travel time on that segment.

Therefore, reliability is concerned with three key elements of this definition. Firstly, it is a probability concerned with achieving consistency or dependability at a statistical confidence level. Secondly, reliability applies to a defined threshold and specific time periods. Lastly, it is restricted to operation under stated conditions as it is impossible to design a system for unlimited conditions.

1.2.2. Freeways and Travel Time Reliability

According to Federal Highway Administration (FHWA) (2014), a freeway is “a divided highway facility with full control of access and two or more lanes for the exclusive use of through traffic in each direction”. The interstate freeway network was authorized by the Federal Aid Highway Act of 1956, and the original portion was completed 35 years later. However, the network has since been extended, and as of 2012, it had a total length of 47,714 miles (76,788 km) (OHPI, 2013a). As of 2011, about one-quarter of all vehicle miles driven in the country used the Interstate system (OHPI, 2013b). All the freeways were originally not designed to accommodate today’s heavy traffic. Instead, they have evolved as urban and suburban traffic has increased. Consequently, congestion has grown substantially over the past 30 years in every city, becoming one of the key urban problems (TTI, 2011). The congestion level in 85 of the largest metropolitan areas in U.S. has grown almost every year from 1982 to 2010. The average yearly delay endured by the average commuter was 34 hours in 2010 compared to the 14 hours in 1982, which triggers over a $100 billion loss (nearly $750 for every commuter) due to congestion (TTI, 2011).

This trend is expected to continue as U.S. becomes increasingly urbanized. The increasing congestion levels have influenced travel time reliability impacting all the transportation system users, such as vehicle drivers, transit riders, freight shippers, and even air travelers. However, the unreliability of travel times forces travelers to plan for these problems by leaving early just to avoid being late. Therefore, extra time out of everyone's day is being devoted to travel; even if it means getting somewhere early which could have been used for other endeavors.

Charles (2008) pointed out that traveler’s willingness to pay extends to reliability of travel time, especially for time-sensitive trips. He referred the willingness to pay for reductions in the day-to-day travel time variability as “value of reliability” (VOR). According to the report, some U.S. studies have found that travelers put twice as much value on travel time variability than on the average travel time. In addition, in terms of travel time certainty and travel time reductions due to reduced average trip time, reliability has an indirect impact on trip costs by potentially reducing fuel consumption, vehicle emissions, and public transport operating costs (Charles, 2008).

Therefore, it is important to understand the factors influencing travel time reliability on freeways. Road agencies and authorities are interested to address these factors. The reliability measure should provide useful information on the total time budgeted for a trip. The computation process for such measure should control for variations that are non-relevant to the trip planning decision, although these elements will vary. This may include factors such as day-to-day and time-of-the-day (TOD) variations due to the decision making using prior knowledge of the day, and time and variation in road characteristics as travelers typically consider their trip travel time than each road section separately (Lomax et al., 2003).

1.2.3. Future of Travel Time Reliability in Transportation Sector

The Federal Highway Administration (FHWA) projects a 65% growth in domestic freight volumes between 1998 and 2020. This rapid growth in truck volume can be attributed to a number of factors, such as the shift of significant freight activity from rail and other modes to truck, and the changes in the economy and business practices, such as just-in-time deliveries of inventory items that increase delivery frequencies (Polzin, 2006). Therefore, it is expected that there will be a three percent annual growth in truck vehicle miles traveled (VMT) (FHWA, 2013).

In addition to that, e-commerce is advancing significantly and will influence the land use patterns and VMT over the next few decades. The home based shopping via catalogs, cable television shows, and the internet, and highly efficient package delivery companies, both private and public entities, will increase trips from local businesses to homes. There is also an expected shift in the shipment procedure which would put more emphasis toward less-than-truck load or smaller truck freight shipments than long-haul carriers as a significant portion of all types of retailing required next-day delivery, same-day delivery, and just-in-time delivery.

Furthermore, the current demographic shifts as well as those likely to occur in the future will also generate more traffic on U.S. urban roadways that will, in turn, increase the congestion level. The U.S. Census Bureau projects the U.S. population will be somewhat better off economically, with smaller households and increased household vehicle ownership (Bonnaire, 2012). In the coming years, the older driver population on the road is expected to be at least double in number, which is attributable to both the overall increase in the older population and the anticipated trend for older women to drive in greater proportions than their previous cohorts (Pisarski, 2006). Therefore, the shift in these household composition, labor force participation and household income changes, and shifts in licensing and vehicle ownership will affect transportation and individual mobility, which is expected to increase the highway VMT by 60% in 2020 (Bonnaire, 2012).

Concurrently, researchers and practitioners are well aware of the impacts of travel time reliability and therefore, consequently have adjusted their methodologies. For instance, in transportation planning, it is found that VOR significantly enhances the mode choice models (Pinjari and Bhat, 2006; Liu et al., 2007). The second Strategic Highway Research Program (SHRP2) identifies travel time reliability as one of the four transportation factors that needs to be addressed during a highway capacity expansion decision making process (Cambridge Systematics et al., 2009).

Travel time reliability research is developing the means for state Departments of Transportation (DOTs) and Metropolitan Planning Organizations (MPOs) to fully integrate mobility and reliability performance measures and strategies into the transportation planning processes. Studies are under way to include reliability factors into the HCM. In addition to that, a guide on roadway design features will be written to support the reduction of delays that in turn, reduce travel time reliability. Such features can be considered for inclusion in the American Association of State Highway and

Transportation Officials (AASHTO) Policy on Geometric Design of Highways and Streets (TRB, 2011).

However, reliability requirements for personal trips vary considerably. The factors include the type of trips (commuter, personal, and social/recreational), TOD (peak versus off-peak period), and the travel setting and conditions. In addition, reliability requirements vary based on the roadway network used, geographic areas (urban or rural), and the factors that contribute to the uncertainty of arrival time, such as traffic crashes or work zones.

Reliability requirements for business trips (freight carriers, shippers, and truckers) vary by situation and business characteristics. Therefore, it is important that transportation agencies must understand these different user requirements if they expect to meet them effectively. As pointed out by Transportation Research Board (TRB): "….Actions taken by transportation agencies to reduce congestion should effectively improve travel time reliability. To assure the effectiveness of those actions, the user requirements regarding travel time reliability must be understood. Different users of the highway network have different requirements for travel time reliability. Moreover, the requirements of each user depend on the situation. A trucker faced with just-in-time delivery has different travel time reliability requirements than an empty backhaul of a mom-and-pop trucking business. Service level agreements for just-in-time delivery can impose severe penalties for not being on time." (TRB, 2011).

1.2.4. Travel Time Reliability as a Measure of Service

In addition to the assessment of the traffic performance, Chen et al. (2003) and Lyman and Bertini (2007) have examined the application of the travel time reliability measurement. As the typical LOS method does not reflect the user's experience during their trip, Chen et al. (2003) discussed the use of travel time reliability in place of LOS. Lyman and Bertini (2007) investigated the travel time reliability measure to quantify the congestion. They analyzed twenty Regional Transportation Plans (RTPs) and found that no RTP used the travel time reliability measure as a congestion measure. The study examined five minute interval data to compare the buffer time index (BTI), the travel time index (TTI), and the planning time index (PTI) for three consecutive years of daily travel time along the Portland highways. They found that even though the three travel time reliability indices gave the same pattern along the roadway, the PTI gave higher index than the other two. Therefore, they compared only the BTI and the TTI to give the priority for the congestion relief through incident response systems, bottleneck improvements, and better traveler information and proposed ranking system to select the highest priority corridor. According to Tannabe et al. (2007), the appropriate functional hierarchy of road may be disturbed by the travel time uncertainty. These findings suggest that a reliability index of travel time is a very useful and important measure to evaluate both actual LOS and functional hierarchy of roadway network.

1.3. Research Objectives

The primary goal of this dissertation is to develop an adaptive freeway LOS method based on the best available travel time related statistics as an alternative to the current density based LOS method resulting in a useful tool from both general road user and planner perspectives. The primary objectives are as follows:

- Research and develop a microscopic simulation model based on field data,
- Perform model calibration and validation by comparing field data,

- Develop and simulate hypothetical freeway sections for analysis and evaluation,
- Compare the HCM based densities and microscopic simulation model generated densities to verify the approach,
- Develop density – travel time (per mile), density – speed and density – travel time reliability relationships,
- Evaluate merging and diverging areas separately,
- Establish criteria for travel time and travel time reliability LOS thresholds, and
- Apply and illustrate the use of established LOSs.

1.4. Importance of the Dissertation

Density, often expressed in pc/mi/ln, is essentially the number of passenger cars at any timestamp within a roadway segment. Getting this information periodically is not only costly but in many cases not feasible. Therefore, though the density value may provide an understanding of the roadway condition at that specific timestamp, it fails to identify the situation beforehand and afterwards. The density calculation also deals with a number of factors. For example, to calculate the weaving section density, the planner needs to know the O–D patterns of the area (weaving and non-weaving movements) to identify the total number of passenger cars entering and leaving through ramps between two timestamps. Collecting all the information is costly and time consuming. In addition, transportation system has evolved a lot since the inception of density based LOS ideology. Further, the modernization of vehicular operation and inception of newer technologies in the transportation field force the decision makers to look for a quick and easy way to understand the operational situation and disseminate the information for the betterment of the network flow.

Therefore, this dissertation discusses the potentiality of travel time and travel time reliability to be used as a measure of freeway operational condition. This is a solution for providing the LOS based on travel time characteristics, which is not only easier to obtain but also provides continuous data collection ability and track the network operation in a continual way.

The establishment of reliability metrics criteria to determine the freeway LOS is still developing; therefore, this dissertation will provide important insights on this approach. It is expected that this travel time reliability based freeway LOS method will be useful to:

- evaluate strategies and tactics to satisfy the travel time reliability requirements of travelers on the urban freeway network,
- monitor the performance of the networks,
- evaluate future roadway improvement options, and
- provide guidance on planning, geometric design, and traffic operations features.

1.5. Organization of the Dissertation

The rest of this dissertation is organized as follows. In the following chapter, previous works in freeway travel time reliability and travel time reliability measures are reviewed. This chapter also includes a discussion on other reliability based LOS methods. Chapter 3 discusses the research method on identifying the procedures to obtain freeway LOS based on travel time metrics. Chapter 4 describes the relationship between density and travel time experienced by the drivers using a calibrated microscopic simulation model. Chapter 5 examines the relationship between density and travel time per mile based on freeway speed limit and number of lanes available. The chapter also discusses how the travel time reliability indices can be used to define the freeway LOS. The last chapter of the dissertation, Chapter 6, concludes based on the findings, discusses limitations of the dissertation, and provides future recommendations.

CHAPTER 2: LITERATURE REVIEW

2.1. Introduction

The previous chapter described the problem statement and research goal and objectives. This chapter discusses previous research and literature on freeway travel time reliability, the techniques to quantify such statistics, and available knowledge on how this information can be used to successfully identify a measure of service (LOS).

2.2. Previous Studies on Freeway Travel Time Reliability

Clark and Watling (2003) proposed a method for estimating the probability distribution of total network travel time considering normal day-to-day variations in the travel demand matrix over a transportation network. They proposed a solution method based on a single run of a standard traffic assignment model. Moments of the travel time distribution were computed using an analytic method based on the multivariate moments of the link flow vector. A flexible family of density functions was fitted to these moments. The researchers also discussed how the resulting distribution in practice may be used to characterize unreliability. They found the method to be effective in identifying sensitive or vulnerable links and examining the impact on network reliability of changes to link capacities.

According to Tu et al. (2005), weaving sections can lead to certain variations in travel time due to intense lane changing maneuvers and complex vehicle interactions. The length of the weaving section is the primary factor for such variability. Therefore, the researchers investigated the relationship between them using both a simulation approach and based on empirical data. Both procedures indicate a relationship between weaving section length threshold and travel time variability increase. The implications call for possible control applications to reduce the travel time variability in the short weaving sections.

Van Lint and van Zuylen (2005) proposed many different aspects of the day-to-day travel time distribution as indicators of reliability. Both mean and variance of a distribution tend to obscure important aspects of the distribution under specific circumstances. The researchers argued that both skewness and width of this distribution are relevant indicators for unreliability. They proposed two reliability metrics based on three characteristic percentiles (10th, 50th, and 90th percentiles) for a given route and TOD-day of week (DOW) period. High values of either metric indicate high travel time unreliability, while the weight of each metric on travel time reliability may be application or context specific. These metrics can be used to construct reliability maps in order to visualize the unreliability of travel times for a given TOD – DOW period and help identify TOD – DOW periods in which congestion will likely set in (or dissolve). The overall process can identify the uncertainty of start and end moments; and hence, length of morning and evening peak hours. The metrics can be used to predict travel time (un)reliability if combined with a long-term travel time prediction model and also may be used in discrete choice models as explanatory variables for driver uncertainty.

Nam et al. (2005) expressed reliability in terms of standard deviation and maximum delay measured based on triangular distribution. The researchers used the multinomial and Nested Logit models to estimate value of time and VOR. They found that reliability is an important factor affecting mode choice decisions. As reliability has higher values than that of time, the researchers noted that the policy to increase travel time reliability has more benefit than to reduce the travel time at the same level of improvement.

According to Al-Deek and Emam (2006), travel time reliability captures the variability experienced by individual travelers and can indicate the operational consistency of a facility over an extended period. A roadway segment's reliability is considered 100% if its travel time is less than or equal to the travel time at the posted speed limit for that segment. They only considered the weekdays as weekends had different peak periods. The researchers noted that the freeway corridor consists of a collection of links arranged and designed such that they achieve desired functions with acceptable performance and reliability. However, the relationship between the freeway corridor system reliability and its link reliability is often misunderstood. For example, all of the links in a system having 95% reliability at a given time does not mean the overall reliability of the system is 95% for that time.

Elefteriadou and Xu (2007) developed models for estimating the travel time reliability on freeway based on four factors (congestion, work zones, weather, and incidents) that may affect travel time. Sumalee and Watling (2007) proposed a partition-based method to evaluate the transport network from the view point of travel time reliability after any disaster. Their algorithm helps classifying the network states into reliable, unreliable, and un-determined partitions. Each reliable and/or unreliable state can be used to determine a number of other reliable and/or unreliable states without evaluating all of them with an equilibrium assignment procedure by postulating the monotone property of the reliability function. A cause-based failure framework was also proposed to represent dependent link degradation probabilities and tested with a medium size test network to illustrate the performance of the algorithm.

Shao et al. (2007) proposed a travel time reliability-based traffic assignment model in order to identify the rain effects on risk-taking behaviors of travelers considering day-to-day demand fluctuations and variations in travel time. The researchers used a Logit-based stochastic user equilibrium framework to incorporate traveler perception errors on travel time and risk-taking behavior on path choices into the model.

Lyman and Bertini (2008) examined the use of measured reliability indices for the improvement of real-time transportation management and traveler information using archived Intelligent Transportation System (ITS) data. The researchers tested several reliability measures including travel time, 95^{th} percentile travel time, TTI, BTI, PTI, and congestion frequency. They used the BTI to prioritize freeway corridors and concluded that MPO should use travel time reliability by incorporating it as a system-wide goal, evaluating roadway segments according to travel time reliability measures, and prioritizing the capacity expansion of roadway segments using these measures.

Tu et al. (2008) proposed a new analytical formula to express travel time unreliability in which the travel time (un)reliability is computed as the sum over the products of the consequences (variability or uncertainty) and corresponding probabilities of traffic breakdown (instability). Their proposed travel time reliability model is considered as a function of a variety of conditional factors under certain circumstances, such as road characteristics, traffic control measures, prevailing traffic state (congested or not), and possible external factors, such as weather and luminance. Empirical data were used to validate and calibrate the model. The researchers noted that with the increase in inflows, both the probability of traffic breakdown and travel time unreliability increase.

Pu (2010) analyzed the reliability measures, including the 90^{th} or 95^{th} percentile travel time, standard deviation, coefficient of variation, BTI, TTI, PTI, skew statistic, misery index, frequency of congestion, on-time arrival, and others to explore their mathematical relationships and interdependencies. The researcher assumed lognormal distributed travel times and by using a percent point function, a subset of reliability measures were expressed in terms of the shape parameter and/or the scale parameter of the lognormal distribution. Contrary to some previous studies and recommendations, the researcher concluded that the coefficient of variation, instead of a standard deviation, is a good proxy for a number of other reliability measures, including PTI, median-based buffer index, and skew statistic. Pu (2010) also recommended using the median based buffer index and failure rate than that of an average based.

Pulugurtha and Pasupuleti (2010) developed and illustrated the working of a method to estimate travel time and its variations, travel delay index due to crashes and their severity, congestion score and reliability of each link in the network. Traffic volume, link capacity, travel speed, crashes and their severity, and estimated time taken for normal traffic conditions to restore after a crash were used in the computations. Sensitivity analysis was also conducted to examine and assess reliability of links based on variations in weights to integrate recurring and non-recurring congestion components.

Nie et al. (2010) enhanced travel reliability of highway users by providing them with reliable route guidance produced by newly developed routing algorithms. The algorithms were validated and implemented with real traffic data. Phase I of the project focused on demonstrating the value of reliable route guidance by developing and disseminating Chicago Test bed for Reliable Routing (CTR). Phase II aimed at bringing the implementation of reliable routing technology to the next stage through initial deployment of CTR. Rakha et al. (2011) examined existing studies that had used video cameras and other on-board devices to collect data. They determined the potential for using such data to explore driver behavior in order to reduce non-recurring congestion and hence, the travel time unreliability.

Cambridge Systematics (2013) analyzed the effects of non-recurring congestion, such as incidents, weather, work zones, special events, traffic control devices, demand, and bottlenecks. Their study explained the importance of travel time distributions for measuring reliability, and recommended specific reliability performance measures. Numerous non-recurring congestion mitigation procedures were identified and models to predict such events were developed with an indication of their relative importance. The models were based on three empirical methods, before and after studies, a "data poor" approach that resulted in a parsimonious and easy-to-apply set of models, and a "data rich model" that used cross-section inputs including data on selected factors known to directly affect non-recurring congestion. The study found that travel time reliability can be improved by reducing demand, increasing capacity, and enhancing operations.

Although these recent studies on the topic provide reasonable methodologies for quantifying travel time reliability, there is a lack of consensus on the suitable reliability based approach.

2.3. Travel Time Reliability Measures

Cambridge Systematics and TTI (2005) suggested various indices regardless of the source or the type of variability, which are commonly divided into statistical measures, buffer measures, and tardy trip indicators. Statistical measures, such as travel time window and percent variation (Equations 2.1 and 2.2) focus on estimating standard deviation of travel times and comparing it with the average travel time.

$$\text{Travel Window} = \text{Average Travel Time} \pm \text{Standard Deviation} \tag{2.1}$$

$$\text{Percent Variation} = \frac{\text{Standard Deviation}}{\text{Mean}} \text{ x } 100\% \tag{2.2}$$

Though these statistical measures provide the extent of unreliability to professionals, it is difficult for individuals to apply the concept of standard deviation to their individual travel time. In addition, the variation due to different events separately is difficult to comprehend for the individuals.

The second category of methods is buffer measures. BTI is the most commonly used index, which represents a measure of trip reliability by expressing the amount of extra buffer time needed to be on time for 95% of the trips. This measure allows the individuals to estimate the extra percent of travel time for the trip due to varying congestion level.

$$\text{BTI} = \left[\frac{\text{95th percentile Travel Time} - \text{Average Travel Time}}{\text{Average Travel Time}}\right] \text{ x } 100\% \tag{2.3}$$

The PTI can also be used, which estimates the extent by which the free-flow travel time will be exceeded.

$$\text{PTI} = \left[\frac{\text{95th percentile Travel Time}}{\text{Travel Time based on Free Flow Speed}}\right] \text{ x } 100\% \tag{2.4}$$

Both of these indices allow individuals to plan a trip to arrive on time in a vast majority of situations. The 95^{th} percentile travel time ensures that the road user is only late 1 out of every 20 trips. These buffer measures can be used to calculate a single value of reliability for the road segment or different values depending on TOD and DOW.

Tardy trip indicators, which include percent of unreliable trips and misery index, are the third way to evaluate the variability in the travel time. The percent of unreliable trips is simply evaluated as the percent of trips with higher than acceptable travel times. The misery index is calculated as the average travel time subtracted from travel time from the top 20% of trips divided by the average travel time.

$$\% \text{ on Time} = \text{Percent Trip Times} < [1.1 * \text{Mean Time}] \tag{2.5}$$

$$\text{Misery Index} = \frac{\begin{array}{c}\text{Average Travel Time for the Longest 20\% of Trips} \\ - \text{Average Travel Time}\end{array}}{\text{Average Travel Time}} \tag{2.6}$$

FHWA (2013) defines frequency of congestion as the frequency when congestion exceeds some expected threshold. This index is typically expressed as the percent of days or time that travel times exceed X minutes or travel speeds fall below Y mph. In case continuous traffic data is available, the frequency of congestion measure is relatively easy to compute. It is typically reported for weekdays during peak traffic periods.

Standard deviation is a widely employed measure of variability or diversity in statistics and probability theory. It shows how much variation or "dispersion" there is from the average (mean or expected value) and is sometimes used as a proxy for other reliability measures. It is, however, a convenient measure when calculating travel time reliability using classical or statistical models (Dowling et al., 2009). The standard deviation treats both late and early arrivals with equal weight while the public cares much about late arrival. Therefore, it is not either easily related to everyday commuting experiences.

$$\text{Standard Deviation} = \sqrt{\frac{\sum(\text{Each Value in the Data Set} \\ -\text{Average Value in the Data Set})}{\text{Number of value in the Data Set}}} \tag{2.7}$$

Coefficient of variation is a ratio of standard deviation to the mean. It also has the same disadvantages as the standard deviation.

$$\text{Coefficient of Variation} = \frac{\text{Standard Deviation}}{\text{Average Travel Time}} \tag{2.8}$$

The standard deviation to an average value combined in a ratio is referred to as percent variation in the 1998 California Transportation Plan (Guo et al., 2010). This is the form of the statistical measure – coefficient of variation. Though the percent variation is expressed as a percentage of average travel time, it is easily understandable (Pu, 2010).

$$\text{Percent Variation}=\frac{\text{Standard Deviation}}{\text{Average Travel Time}} \text{ x } 100\% \tag{2.9}$$

Failure rate or percent of on-time arrival estimates the percentage of time that a traveler arrives on time based on an acceptable lateness threshold. The threshold travel time to determine an on-time arrival ranges from 110% to 113% of average travel time.

$$\text{Failure Rate} = 100\% - \text{Percent of On-Time Arrival} \tag{2.10}$$

Florida DOT’s reliability method uses a percentage of the average travel time in the peak to estimate the limit of the acceptable additional travel time range (FDOT, 2014). The sum of the additional travel time and the average time define the expected time and the travel times longer than this expected time would be termed as “unreliable”. However, this computational method has its disadvantage of using travel time rather than travel rate, while travel rate variations provide a length-neutral way of grading the system performance providing the provision of easy transmission to travelers (Lomax et al., 2003).

Florida Reliability Statistic (% of Unreliable Trips) =100% – (Percent of trips with travel times greater than expected) (2.11)

In addition to the statistical methods of estimating travel time reliability, Elefteriadou (2006) proposed econometric modeling. She developed linear regression models to estimate average travel time for scenarios with different combinations of weather, accidents, congestion and work zones. Furthermore, the researcher determined probability of a reliable trip under various definitions of reliability. This study was unable to provide models for all scenarios due to the lack of data and did not clarify how the travel times were obtained for modeling.

TTI (2005) suggested a threshold of 10% higher than the average travel time (or travel rate) for travel time reliability. However, the 10% late arrival has the disadvantage of being relatively conservative for some applications. Clark and Watling (2005) used the probability distribution of the actual values of the performance measure to define unreliability. The planning state occurs when the performance measure equals the mode of around 1; the critical value is defined as a tolerance of 400 percent above the performance measure value in the planning state, yielding to a critical value of 5. Afterwards, the unreliability is defined, for instance, in terms of the probability of exceeding the critical value Pr ($M > 5$), i.e., the area under the curve in the range labeled "degraded performance" (Figure 2.1). Thus in percentage terms, the reliability is

$$\rho = (1 - \Pr(M > 5)) \times 100\% \quad (2.12)$$

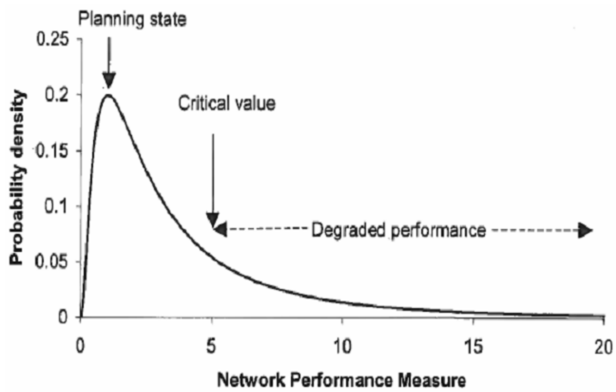


FIGURE 2.1: Performance measure distribution (TTI, 2005)

SHRP 2 Project L03 (Cambridge Systematics et al., 2013) examined the potential performance measures used to describe travel time reliability, including the skew statistic as proposed by European researchers and some other parts of the world. In addition, the researchers added the 80th percentile TTI because analysis indicated that this measure is especially sensitive to operational improvements and has also been used in previous studies on the valuation of reliability. The travel time distribution can enhance the difference between all of these measures, as illustrated in Figure 2.2. In SHRP 2 Project L03, the researchers used TTI as the variable of interest which also satisfies the need to normalize travel time. Therefore, the base distribution is actually based on the distribution of the TTI, rather than raw travel times.

The L03 research (Cambridge Systematics et al., 2013) also demonstrated that the BTI can be an unstable measurement for tracking trends over time because in part of its linkage to two factors (average and 95th percentile travel times) that change. If one changes more in relation to the other, counterintuitive results can appear. Even though SHRP 2 Project L03 did not define standard deviation of travel time or travel rate as a reliability performance metric, it has been added due to its inclusion in several other

SHRP 2 research projects. SHRP 2 Project L03 also included predictive methods for the standard deviation, even though it was not formally identified as a useful performance measure due to the difficulty in explaining it to non-technical audiences.

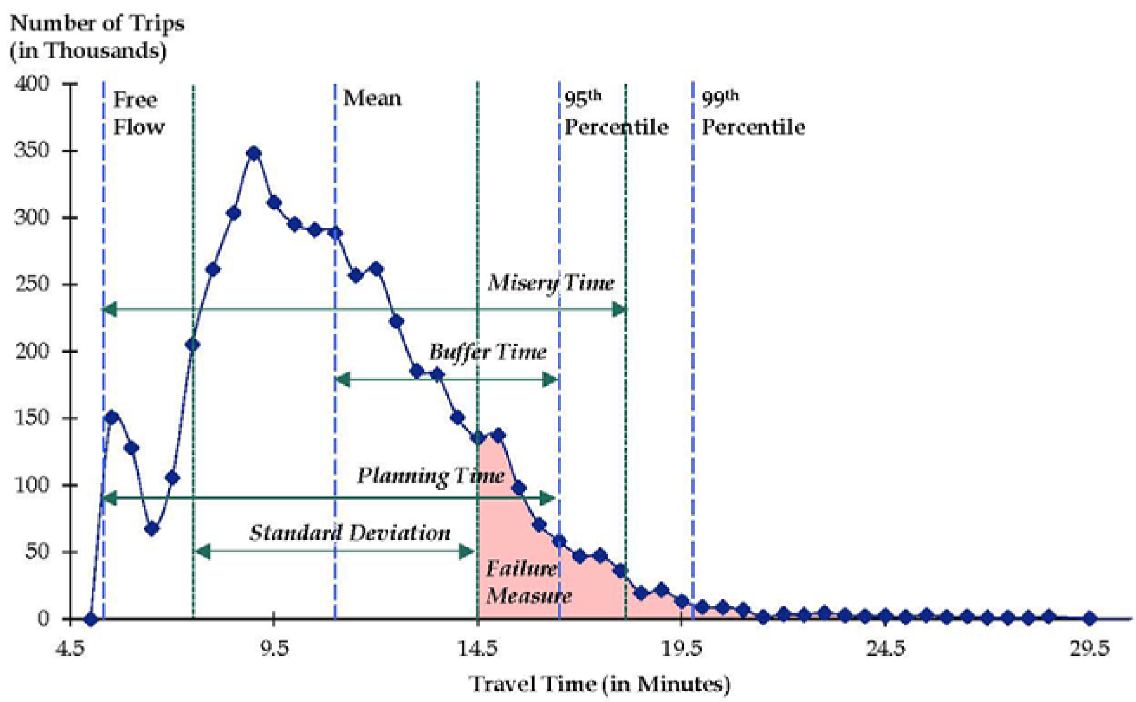


FIGURE 2.2: Travel time distribution for defining travel time reliability metrics (Kittelson and Vandehey, 2013)

While various travel time percentiles historically have been used for the TTI, SHRP 2 Reliability Project L08 (Kittelson and Vandehey, 2013) recommended that the 80th percentile highest travel time be used for the predicted travel time due to its more stable relationship with mean travel time than the 90th, 95th, or 99th percentiles. This is useful in predicting changes in reliability based on changes in the mean travel time.

The formula for computing a system-wide TTI is given below.

$$80\%\ \text{TTI} = \frac{\text{VHT (80\%)/ VMT(80\%)}}{\text{VHT (FF)/VMT(FF)}} \tag{2.13}$$

where,

80% TTI = 80th percentile Travel Time Index,

VHT (80%) = 80th percentile highest vehicle-hours traveled among the scenarios evaluated,

VMT (80%) = Vehicle miles traveled for scenario with 80th percentile highest vehicle-hours traveled among the scenarios evaluated,

VHT (FF) = Vehicle-hours computed with segment free-flow speeds, and

VMT (FF) = Vehicle miles traveled with segment free-flow speeds.

The SHRP 2 Reliability Project L02 (ITRE et al., 2013) recommends using the probability density function and cumulative density function of travel time rate in seconds per mile as the primary reliability measure to identify the reliability performance under different regimes and influencing factors. The SHRP 2 Project L02 also utilizes the semi-variance to determine the unreliability contribution factors including high demands, bad weather, and incidents (Table 2.1).

The SHRP 2 Reliability Project L08 (Kittelson and Vandehey, 2013) also discussed the use of the policy index (PI) based on the agency's congestion management goal of operating its freeways at a certain speed (for example, 40 mph). The PI is computed based on the agency's target speed in place of the free-flow speed, also defined in Table 2.1. As the agency wants to maintain the mean annual peak period speed on the facility to be at the target speed or higher, the PI value over 1.00 means the reliability of the facility will be considered unacceptable (Kittelson and Vandehey, 2013).

TABLE 2.1: Reliability performance measure utilization in SHRP 2 projects (Hadi et al., 2014)

Reliability Performance Metric	**Description**	**Measure Utilization**
Buffer Time Index (BTI)	The difference between the 95th percentile travel time and the average travel time, normalized by the average travel time	L03, L08
Failure/On-Time Performance	Percentage of trips with travel times less than: • 1.1 * median travel time • 1.25 * median travel time Or percentage of trips with speed less than 50, 45, 40 or 35 mi/h	L03, L08
95th Planning Time Index (PTI)	95th percentile of the travel time index distribution (95th percentile travel time divided by the free-flow travel time)	L03, L08
80th Percentile Travel Time Index (TTI)	80th percentile of the travel time index distribution (80th percentile travel time divided by the free-flow travel time)	L03, L08
Skew Statistics	The ratio of 90th percentile travel time minus the median travel time divided by the median travel time minus the 10th travel time percentile	L03
Misery Index	The average of the highest 5% of travel times divided by the free-flow travel time	L03
Probability Density Function of Travel Time Rate	Probability density function of travel time rate distribution	L02
Cumulative Density	Function of Travel Time Rate Cumulative density function of travel time rate	L02
Semi-Variance	The variance of travel time rate (in sec/mi) pegged to the free-flow travel time instead of the mean travel time	L02
Standard Deviation	Usual statistical definition	L08
Kurtosis	Usual statistical definition	L08
Reliability Rating	Percentage of VMT at a TTI less than certain threshold (for example, 1.33 for freeway and 2.5 for urban streets)	L08
Policy Index	Mean travel time divided by travel time at target speed	L08
Semi-Standard Deviation	One-sided standard deviation that is referenced to the free-flow travel time	L08

The SHRP 2 Reliability Project L05 (Cambridge Systematics, 2013) encourages agencies to estimate multiple reliability performance measures. Different measures capture different aspects of the travel time distribution, which in turn, may suggest different strategies to employ. The L05 project mentioned that "reliability is complex and its proper measurement requires multiple metrics." Furthermore, the project document reported that "the use of multiple measures provides a clearer picture as to the size and shape of the travel time distribution. It can be confusing to interpret multiple reliability performance metrics. Some metrics may appear to indicate improvement in reliability between alternatives, while others may not". SHRP 2 Reliability Project L38(C) (Hadi et al., 2014) estimated and analyzed multiple metrics to assess the reliability of the corridor, according to L05 recommendations. However, project L38(C) stressed on the point that having many performance measures, which may not point to the same conclusions without good explanations, may be the reason to create confusion to the analysts and the users of analyses.

Pulugurtha et al. (2015) found that the average travel time has a good correlation with all the travel times and travel time variations. However, it is not correlated with the BTI, PTI, λ skew, λ variance and TTI. They also found that travel times are not correlated with the BTI. However, other travel time variability indices and reliability measures have good correlation with the BTI. The PTI also shows similar trends as the BTI. It is not correlated with travel times but correlated with other travel time variability and reliability measures. The TTI does not have a correlation with travel time. Except the λ skew, all the indices are correlated with TTI. The different travel time based measures and recommendations by selected researchers are summarized in Table 2.2.

TABLE 2.2: Travel time reliability measures recommended by different sources (modified from Pu, 2010)

Travel Time Reliability Measures	Lomax et al. (2003)	FHWA Guide (2006)	NCHRP Report 618 (2008)	SHRP2 (2008)	California Transportation Plan (1998)
95th or Other Percentile Travel Time		✓			
Standard Deviation		X	X		
Coefficient of Variation		X	X		
Percent Variation	✓		✓		✓
Skew Statistic				✓	
Buffer Time Index (BTI)	✓	✓	✓	✓	
Planning Time Index (PTI)		✓	✓	✓	
Frequency of Congestion		✓			
Failure Rate (Percent On-Time Arrival)			✓	✓	
Misery Index	✓		✓	✓	

✓= Encouraged, X= Discouraged

2.4. Current Practices to Determine Freeway LOS

HCM 2010 (TRB, 2010) presents the best available techniques at the time of publishing to determine capacity and LOS. The purpose of the HCM is to provide a set of methodologies and required procedures to estimate, predict, and evaluate the multimodal performance of highway and street facilities through operational measures and quality-of-service indicators. A brief discussion on LOS criteria developed by Transportation Research Board (TRB) for different freeway segments is presented next.

2.4.1. Basic Freeway Sections

TRB defined LOS on a basic freeway section by density. They argued that using speed matrix will be difficult to describe LOS as it remains constant up to flow rates of 1,000 to 1,800 pc/hr/ln, depending on the free flow speed (FFS), even though speed is a major concern of drivers as related to service quality. They concluded that density is sensitive to flow rates throughout the range of flows and describes the proximity to other vehicles. Table 2.3 provides the definitions of different LOSs on a basic freeway section provided by TRB (2010).

Three performance measures can characterize basic freeway section as an indication of traffic accommodation: density in pc/mi/ln, space mean speed in miles per hour (mph), and the ratio of demand flow rate to capacity (v/c). However, TRB (2010) reiterated that as speed is constant through a broad range of flows and the v/c ratio is not directly discernible to road users (except at capacity), the service measure for basic freeway sections is density. Table 2.4 shows the LOS criteria based on density.

TRB (2010) designated the same density boundaries on basic freeway sections for those of surface multilane highways, except at the LOS E–F boundary due to the fact that the maximum flow rates at any given LOS are lower on multilane highways than on similar basic freeway sections. The specification of maximum densities for LOS A to D is based on the collective professional judgment of the members of the TRB’s Highway Capacity and Quality of Service Committee. The upper value shown for LOS F (45 pc/mi/ln) is the maximum density at which sustained flows at capacity are expected to occur (Figure 2.3).

TABLE 2.3: Descriptions of different LOS measures on a basic freeway section (TRB, 2010)

LOS	Description
A	LOS A describes free-flow operations. FFS prevails on the freeway, and vehicles are almost completely unimpeded in their ability to maneuver within the traffic stream. The effects of incidents or point breakdowns are easily absorbed.
B	LOS B represents reasonably free-flow operations, and FFS on the freeway is maintained. The ability to maneuver within the traffic stream is only slightly restricted, and the general level of physical and psychological comfort provided to drivers is still high. The effects of minor incidents and point breakdowns are still easily absorbed.
C	LOS C provides for flow with speeds near the FFS of the freeway. Freedom to maneuver within the traffic stream is noticeably restricted, and lane changes require more care and vigilance on the part of the driver. Minor incidents may still be absorbed, but the local deterioration in service quality will be significant. Queues may be expected to form behind any significant blockages.
D	LOS D is the level at which speeds begin to decline with increasing flows, with density increasing more quickly. Freedom to maneuver within the traffic stream is seriously limited and drivers experience reduced physical and psychological comfort levels. Even minor incidents can be expected to create queuing, because the traffic stream has little space to absorb disruptions.
E	LOS E describes operation at capacity. Operations on the freeway at this level are highly volatile because there are virtually no usable gaps within the traffic stream, leaving little room to maneuver within the traffic stream. Any disruption to the traffic stream, such as vehicles entering from a ramp or a vehicle changing lanes, can establish a disruption wave that propagates throughout the upstream traffic flow. At capacity, the traffic stream has no ability to dissipate even the most minor disruption, and any incident can be expected to produce a serious breakdown and substantial queuing. The physical and psychological comfort afforded to drivers is poor.
F	LOS F describes breakdown, or unstable flow. Such conditions exist within queues forming behind bottlenecks. Breakdowns occur for a number of reasons: • Traffic incidents can temporarily reduce the capacity of a short segment, so that the number of vehicles arriving at a point is greater than the number of vehicles that can move through it. • Points of recurring congestion, such as merge or weaving sections and lane drops, experience very high demand in which the number of vehicles arriving is greater than the number of vehicles that can be discharged. • In analyses using forecast volumes, the projected flow rate can exceed the estimated capacity of a given location. In all cases, breakdown occurs when the ratio of existing demand to actual capacity, or of forecast demand to estimated capacity, exceeds 1.00. Operations immediately downstream of, or even at, such a point, however, are generally at or near LOS E, and downstream operations improve (assuming that there are no additional downstream bottlenecks) as discharging vehicles move away from the bottleneck. LOS F operations within a queue are the result of a breakdown or bottleneck at a downstream point. In practical terms, the point of the breakdown has a v/c ratio > 1.00, and is also labeled LOS F, although actual operations at the breakdown point and immediately downstream may actually reflect LOS E conditions. Whenever queues due to a breakdown exist, they have the potential to extend upstream for considerable distances.

TABLE 2.4: LOS criteria for basic freeway sections (TRB, 2010)

LOS	Density (pc/mi/ln)
A	≤ 11
B	> 11-18
C	> 18-26
D	> 26-35
E	> 35-45
F	> 45 or demand exceeds capacity

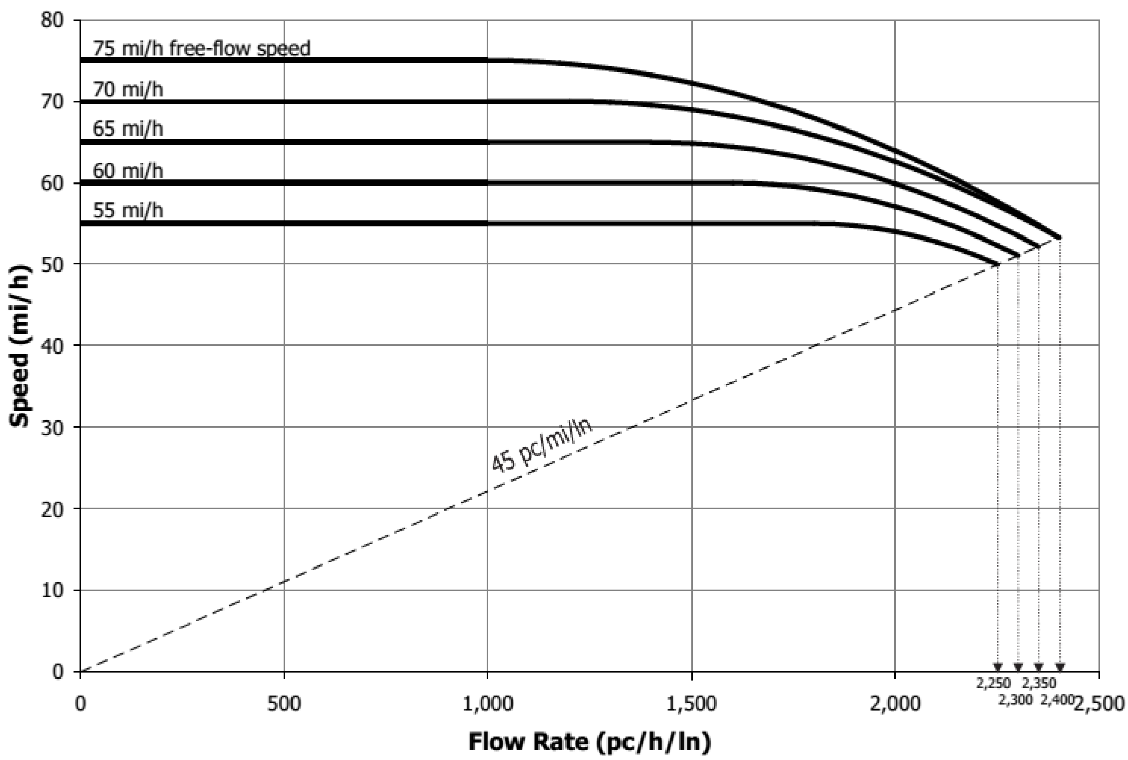


FIGURE 2.3: Speed − flow curves for basic freeway sections under base conditions (TRB, 2010)

However, the analytical method does not provide density when demand exceeds capacity although the density will be greater than 45 pc/hr/ln (LOS F). Figure 2.4 illustrates the defined LOS on the base speed−flow curves. On a speed − flow plot, density is a line of constant slope beginning at the origin and the LOS boundaries were defined to produce reasonable ranges within each LOS on these speed − flow relationships.

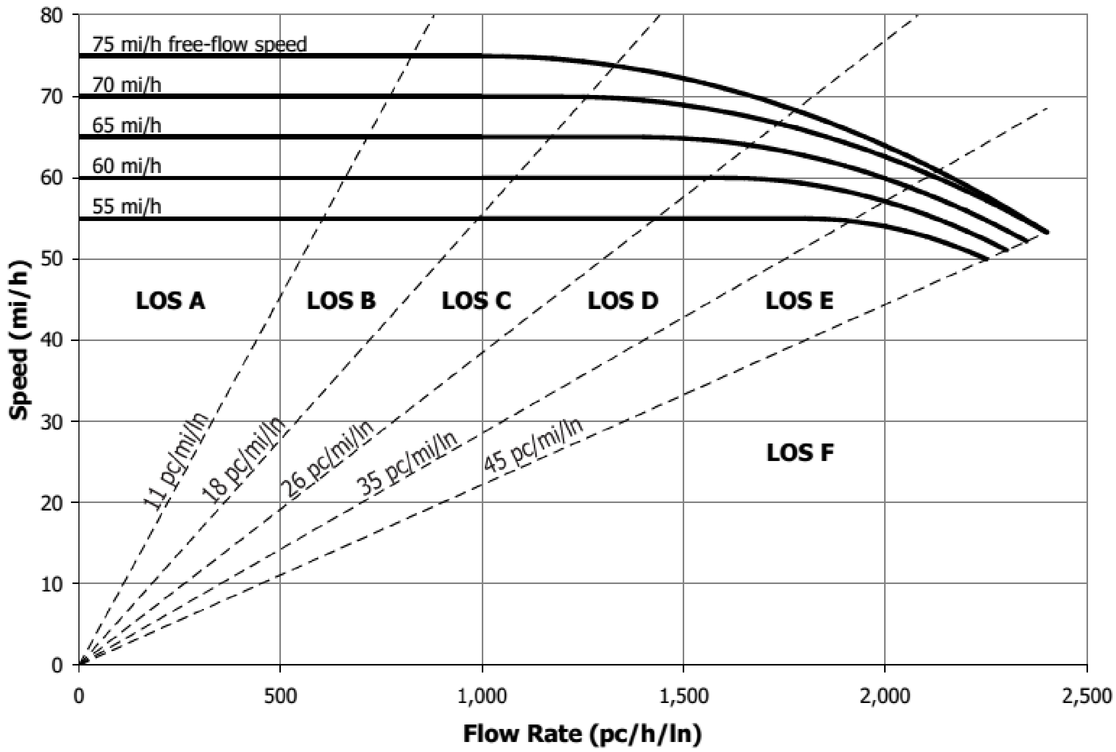


FIGURE 2.4: LOS illustration for basic freeway sections (TRB, 2010)

2.4.2. Weaving Sections

The LOS for a weaving section is also defined by density thresholds. Table 2.5 provides LOS criteria for weaving sections on freeways, collector–distributor (C–D) roadways, and multilane highways, applicable on uninterrupted segments of multilane surface facilities, although its use in such cases is approximate. The threshold densities for levels of service A to D were set relative to the criteria for basic freeway sections (or multilane highways). The boundary between stable (LOS E) and unstable flow (LOS F) occurs when the demand flow rate exceeds the capacity of the segment (v/c>1). The primary reason that density thresholds in weaving sections are higher than those for similar basic freeway sections (or multilane highways) is drivers are believed to tolerate higher densities in an area where lane changing turbulence is expected than that of on basic freeway segments (TRI and KAI, 2008).

TABLE 2.5: LOS criteria for weaving sections (TRB, 2010)

LOS	Density (pc/mi/ln)	
	Freeway	Multilane Highway or C-D Roadways
A	0-10	0-12
B	> 10-20	> 12-24
C	> 20-28	> 24-32
D	> 28-35	> 32-36
E	> 35	> 36
F	Demand exceeds capacity	

2.4.3. Merging/Diverging Areas

Like basic freeway and weaving sections, merging/diverging area LOS is also defined in terms of density for all cases of stable operation (LOS A–E). LOS F is defined when the freeway demand exceeds the capacity of the upstream (diverges) or downstream (merges) freeway segment, or where the off-ramp demand exceeds the off-ramp capacity.

Table 2.6 summarizes the LOS criteria for freeway merge and diverge segments, applicable to all ramp–freeway junctions and may be to major merges and diverges; high speed, uncontrolled merge or diverge ramps on multilane highway sections; and merges and diverges on freeway C–D roadways. However, LOS is not defined for ramp roadways. In general, when facility-level analysis is considered, TRB (2010) identified LOS F for a component segment in two different ways:

• when v/c is greater than 1.00, or

• when the density is greater than 45 pc/mi/ln for basic freeway sections or 43 pc/mi/ln for weaving, merge, or diverge segments.

The latter identifies segments in which queues have formed as a result of downstream breakdowns.

TABLE 2.6: LOS criteria for freeway merging and diverging areas (TRB, 2010)

LOS	Density (pc/mi/ln)	Comments
A	≤ 11	Unrestricted operations
B	> 11-18	Merging and diverging maneuvers noticeable to drivers
C	> 18-26	Influence area speeds begin to decline
D	> 26-35	Influence area turbulence becomes intrusive
E	> 35-45	Turbulence felt by virtually all drivers
F	Demand exceeds capacity	Ramp and freeway queues form

2.5. Literature on Reliability based Level-of-Service Methods

Reliability based LOS may be defined in a variety of ways. The intent of this discussion is to identify literature regarding options for consideration and their relative strengths and weaknesses. In addition, LOS definitions require cut-off points (boundaries) of the measurement unit to be established to define each LOS range. However, based on the statistical analysis, one or more options for defining reliability based LOS can be proposed. It should be noted that all reliability measures should be scoped for accurate comparison between facilities. SHRP 2 Reliability Project L08 (Kittelson and Vandehey, 2013) discussed several options for defining reliability based LOS:

- Reliability LOS based on current LOS ranges
- Freeway reliability LOS based on travel speed ranges
- Freeway reliability LOS based on most restrictive condition
- Reliability LOS based on the value of travel

2.5.1. Reliability LOS based on Current LOS Ranges

The simplest method for defining reliability LOS is to use the existing LOS definitions, which is density for freeway facilities and basic freeway sections. This is most consistent with current LOS concepts in the HCM. For each facility type, the analysis procedure produces a distribution of the LOS measure. This distribution represents the percent of trips that fall into each LOS range. However, the definition could be based solely on the percent of trips in LOS F alone. Density based LOS criteria for freeways are a significant departure from the concept of travel time reliability. Furthermore, the travel times do not vary much over a wide range of density based LOS ranges (A to E are in the unsaturated range). All oversaturated conditions are grouped into a single LOS F category. In addition, the density thresholds for weaving sections are lower than that of other freeway sections which, in turn, further complicate the use of density as the fundamental measure of reliability.

However, it is difficult to disseminate the distribution rather than a single LOS value to the non-technical audiences. Therefore, only the percent of trips in LOS F or E and F can be reported. While a single value has the most consistency with HCM practices, the use of a distribution provides a provision to a reliability analysis. A reliability analysis always captures a range of operating conditions on the same facility, attributed to the various sources of (un)reliability. Therefore, using a distribution of LOS values intrinsically mirrors the variability of traffic conditions on the facility.

2.5.2. Reliability LOS based on Travel Speed Ranges

In this approach, travel speed ranges are constructed for all roadway types similar to that for urban streets. The travel speed is analogous to space means speed (SMS) over the entire facility or segment and the LOS ranges may be based on percentages of the free flow speed, the same for urban streets or may be set at fixed SMS values. Due to the insensitivity of travel speeds to a wide range of density and v/c values (current LOS A through D), LOS ranges can be extended to the oversaturated conditions. This option would conceptually make freeway reliability LOS consistent with urban streets and urban streets segments. However, the problem is to present a distribution rather than a single LOS value.

2.5.3. Reliability LOS based on Most Restrictive Condition

The above two methods provide a distribution rather than a single “grade” to define a highway’s LOS, which may be confusing to the non-technical parties who are used to a single LOS value. As mentioned earlier, one approach would be to focus on the percentage of trips in LOS F, rather than providing percentages for each LOS range, which is a departure from how LOS ranges are defined in the current HCM.

As an alternative to all these, reporting the “most restrictive condition” would provide travel speed boundaries to observe the percentage of trips greater than or equal to each travel speed. A second threshold value, the cumulative percentage of trips for the restrictive condition, is required. The cell value greater than or equal to the threshold gives the LOS value. This is functionally equivalent to selecting a percentile value for a threshold and seeing where it fits. Instead of using the 75^{th} percentile TTI, the 25^{th} percentile space mean speed could be used. Instead of travel speed, reliability metric can also be used as the threshold value establishing reliability LOS based on a single value but in a simpler manner. It establishes LOS ranges for a reliability metric and makes the assignment solely based on where the facility’s calculated value falls. For consistency, all the space means speed may be converted to TTI. The LOS can be defined on the basis of the full distribution of TTI on the fraction of time TTI exceeds a given value, which can be associated with LOS F, or on the basis of a range at a specified TTI percentile of 75th, 80th, or 95th. Though this method overcomes the problem of presenting a distribution, two values are required to be set for providing a percentage threshold for the trips that fail to meet LOS criteria, and the ranges for each LOS category.

2.5.4. Reliability LOS based on the Value of Travel

The concept for this method is to translate both the value of typical (average) travel time and travel time reliability into travel time equivalent values to assign a cost to them. Afterwards, the LOS ranges are assigned based on unit costs per traveler. This is a departure from traditional LOS approach; however, such an option can be applied to both interrupted an uninterrupted facilities.

Small et al. (2012) adopted the quantitative measure of variability as the upper tail of the distribution of travel times. This is specifically the difference between the 80th and 50th percentile travel times, arguing that this measure is better than a symmetric standard deviation, as travelers worry about being late than being early. Planning for the 80th percentile travel time would mean arriving late for only 20% of the trips. Based on this, “travel time equivalents” can be defined which have both typical (average) and reliability components as the same unit. That means the reliability is equilibrated to average travel time.

The calculation of travel time equivalents is:

$$TT_e = TT_m + a * (TT_{80} - TT_{50}) \tag{2.14}$$

where,

TT_e = Travel time equivalent on the segment or facility,

a = Reliability Ratio (VOR / Value of Time (VOT)),

TT_m = Mean travel time, and,

TT_{80} and TT_{50} are the 80^{th} and 50^{th} percentile travel times, respectively.

The end result is an estimation of equivalent delay value, normalized to segment length (delay per mile). The LOS ranges would then be set on delay per mile. Though this method provides a single composite value for facility performance, calculation methods and reliability ratios are required to be established. SHRP 2 Project C04 suggests a range of 0.5 to 1.5, but a review of past studies suggests that the range is more in the 0.9-1.2 range. Therefore, a value of 1.0 seems to be very reasonable for composite trips, though previous research indicates that the value of reliability varies by trip purpose.

2.6. Integration of Fuzzy Logic in Transportation Related Problems

2.6.1. Fuzzy Logic

For the analysis and evaluation of any process, model definition is one of the most important objectives (Arabani and Pourzeynali, 2005). According to Kosko (1996), the results of a study depend on how to make decisions and approximations so that the model can more realistically identify the system. The theory of fuzzy sets and fuzzy logic is a fairly new field in mathematics identified by Iranian mathematicians and applied as a powerful method in decisions. The basis of fuzzy logic is based on a mathematical model of a physical phenomenon as a human argument and decision, and unlike the Bolean logic, a logical proposition can admit a continuous value through zero and one of distance instead of a “true” or “false” value (Arabani and Pourzeynali, 2005). Zadeh (1973) proposed that all discontinuous theories can be transformed into a continuous state through a “fuzzification” process. Zadeh (1973) successfully showed that vague logical statements enable the formation of algorithms that can use vague data to derive vague inferences. The researcher assumed that the approach would be beneficial above all in the study of complex humanistic systems. Mendel (1995) explained the concept of a fuzzy logic system (FLS) as “ a non-linear mapping of an input data (feature) vector into a scalar output (the vector output case decomposes into a collection of independent multi-input/single-output systems)”. In defining the fuzzy model, Cox (1995) identified the main tasks as listed next.

- To choose the appropriate family of membership function corresponding to each parameter,
- To determine the different parameters of the membership functions using the previous studies and expert knowledge, and
- To modify the parameters of membership functions using an optimization method.

According to Nahman (1997), the fuzzy logic is an appropriate theoretical basis for network reliability evaluation in most applications. Such an approach has successfully been used in the field of assurance sciences (Werma and Knezevic, 1994), process-control systems (Bastani et al., 1994), and system criticality analysis (Palaez and Bowles, 1994) associated with reliability assessment and failure criteria definition. Fuzzy arithmetic has also been proved as a successful approach for the reliability analysis of power systems with fuzzy loads and component reliability indices (Miranda, 1996).

2.6.2. Application of Fuzzy Logic in Transportation Operation

Pappis and Mamdani (1977) successfully used fuzzy logic to solve a practical traffic and transportation problem. In the mid- and late-1980s, a group of Japanese authors made a significant contribution to fuzzy set theory applications in traffic and transportation. Nakatsuyama et al. (1983), Sugeno and Nishida (1985) and Sasaki and Akiyama, (1986, 1987, and 1988) solved complex traffic and transportation problems which indicate the great potential of using fuzzy set theory techniques in solving transportation problems. At the end of the 1980s and beginning of the 1990s, the fuzzy set theory in traffic and transportation was extensively used to examine highway LOS (Chakroborthy, 1990), research transshipment problem (Perincherry and Kikuchi, 1990), solve linear programming in transportation (Kikuchi et al., 1991), and model vehicle scheduling (Kikuchi, 1992). Different traffic and transportation related problems were successfully solved using fuzzy set theory techniques, that includes freeway ramp analysis (Chen et al., 1990), route choice (Lotan and Koutsopoulos, 1993), origin–destination matrix (Xu and Chan, 1993), traffic safety planning (Akiyama and Shao, 1993), traffic signalization (Chang and Shyu, 1993), and travel time information service device (Akiyama and Yamanishi, 1993).

Many problems related to transportation planning and traffic control are often not well defined, ambiguous, and vague, and many of the problems, phenomena, and parameters are characterized by subjectivity (Sarkar et al., 2012). According to the researchers, subjective judgment is present in problems dealing with the choice of route, mode of transportation and carrier, a driver's perceptions and reactions, an established LOS, defining safety standards, and defining criteria to rank alternative transportation plans and projects. Both deterministic and stochastic models, developed to solve a variety of complex traffic and transportation problems, are characterized by binary logic. Though using the binary logic as the basis of the development of many scientific disciplines and technology led to the societal prosperity, it should be emphasized that it cannot deal effectively with passengers', dispatchers', or drivers' feelings of uncertainty, vagueness and ambiguity. As the fuzzy set theory recognizes that this vagueness exists in some sets, different fuzzy set theory techniques need to be used to properly model traffic and transportation problems characterized by ambiguity, subjectivity, and uncertainty (Sarkar et al., 2012).

2.7. Limitations of Prior Researches

Though a lot of researches have been conducted on identifying the best travel time reliability statistics, there is still no consensus on this issue. The SHRP 2 Reliability Project L08 (Kittelson and Vandehey, 2013) documentations stated that "it is difficult to say which metric should be highlighted as the primary reliability metric; a lot depends on the specific application being used".

TRB (2010) argued that speed cannot represent LOS on freeway segments as they remain constant through a broad range of flows. From an operational analysis perspective, LOS A to C do not carry much weight as the road users experience little to no-variation in performance. Therefore, some researchers argue that these three LOS can be merged together for the operational analysis purposes.

In addition, it is extremely difficult to appoint LOS F based on the current method as it is practically a difficult task to record when v/c goes over 1. Therefore, a new LOS method for planning purposes that can carry meaningful explanation to both road users and planners is a timely response to this problem.

Kittelson and Vandehey (2013) conceptualized and described several ideas to use travel time reliability to establish freeway LOS. However, it is still in preliminary stage and there is a lack of consensus on the best approach. This dissertation brings harmony among all the density criteria and replaces them with travel time, speed, and travel time reliability thresholds. This approach should be compatible with what the planners have been using so far. Nevertheless, it will provide a readily identifiable LOS under all circumstances, which in turn will provide more opportunity to deploy a dynamic traffic control and planning scheme.

CHAPTER 3: RESEARCH METHOD

3.1. Introduction

The previous chapter provided literature related to the travel time based metrics and LOS criteria, and how the reliability measures can be used as a tool to express freeway LOS. This chapter identifies a new method to define travel time and travel time reliability based LOS and provides insights into the method to achieve the research objectives.

3.2. Microscopic Simulation Model Development

A large number of scenarios arise due to a combination of various factors for estimating the vehicular density. These range from network conditions to traffic characteristics. It is not practically feasible and not always possible to obtain data for all such scenarios from the field. In addition, there is no analytical model available that can represent the freeway conditions realistically considering all these scenarios. Therefore, VISSIM traffic simulation software was used to compute the vehicular density and speed/travel time information for different freeway sections, such as basic freeway section, merging/diverging area, and weaving section.

VISSIM is a state-of-the-art microscopic, behavior-based multimodal traffic simulation model (PTV, 2014). Its dynamic traffic assignment model can answer route choice dependent questions, which in turn, can help assess the relationship between the vehicular density and user travel time/speed.

3.2.1. Database

The database required to develop microscopic models include speed profile along the freeway, street centerline network, and traffic counts obtained from the permanent traffic count stations. For the calibration purpose, manual travel time data and queue length data are also required.

3.2.2. Model Calibration and Validation

Model calibration is utmost important to ensure the adaptability and generality of a model. Otherwise, the model will lose its validity to be used for all scenarios. Calibration ensures model's consistency, accuracy, and reliability (MSL, 2009), which are extremely important in developing generalized model criteria.

VISSIM model calibration is the process used to achieve adequate validity of the model by establishing suitable parameters so that the model replicates local traffic conditions as closely as possible. Calibration, described in the following chapter, is achieved by iteratively changing model parameters to replicate the traffic patterns, congestion, bottlenecks, and driver behavior observed within the study area. Validation of the calibrated parameters is also important to produce an accurate and credible model. From a performance modeling standpoint, the criteria for judging the goodness of models should be based on how accurately measures extracted from the model correspond to the measures which would be obtained from the represented system (field).

3.2.3. Development of Hypothetical Sections

Several hypothetical sections are developed for the establishment of density–travel time, density–speed and density–travel time reliability relationships. The experiments are performed based on different speed limit criteria that prevail on the freeways with the use of VISSIM traffic simulation software. The three section types (basic freeway section, weaving section, and merging/diverging area) are evaluated based on demand fluctuations, different lane configurations, and lengths. Demand on the freeway and on-ramp/off-ramp are fluctuated to obtain the randomness in general. Three different mainline demand conditions (off-peak, mid-day, and peak) are evaluated based on 15-minute flow rate for each combination. Other critical criteria include the length and the number of lanes of the section. Both of the factors affect the maneuverability of the roadway users and operation on the freeway.

3.2.4. Density Computation

The densities for each of these combinations are obtained from VISSIM traffic simulation model. In addition, according to FHWA, the HCS software is intended to be a faithful implementation of the HCM, which computes the LOS exactly according to the HCM methods (FHWA, 2013). HCS, in general, can be used to compute the LOS of basic freeway section, weaving section, and merging/diverging areas. Hence, for each of these hypothetical section combinations, densities are also computed based on the HCM method. HCS 2010 software was used to perform the analysis. Both of the densities are computed on hourly basis.

3.2.5. Density Comparison

A comparison between densities obtained from HCS using HCM method and densities generated through VISSIM for the same combination provides better understanding of the relationship between densities from the two methods. A correlation matrix table is generated to show the correlation between VISSIM based densities and HCM method based densities. A high correlation coefficient between them indicates that the calibrated microscopic simulation model can effectively represent the roadway operation identified by HCM and the results can be used to generate appropriate LOS criteria.

3.2.6. Generation of VISSIM Travel Time based Metrics

The average travel times for each hour are generated from the VISSIM software for the same combinations described in Sub-section 3.2.4. The travel times are converted to travel time per mile prior to post-processing and further analyses to have a meaningful relationship due to the length variations used in hypothetical combinations.

One-minute travel time data are also generated for all the combinations to compute 95th, 50th, and 5th percentile travel times for each hour. Travel time reliability is a measure of consistency based on data for several days during a time period (preferably for a year). Though data for several days were not used, the concept is still applicable and adopted based on microscopic simulation outputs for considered time periods in this research. The equations used to compute PTI and BTI are as follows.

$$\text{PTI (T)} = \frac{95\text{P (T)}}{5\text{P (T)}} \tag{3.1}$$

$$\text{BTI (T)} = \frac{95\text{ P (T)} - 50\text{ P (T)}}{50\text{ P (T)}} \tag{3.2}$$

where,

95P = 95th percentile of average travel time,

50P = 50th percentile of average travel time,

5P = 5th percentile of average travel time, and

T = Observation period.

3.2.7. Density – Travel Time based Metrics Relationship Development

The hourly average travel time (50th percentile) per mile and their respective hourly density values from VISSIM traffic simulation software models are paired together to establish a density – travel time per mile relationship for different speed limit criteria. Based on these relationships, the travel time per mile thresholds for different LOSs comparable to the density LOS thresholds suggested by TRB's Highway Capacity and Quality of Service Committee are developed. Likewise, density – speed thresholds are also developed for different freeway posted speed limits. Speed values (space mean speed) are computed from the average travel time per mile values.

Similarly, the hourly PTI and BTI values computed from VISSIM travel time outputs are paired with their respective VISSIM hourly density values to obtain density – PTI and density – BTI relationships for different posted speed limits. Travel time reliability thresholds are computed based on different density threshold values suggested by TRB's Highway Capacity and Quality of Service Committee.

3.3. Implementation of Travel Time Reliability LOS

3.3.1. Travel Time Data

For the implementation of obtained results, a freeway corridor is selected for travel time and travel time reliability analysis over a certain period of time. The travel time data from the INRIX (a travel time database provider) provide the required historic travel time data for that specific corridor. Based on the INRIX data, Traffic Message Channel (TMC) codes along that selected corridor are obtained. Information on average travel time, average speed, and link lengths for each TMC codes for every 5-minute interval are collected and analyzed for segment level analysis.

3.3.2. Section Level LOS

The PTI and BTI values are computed for the selected TMC code for different observation periods. The observation period is a significant criterion in identifying the travel time reliability indices. The travel time reliability index values depend largely on how much data are processed. An analysis is performed to show their significance in the travel time reliability computation. The travel time reliability based metrics are compared against their respective travel time reliability LOS thresholds to identify the travel time reliability LOS for each timestamp considered.

3.3.3. Section Level Composite LOS

By using fuzzy logic set, the segment level composite LOS consisting of several timestamp LOSs and, in turn, an overall network level LOS can be achieved for a specific time window. The compatibility index (CI) for each segment of the freeways, which defines the specified conditions, is computed using the average of the membership function values as (Cox, 1995):

$$CI = \frac{\sum_{i=0}^{n} \mu i(x)}{n} \tag{3.3}$$

where, $\mu_i(x)$ is the degree of membership for the parameter in determining the service level and N is the number of parameters. For each service level, CI is the average value of the degree of membership of its basic parameters. This provides an idea of how much each LOS approaches the desired conditions. By obtaining CI for each LOS, a section level composite LOS identification can be achieved.

CHAPTER 4: MICROSCOPIC SIMULATION MODEL CALIBRATION AND VALIDATION

4.1. Introduction

The previous chapter discussed the method to obtain freeway LOS based on travel time and travel time reliability. This chapter focuses on the microscopic simulation procedure to develop density – travel time metrics relationships based on a calibrated model for different posted speed limits on freeways.

4.2. Model Calibration

Model calibration is of utmost important to ensure the adaptability and generality of a model. Otherwise, the model will lose its validity to be used for other scenarios. Calibration ensures model consistency, accuracy, and reliability (MSL, 2009), which are extremely important in developing generalized model criteria.

VISSIM model simulation calibration helps the model replicate local traffic conditions as closely as possible to achieve adequate validity of the model by establishing suitable parameter values. The process includes iteratively changing model parameters for replicating the traffic patterns, congestion, bottlenecks, and driver behavior observed within the study area.

The calibration criteria from the FHWA's *Traffic Analysis Toolbox Volume III: Guidelines for Applying Traffic Microsimulation Modeling Software Report* (Dowling et al., 2004) has been utilized as a guide for the calibration purpose. Table 4.1 provides the established VISSIM model calibration criteria.

TABLE 4.1: Calibration criteria based on FHWA's Traffic Analysis Toolbox (Dowling et al., 2004)

Criteria and Measures	Calibration Acceptance Targets
Hourly Flows, Model Versus Observed	
Individual link flows – within 15%	> 85% of cases
Within 100 veh/hr, for flow < 700 veh/hr	> 85% of cases
Within 15%, for flow > 700 veh/hr and for flow < 2,700 veh/hr	> 85% of cases
Within 400 veh/hr, for flow > 2,700 veh/hr	> 85% of cases
Sum of all link flows	Within 5% of sum of all link counts
GEH Statistic* < 5 for Individual Link Flows	> 85% of cases
Travel Times, Model Versus Observed	
Journey times, network – within 15% (or 1 min, if higher)	> 85% of cases
Visual Audits	
Individual link speeds – visually acceptable speed–flow relationship	To analyst's satisfaction
Bottlenecks – visually acceptable queuing	To analyst's satisfaction

* The GEH statistic, used in traffic modeling to compare two sets of traffic volumes, is an empirical formula that has proven useful for a variety of traffic analysis purposes. The use of GEH as an acceptance criterion for travel demand forecasting models is recognized in the Wisconsin microscopic simulation modeling guidelines and FHWA's Traffic Analysis Toolbox.

The GEH statistic is obtained as follows:

$$\text{GEH} = \sqrt{\frac{(E-V)^{\wedge}2}{(E+V)/2}} \tag{4.1}$$

where,

E = Model estimated volume, and

V = Field count.

4.3. Case Study Area

Interstate 295 (I-295) is a north-south limited access facility in the city of Jacksonville, Florida that provides an eastern and western bypass around the city. The case study area comprised of a six-mile portion of the I-295 eastern bypass from the Town Center Parkway service interchange to the State Route 5 (SR 5) (or US 1– Phillips Highway) service interchange. This segment of I-295 is an important link in the state's interstate system providing a major north-south access around the Jacksonville urban area.

Six interchanges along I-295 (four service interchanges and two system-to-system interchanges) were considered for the simulation. The model also includes two service interchanges along SR 202 (J. Turner Butler Boulevard) due to their close proximity to I-295. The following interchanges were included in the study:

I-295 Interchanges:

- SR 5 – Service interchange
- SR 9B – System-to-system interchange
- SR 152 (Baymeadows Rd) – Service interchange
- Gate Parkway – Service interchange
- SR 202 – System-to-system interchange
- Town Center Parkway – Service interchange

SR 202 (J. Turner Butler Blvd) Interchanges:

- Gate Parkway – Service interchange
- Kernan Boulevard – Service interchange

In addition, along the arterials, the model included the ramp terminal intersections and one signalized intersection adjacent to these ramp terminal intersections. Figure 4.1 illustrates the case study area location and the area of influence.

FIGURE 4.1: Case study area for microscopic simulation model development

4.4. Data Collection for Calibration and Validation

The data collection process involved the collection and preparation of all the data necessary for the microscopic simulation analysis, which requires extensive input data including but not limited to:

- Roadway geometry data
- Existing demand (tube counts, turning movement counts, etc.)
- Control data (signal timings, stop/yield signs, regulatory/advisory speed limits, etc.)
- Calibration data (capacities, travel times, queues, etc.)

4.4.1. Roadway Geometry Data

Year 2011 aerial imageries were obtained from the Florida Department of Transportation (FDOT) in the form of Multi-resolution Seamless Image Database (MrSID) files. These MrSID files provided the necessary roadway geometry information, including the number of lanes, length of acceleration/deceleration lanes, curvature, and similar elements. However, the roadway geometry data (MrSID files) were later modified using the aerial imagery at I-295 & SR 9B Interchange and two other intersections due to the geometric modification carried out by FDOT and local authority.

4.4.2. Demand Data

Tube counts containing 48-hour weekday vehicular data were collected at all the entrance and exit ramps within the study area. Additionally, 48-hour vehicle classification counts were also collected along I-295. AM and PM peak periods (AM peak period – 6:00 AM to 9:00 AM, and PM peak period – 3:00 PM to 6:00 PM) turning movement counts were collected at 21 ramp terminal intersections (Figure 4.1), four mainlines, and 24 on-/off-ramps along I-295. Traffic volume data collected in the field was supplemented with the most recent data (Year 2012) available from FDOT Florida Traffic Information (FTI) online website (TSO, 2014). According to suggestion of

FDOT, a one percent annual growth rate was applied to this information to obtain the base year (2013) volumes.

The volume balancing was carried out by holding the ramp volumes constant and balancing the arterial movements using the turn percentages obtained from the traffic counts. These base year (year 2013) traffic volumes were used in the VISSIM models, where multi-hour simulations were performed at 15-minute intervals.

4.4.3. Bluetooth Data

Preliminary travel time and speed data along the I-295 corridor were collected from the *I-95 and I-295/S.R. 9A Bluetooth Data Analysis* report (FDOT District 2, 2012). This report also contains preliminary origin-destination information along the I-295 corridor.

4.4.4. Control Data

Traffic signal timing data for both AM and PM peak hours were obtained from FDOT District 2 Traffic Operations. Field visits were carried out to verify the signal timing and phasing information, and record stop/yield sign locations, regulatory/advisory speed limits, and guide sign locations.

4.4.5. Calibration Data

4.4.5.1. Traffic Volumes

The balanced AM and PM peak hour traffic volume data and the 15-minute distribution percentage were obtained from the Design Traffic Memorandum (DTM) provided by FDOT. This data set was used to verify the traffic volume criteria during the calibration of the VISSIM models. Table 4.2 provides the hourly volume percentages from the DTM.

TABLE 4.2: Conversion percentages for hourly volumes

Time Period	AM Peak Period	PM Peak Period
Pre-Peak Hour (Hour 1)	64.18%	84.76%
Peak Hour (Hour 2)	100.00%	100.00%
Post-Peak Hour (Hour 3)	86.95%	83.16%

4.4.5.2. Travel Times

To identify and quantify congestion along the I-295 corridor, travel time data were collected during normal weekdays. This data set was used to calibrate the travel times in the VISSIM models. AM and PM peak hour travel time runs were conducted using probe vehicles. Runs were conducted from the south of US-1 to the north of Town Center Parkway. A total of eight runs in the AM peak hour and six runs in the PM peak hour were conducted along the I-295 corridor. A total of four runs were conducted along the SR 202 corridor. The average values of travel times recorded were used for the VISSIM model travel time calibration.

4.4.5.3. Visual Bottleneck Locations

During the travel time data collection, visual bottlenecks and speed drop zones were identified and the maximum back of queues along I-295 mainline and ramps within the study area were documented. Maps showing the bottleneck locations and the extent of the queues were prepared to aid in the VISSIM model calibration.

4.4.5.4. Queue Study

A queue study was conducted independently through FDOT between at the interchange intersections shown in Figure 4.1 to record the maximum and average queue lengths for all the intersection approaches. The queue study data were used to calibrate the microscopic simulation models.

4.5. Base Model Development

The VISSIM simulation model for this study included the I-295 mainline travel lanes, ramp merge/diverge areas, ramp terminal intersections, and adjacent signalized intersections.

4.5.1. Roadway Geometry

The VISSIM network for the existing conditions analysis was developed using year 2011 aerial imagery (MrSID files) obtained from FDOT as outlined in Sub-section 4.4.1. A preliminary roadway network comprising of links, connectors and storage bays for turn movements was created. Links are one-directional segments of freeways or surface streets representing the length of the segment and containing data on the geometric characteristics of the road or highway between connectors. HCM method was used in coding the links and is described as below:

- Basic freeway sections – used for all the locations that do not encounter a merging/diverging area or weave section
- Merging/diverging areas – 1,500 ft downstream of merging point and 1,500ft upstream of diverging point
- Weaving sections – between successive on- and off-ramps where interference with freeway traffic was observed
- Urban arterials – arterial roadways

4.5.2. Speed Distributions

The “Desired Speed Decisions” and “Reduced Speed Areas” were coded in VISSIM based upon the type of roadway segment/facility. Desired speed decision points in VISSIM change the speed of vehicles while crossing them, and should be used when the free-flow speeds of an area have a significant change due to the posted speed limit, geometric change, topography, or other factors. Additionally, reduced speed areas are temporary zones with a reduced speed and should be used to code small sections where vehicles have a significant change in speed (e.g., ramps, turning movements etc.).

The desired speed decisions were based on the posted speed limits. The speed profiles for I-295, SR 9B, and SR 202 (J. T. Butler Blvd) were generated from the speed data of the Bluetooth report as discussed in Sub-section 4.4.3 (Figure 4.2). For arterials, the upper and lower limits for the speed distribution were selected as a linear distribution within a range of five mph of the posted speed as suggested in the "Protocol for VISSIM Simulation" by Oregon Department of Transportation (ODOT) (Mai et al., 2011). A 2.5 mph range of the advisory speed was used on system ramps.

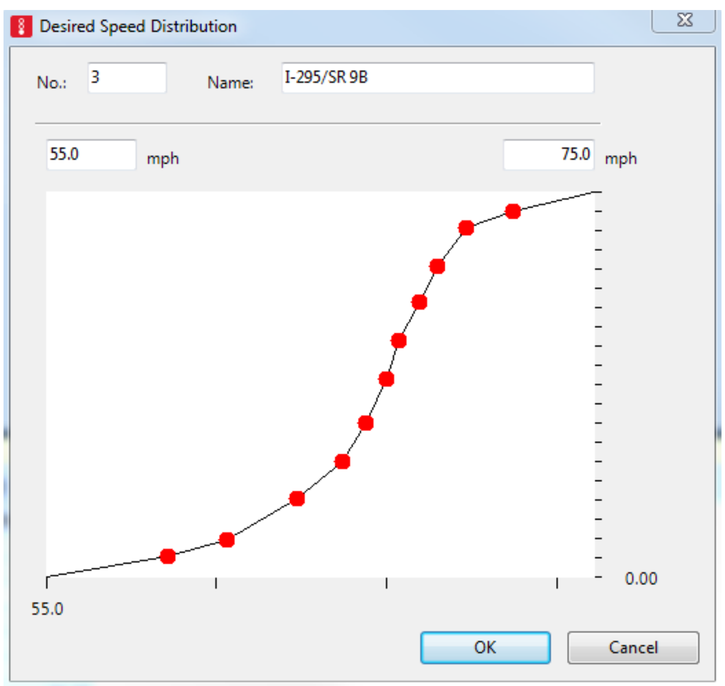


FIGURE 4.2: Speed profile for I-295/SR 9B/SR 202

Mai et al. (2011) also suggests an upper limit of 15 mph and a lower limit of 9 mph for right turning vehicles. They also recommend a value of about 15 mph for left turning vehicles. A 2.5 mph range of this speed criterion was used for the left turns to achieve the upper and lower limits.

4.5.3. Vehicular Composition

The vehicular traffic in VISSIM comprises of different vehicle types. The vehicle types used for the VISSIM models are:

- Cars (vehicle type: car - 100)
- Trucks or heavy goods vehicle (vehicle type: HGV - 200)

Truck percentages for I-295, SR 202 and SR 5 for the peak-period analyses were determined using the recommended values from FDOT. The peak hour truck percentage for SR 5 was used for all other arterials and side streets within the study area. Table 4.3 summarizes the truck percentages used in the calibration process.

TABLE 4.3: VISSIM vehicle composition

Roadway	**Car Percent**	**Truck Percent**
I – 295, SR 9B, I-295 and I-95 Ramps	96.50 %	3.50 %
SR 202 (J. T. Butler Blvd)	97.75 %	2.25%
SR 5 (US-1), SR 152 (Baymeadows Rd), Gate Pkwy, Town Center Pkwy, Kernan Blvd	99.20 %	0.80%
Cross Streets	99.20 %	0.80%

The default vehicle models in VISSIM are European vehicles, which do not represent the vehicle type composition that is typical for North America. PTV Vision, the software developers for VISSIM, have developed a “NorthAmericaDefault.inp” file with vehicle models that contain inventory of vehicle fleet that provide an accurate representation of vehicles types found in North America. Table 4.4 and 4.5 provide the passenger car and heavy vehicle model distributions used in the VISSIM model, respectively. The default VISSIM software values for the maximum and desired acceleration range (ft./sec^2) , maximum and desired deceleration range (ft./sec^2), weight (kg) and power (KW) attributes were used for the two vehicles types (Cars and HGV).

TABLE 4.4: VISSIM model distribution for passenger cars

VISSIM Model Name	Vehicle Name	% Share	Length (ft)
lttruck_ford_f150_2009	Ford F-150	19.20%	17.75
lttruck_chevrolet_silverado_2008	Chevy Silverado	15.10%	21.89
car_toyota_camry_2006	Toyota Camry	13.50%	15.57
suv_ford_explorer_2008	Ford Explorer	10.60%	16.05
car_honda_accord_2003	Honda Accord	12.90%	15.62
van_plymouth_voyager_1999	Plymouth Voyager	5.50%	16.01
suv_jeep_grand_cherokee_2002	Jeep Cherokee	5.80%	15.23
van_nissan_quest_1995	Nissan Quest	6.40%	15.76
suv_gmc_yukon_xl_2008	GMC Yukon	5.00%	17.83
car_nissan_altima_2005	Nissan Altima	6.00%	16.02

TABLE 4.5: VISSIM model distribution for heavy vehicles

VISSIM Model Name	Vehicle Name	% Share	Length (ft)
hgv_aashto_wb50_tractor, hgv_aashto_wb50_trailer	WB50 Tractor-Trailer	47.78%	55.00
truck	3-Axle Single Unit Truck	27.67%	33.51
hgv_aashto_wb40_tractor, hgv_aashto_wb40_trailer	WB40 Tractor-Trailer	10.55%	45.72
hgv_flatbed_truck	4-Axle or More Single Unit Truck	4.89%	32.58
hgv_aashto_wb67d_tractor, hgv_aashto_wb67d_trailer,	WB67D Tractor-Trailer	4.67%	73.25
hgv_aashto_wb65_tractor, hgv_aashto_wb65_trailer	WB65 Tractor-Trailer	4.44%	73.58

4.5.4. Control Devices

The signal timing and phasing data for the signal heads coded in VISSIM for all the signalized intersections were provided by FDOT. A total of 19 signalized intersections and two unsignalized intersections were included in the AM and PM peak hour VISSIM models. Ring Barrier Controller (RBC) files were developed using SYNCHRO 8 software for the AM and PM peak periods and later imported into VISSIM. Right-turn overlaps, permitted-protected left turning movements, and off-set information were coded into the models to reflect field traffic signal operation.

Stop signs, conflict areas, and/or priority rules were coded for unsignalized intersections. Conflict areas and/or priority rules were also coded into the microscopic simulation model yielding conditions within the VISSIM network where traffic on a minor street has to yield right-of-way for major street traffic (e.g., channelized right turns and permissive left turns).

4.5.5. Traffic Volume Input

A 3.5 hour peak period VISSIM model that depicts buildup and dissipation of congestion along I-295 within the study area was created for the AM and PM peak hours. The 3.5 hours of simulation consists of 30 minutes of seeding time to load the network with traffic to reach equilibrium between the number of vehicles entering and exiting the network, an hour prior to the peak hour, the actual peak hour, and an hour after the peak hour to recover from congestion after the peak hour. The input volumes in VISSIM models were developed from the peak hour volumes and off-peak volume distribution percentages from the DTM provided by FDOT, described in Sub-sub-section 4.4.5.1. Volumes were entered into VISSIM as 15-minute flow rates through the entry links.

4.5.6. Traffic Routing

Static routing was used for directing traffic from one entry link to a desired destination exit link for the AM and PM peak period VISSIM models. These static routes were consolidated utilizing the “Combined Routes” feature in VISSIM to determine O–D patterns that originate from the entry links and end at various destinations within the VISSIM network to avoid unrealistic driver behavior.

4.6. VISSIM Calibration Parameters

There are three calibration parameters in VISSIM based on operational characteristics to replicate the field conditions. These three operational parameters are generally modified in VISSIM to replicate the capacity observed along mainline basic freeway sections, merging/diverging areas, and weaving sections. These parameters play a large role in the capacity calibration of a micro-simulation model.

4.6.1. Driver Behavior Types

Wiedemann's car following model (Wiedemann, 1974), a psycho-physical model, is used for the microscopic simulation purpose in this research (a VISSIM default). The basic assumption of the Wiedemann car following model is that a vehicle is in one of four states of car following: free, approaching, following, or braking. The model defines the driver perception thresholds and the regimes formed by these thresholds (Figure 4.3). Both Wiedemann 74 and Wiedemann 99 car following models are similar in many ways except that some of the thresholds in the Wiedemann 99 model are simpler and user adjustable to model freeway traffic better. Therefore, the former one was applied to urban arterial roads and the later one was used for freeways (VISSIM, 2012).

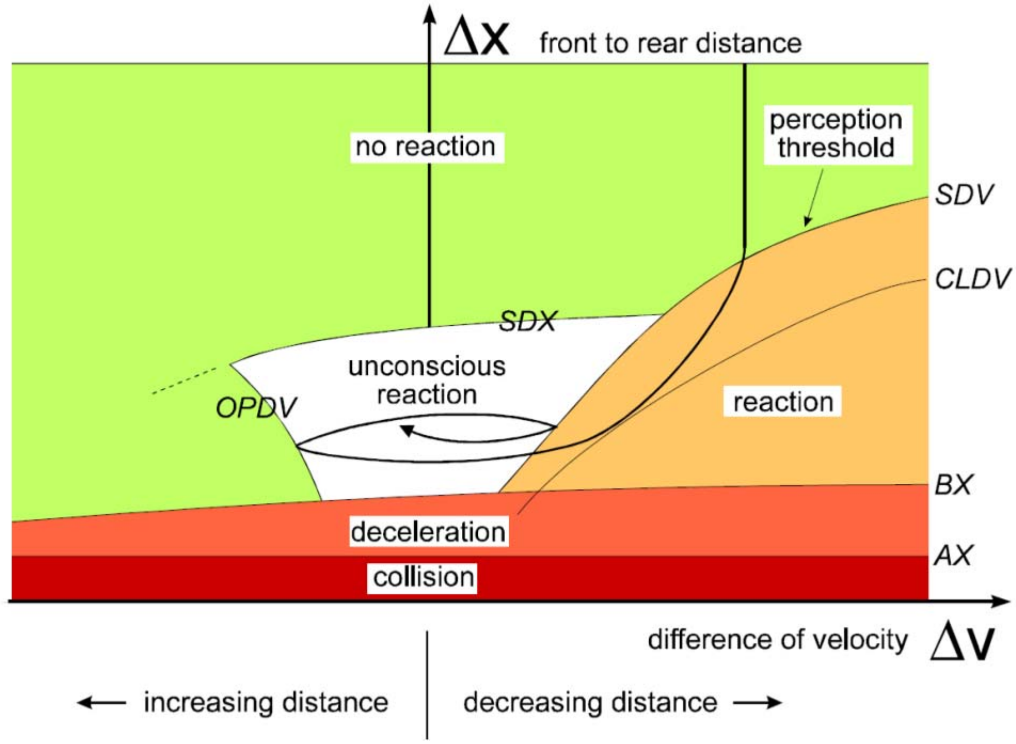


FIGURE 4.3: Thresholds and regimes in the Wiedemann car following models (Wiedemann, 1974)

4.6.1.1. Arterials

The default Wiedemann 74 parameters and lane changing parameters were used for all the arterial driver behavior types and were reflecting field observed driver behaviors. The following parameters incorporate the model (VISSIM, 2012).

- Average standstill distance (ax) defines the average desired distance between stopped cars. It has a variation between ± 3.2 ft, which is normally distributed around 0.0 ft with a standard deviation of 0.1 ft.
- Additive part of desired safety distance (bx_add) and multiplication part of desired safety distance (bx_mult) affect the computation of the safety distance. The distance "d" between two vehicles is computed using this formula below.

$$d = ax + bx \tag{4.2}$$

where, ax is the standstill distance.

$$bx = (bx\_add + bx\_mult * z) * v \tag{4.3}$$

where, v is the vehicle speed (mph), z is a value of range [0,1] which is normally distributed around 0.5 with a standard deviation of 0.15.

4.6.1.2. Freeways

The modeling process started with the creation of the four generic driver behaviors that are observed on a freeway:

- Basic freeway sections
- Merging/diverging areas
- Weaving sections

The model did not replicate the field observed capacities along three sections of I-295. The existing roadway characteristics and traffic operations along these three sections were observed to be different from one another and a need to create three additional driver behavior types was identified.

The dissertation used Wiedemann's 99 car following model to replicate the usual traffic behavior on freeways. Wiedemann's 99 car following model uses ten parameters with prefix "CC". The first seven parameters (CC0–CC6) were used to determine the car-following thresholds, while the rest were used to determine other states. The parameters are defined next (VISSIM, 2012).

- CC0 (Standstill distance) defines the desired distance between stopped cars. It has no variation.

- CC1 (Headway time) is the time (in sec) a driver wants to keep. The higher the value, the more cautious the driver is. Thus, at a given speed v (ft/sec), the safety distance dx_safe is computed as:

  dx_safe = CC0 + CC1 * v.

  The safety distance is defined in the model as the minimum distance a driver will keep while following another car. This distance becomes the value with the strongest influence on capacity in case of high traffic volume.

- CC2 ('Following' variation) restricts the longitudinal oscillation or how much more distance than the desired safety distance a driver allows before the driver intentionally moves closer to the car in front. If this value is set to, for example, 13.12 ft, the following process results in distances (in ft) between dx_safe and dx_safe +13.12.

- CC3 (Threshold for entering 'Following') controls the start of the deceleration process, i.e., when a driver recognizes a preceding slower vehicle. In other words, it defines how many seconds before reaching the safety distance the driver starts to decelerate.

- CC4 and CC5 ('Following' thresholds) control the speed differences during the 'Following' state. Smaller values result in a more sensitive reaction of drivers to accelerations or decelerations of the preceding car, i.e., the vehicles are more tightly coupled. CC4 is used for negative and CC5 for positive speed differences.

- CC6 (Speed dependency of oscillation): Influence of distance on speed oscillation while in following process. If set to 0 the speed oscillation is independent of the distance to the preceding vehicle. Larger values lead to a greater speed oscillation with increasing distance.

- CC7 (Oscillation acceleration): Actual acceleration during the oscillation process.
- CC8 (Standstill acceleration): Desired acceleration when starting from standstill (limited by maximum acceleration defined within the acceleration curves)
- CC9 (Acceleration at 50 mph): Desired acceleration at 50 mph (limited by maximum acceleration defined within the acceleration curves).

The four states of car following are determined using the following six thresholds.

- AX: the desired distance between two stationary vehicles,
- BX: the minimum following distance which is considered safe by the drivers,
- CLDV: the points at short distances where drivers perceive that their speeds are higher than their lead vehicle speeds,
- SDV: the points at long distances where drivers perceive speed differences when they are approaching slower vehicles,
- OPDV: the points at short distances where drivers perceive that they are travelling at a lower speed than their leader, and
- SDX: The maximum following distance indicating the upper limit of car-following process.

The relationship between the parameters and thresholds are defined by Equations 4.4 to 4.9.

$$AX = L + CC0 \tag{4.4}$$

where, L is the length of the lead vehicle.

$$BX = AX + CC1 * v \tag{4.5}$$

where, v is equal to subject vehicle speed if it is slower than the lead vehicle; otherwise, it is equal to lead vehicle speed with some random errors. The error is determined randomly by multiplying the speed difference between the two vehicles by a random number between -0.5 and 0.5.

$$SDX = BX + CC2 \tag{4.6}$$

$$(SDV)_i = -\frac{-\Delta x - (SDX)i}{CC3} - CC4 \tag{4.7}$$

where, Δx is the space headway between the two successive vehicles calculated from front bumper to front bumper.

$$CLDV = \frac{CC6}{17000} * (\Delta x - L)^2 - CC4 \tag{4.8}$$

$$OPDV = -\frac{CC6}{17000} * (\Delta x - L)^2 - \delta * CC4 \tag{4.9}$$

where, δ is a dummy variable which is equal to 1 when the subject vehicle speed is greater than CC5; 0 otherwise.

The CC7 parameter defines the actual acceleration during the oscillation process. The CC8 parameter defines the desired acceleration when starting from standstill condition and the CC9 parameter determines the desired acceleration at the speed of 50 mph.

The primary objective is to calibrate the CC parameters in order to adjust the operational behavior on freeways and enable the microscopic simulation model operation replicate and comparable to the general traffic operation. The car following parameters were modified based on the suggested values provided by:

- FDOT in the *Traffic Analysis Handbook: A Reference for Planning and Operations* report (SPO, 2014),

- ODOT in the *Protocol for VISSIM Simulation* report (Mai et al., 2011),
- Other previous research and studies, such as Menneni et al. (2008), Gomes et al., (2004), and (Woody, 2006).

The suggested ranges for the CC parameters from the FDOT Traffic Analysis Handbook (SPO, 2014) and the ODOT Protocol for VISSIM Simulation report (Mai et al., 2011) are provided in Tables 4.6 and 4.7, respectively.

TABLE 4.6: CC parameter suggested ranges by FDOT (SPO, 2014)

Calibration Parameter	Default Value	Suggested Range	
		Basic Segment	Weaving / Merge / Diverge
Freeway Car Following (Wiedemann 99)			
CCO Standstill distance	4.92 ft	> 4.00 ft	> 4.92 ft
CC1 Headway time	0.9 s	0.70 to 3.00 s	0.9 to 3.0 s
CC2 'Following' variation	13.12 ft	6.56 to 22.97 ft	13.12 to 39.37 ft
CC3 Threshold for entering 'following'	- 8	use default	
CC4 Negative 'following' threshold	- 0.35	use default	
CC5 Positive 'following' threshold	0.35	use default	
CC6 Speed Dependency of oscillation	11.44	use default	
CC7 Oscillation acceleration	0.82 ft/s^2	use default	
CC8 Standstill acceleration	11.48 ft/s^2	use default	
CC9 Acceleration at 50 mph	4.92 ft/s^2	use default	

The FDOT and ODOT handbooks suggested changing the Standstill Distance (CC0), Headway Time (CC1), and ‘Following’ Variation CC2 parameters to obtain calibration criteria compliance. The models were initially run with the default values for all driving behaviors. Calibration criteria were checked for compliance. Then an iterative process was conducted to identify the modeling parameters that produced compliance with all of the calibration criteria. During the iterative process the Standstill Distance

(CC0) value was not found to be having a major impact on the capacity of the sections being analyzed. The Headway Time parameter (CC1) used in conjunction with 'Following' Variation (CC2) has the largest impact on capacity.

TABLE 4.7: CC parameter suggested ranges by ODOT (Mai et al., 2011)

	Default	Unit	Suggested Range	
			Basic Segment	Merging / Weaving
CCO Standstill distance	4.92	ft	4.5 - 5.5	> 4.92
CC1 Headway time	0.9	s	0.85 - 1.05	0.90 - 1.50
CC2 'Following' variation	13.12	ft	6.56 - 22.97	13.12 - 39.37
CC3 Threshold for entering 'following'	- 8		use default	
CC4 Negative 'following' threshold	- 0.35		use default	
CC5 Positive 'following' threshold	0.35		use default	
CC6 Speed Dependency of oscillation	11.44		use default	
CC7 Oscillation acceleration	0.82	ft/s^2	use default	
CC8 Standstill acceleration	11.48	ft/s^2	use default	
CC9 Acceleration at 50 mph	4.92	ft/s^2	use default	

The car following calibration parameters used are provided in Table 4.8 and were developed as part of an iterative process. The CC1 and CC2 parameters were changed and all other CC parameters were set as default values. For the validation purposes, the final calibration parameters for the various driver behavior types were incorporated into both the AM and PM peak period models.

TABLE 4.8: VISSIM car following calibrated parameters

Wiedemann 99 Model Parameters	Default	Basic Freeway	Merging / Diverging	Weaving	Ramps	Segment 1	Segment 2	Segment 3
CC0 (Standstill Distance) (ft.)	4.92	4.92	4.92	4.92	4.92	4.92	4.92	4.92
CC1 (Headway Time) (s)	0.9	0.9	0.9	0.9	0.9	**1**	**1.3**	**1.4**
CC2 ('Following' Variation) (ft.)	13.12	13.12	13.12	13.12	13.12	**22.97**	**22.97**	**22.97**
CC3 (Thresh. for Entering Following)	-8	-8	-8	-8	-8	-8	-8	-8
CC4 (Negative 'Following' Threshold)	-0.35	-0.35	-0.35	-0.35	-0.35	-0.35	-0.35	-0.35
CC5 (Positive 'Following' Threshold)	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35
CC6 (Speed depend. of Oscillation)	11.44	11.44	11.44	11.44	11.44	11.44	11.44	11.44
CC7 (Oscillation Acceleration) (ft./s^2)	0.82	0.82	0.82	0.82	0.82	0.82	0.82	0.82
CC8 (Standstill Acceleration) (ft./s^2)	11.48	11.48	11.48	11.48	11.48	11.48	11.48	11.48
CC9 (Acceleration at 50 mph) (ft./s^2)	4.92	4.92	4.92	4.92	4.92	4.92	4.92	4.92

Note: The entries in bold have been updated from the default value.

4.6.2. Lane Change Parameters

The lane change parameters were also adjusted in the model to replicate existing field traffic operations. The below parameters were found to have some effect on driver behavior during a sensitivity analysis.

- Necessary lane change parameters
    - Maximum deceleration
    - -1 ft/s^2 per distance
- Safety distance reduction factor
- Cooperative lane change

The suggested ranges for the lane change parameters from the FDOT Traffic Analysis Handbook (SPO, 2014) and the ODOT Protocol for VISSIM Simulation reports (Mai et al., 2011) are provided in Tables 4.9 and 4.10, respectively.

TABLE 4.9: Lane change parameter suggested ranges by FDOT (SPO, 2014)

Calibration Parameter	Default Value	Suggested Range	
		Basic Segment	Weaving / Merge / Diverge
Lane Change			
Maximum deceleration	-13 ft/s^2 (Own) -9.84 ft/s^2 (Trail)	< - 12 ft/s^2 < - 8 ft/s^2	
- 1 ft/s^2 distance	200 ft (Freeway) 100 ft (Arterial)	> 100 ft > 50 ft	
Accepted deceleration	-3.28 ft/s^2 (Own) -1.64 ft/s^2 (Trail)	< - 2.5 ft/s^2 < - 1.5 ft/s^2	
Waiting time before diffusion	60 s	use default	
Min. headway (front/rear)	1.64 ft	1.5 to 6 ft	
Safety distance reduction factor	0.6	0.1 to 0.9	
Max. deceleration for cooperative braking	-9.84 ft/s^2	-9.84 ft/s^2	
Overtake reduced speed areas	Depends on field observations		
Advanced Merging	checked		
Emergency stop	16.4 ft	Depends on field observations	
Lane change	656.2 ft	> 656.2 feet	
Reduction factor for changing lanes before signal	0.6	default	
Cooperative lane change	Unchecked	Checked especially for freeway merge / diverge areas	

TABLE 4.10: Lane change parameter suggested ranges by ODOT (Mai et al., 2011)

Defaults				
General Behavior Necessary Lane Change (route)	Free Lane Selection			
	Own	Unit	Trailing Vehicle	Unit
Maximum deceleration:	-13.12	ft/s^2	-9.84	ft/s^2
- 1 ft/s^2 per distance:	200	ft	200	ft
Accepted deceleration:	-3.28	ft/s^2	-1.64	ft/s^2
Waiting time before diffusion:			60	s
Min. headway (front/rear):			1.64	ft
To slower lane if collision time above:			0	s
Safety distance reduction factor:			0.6	
Maximum deceleration for cooperative braking:			-9.84	ft/s^2
Overtake reduced speed areas:			☐ *	
Suggested Ranges				
General Behavior Necessary Lane Change (route)	Free Lane Selection			
	Own	Unit	Trailing Vehicle	Unit
Maximum deceleration:	-15 to -12	ft/s^2	-12 to -8	ft/s^2
- 1 ft/s^2 per distance:	150 to 250	ft	150 to 250	ft
Accepted deceleration:	-2.5 to -4	ft/s^2	-1.5 to -2.5	ft/s^2
Waiting time before diffusion:			60	s
Min. headway (front/rear):			1.5 to 2	ft
To slower lane if collision time above:			0 to 0.5	s
Safety distance reduction factor:			0.25 to 1.00	
Maximum deceleration for cooperative braking:			-8 to -15	ft/s^2
Overtake reduced speed areas:			☐ *	

* Leave box un-checked

The lane change calibration parameters used are provided in Table 4.11. The final calibration parameters for the various driver behavior types were incorporated into both the AM and PM peak period models for calibration and validation purposes, respectively. The default value of 60 seconds for “Wait time before diffusion (sec)” was changed to 160 seconds to match the maximum cycle length observed within the AM and PM peak period models. This enabled the vehicles not to diffuse prior to getting the green signal and provided accurate queuing estimates near intersections.

TABLE 4.11: VISSIM lane change calibrated parameters

Lane Change Parameters	**Default**	**Basic Freeway**	**Merge / Diverge**	**Weave**	**Ramps**	**Seg. 1**	**Seg. 2**	**Seg. 3**
Max Decel. (own) (ft./s^2)	-13.12	-13.12	**-15.00**	**-15.00**	-13.12	**-15.00**	**-15.00**	**-15.00**
Max Decel. (trailing) (ft./s^2)	-9.84	-9.84	**-12.00**	**-12.00**	-9.84	**-12.00**	**-12.00**	**-12.00**
-1 fps^2 per dist. (own) (ft.)	200	200	**150**	**150**	200	**150**	**150**	**150**
-1 fps^2 per dist. (trailing) (ft.)	200	200	**150**	**150**	200	**150**	**150**	**150**
Accepted Decel. (own) (ft./s^2)	-3.28	-3.28	-3.28	-3.28	-3.28	-3.28	-3.28	-3.28
Accepted Decel. (trailing) (ft./s^2)	-1.64	-1.64	-1.64	-1.64	-1.64	-1.64	-1.64	-1.64
Wait time before diffusion (sec)	**160**	**160**	**160**	**160**	**160**	**160**	**160**	**160**
Min. headway (front / rear)	1.64	1.64	1.64	1.64	1.64	1.64	1.64	1.64
To slower lane - collision time above (sec)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Safety distance reduction factor	0.60	0.60	0.60	**0.25**	0.60	0.60	0.60	0.60
Max. Decel. for coop. braking (ft./s^2)	-9.84	-9.84	-9.84	-9.84	-9.84	-9.84	-9.84	-9.84
Overtake reduced speed areas	Off	Off	Off	Off	Off	Off	Off	Off
Advanced Merging	On	On	On	On	On	On	On	On
Cooperative Lane Change	Off	**On**	**On**	**On**	**On**	Off	**On**	Off
Lateral correction of rear end position	Off	Off	Off	Off	Off	Off	Off	Off

Note: The entries in bold have been updated from the default value.

4.6.3. Lane Change Distance for Links

The "Lane Change Distance" in VISSIM can significantly affect freeway operations and can be defined as the distance upstream of the merging/diverging area, such as off-ramps and lane drops where drivers will start attempting to change lanes to position them for the conditions ahead. The default lane change distance on a connector is 656.2 ft, which is too short to perform the appropriate lane changes. Therefore, there is a need to adjust these values to match real-world driver reaction points as the commuters often react well before the anticipated conditions ahead of them. The default value in VISSIM was changed to 2,640 ft (0.5 mile) per lane for freeways and 1,320 ft. (0.25 mile) per lane for arterial links.

4.7. Measures of Effectiveness for Calibration

The VISSIM model provides various Measures of Effectiveness (MOEs) to describe the operational performance of a modeled scenario. Several MOEs are available for the comparison of field data to the modeled data, such as vehicular volumes, travel times, speeds, delays, queue lengths, etc. The critical outputs for the calibration of traffic operations on I-295 include the mainline volumes, travel times and speeds, and ramp terminal intersection queues. The various data collection elements that need to be defined to obtain the MOEs of interest are listed as follows.

- Link evaluations: volume, density, and speed information for all the roadways in the VISSIM network.
- Travel time sections: number of vehicles, travel times, and speeds for freeway mainline sections.
- Node evaluations: turning movement volume, delay time, and queue length for study area intersections.

4.8. Number of Simulation Runs Required

Due to the stochastic nature of VISSIM microscopic simulation, the assigned vehicle entry types, and route choices change based on the random seed value. The results obtained from different individual random seeds can vary significantly. It is necessary to run VISSIM models multiple times with different random seeds to gain an accurate reflection of the performance of the models. The VISSIM software has a built-in, multi-run capability and an output processor that records performance measures from each run, and summarizes them. This multi-run feature in VISSIM runs the model multiple times, by changing the random seed number for each run. The output processor collects user defined MOE data for the network over multiple runs and organizes the data into a single database file.

The maximum number of runs required for a simulation depends on two primary variables. They are:

- The variance in the mean of one or more MOEs selected, and
- The tolerable error as selected by the analyst (5-10%).

The formula used to determine the required number of simulation runs is presented below (Sabra and Halkias, 2009):

$$n = \frac{(1.96)^2\sigma^2}{E^2} \qquad \text{Eq 4.10}$$

where,

n = required sample size (e.g., number of simulation runs),

1.96 = Z- value for the standard normal curve for 95% confidence,

σ^2 = sample variance computed from the simulation results, and,

E = tolerable error for the sample mean (in same units as the mean).

The multi-run process requires an initial data set to be generated. An initial sample with 10 simulations runs was performed for both AM and PM peak period models. In most cases, these 10 runs will generate a sample size large enough to produce a true statistical average of the results. However, this was verified using the above equation. The total network delay, average speed, and total network travel time were chosen as the key MOEs to verify if the initial 10 runs were producing “statistically significant” outputs. A 95% confidence interval and an allowable error of 7.5% (average value of allowable range 5 to 10%) were assumed. Table 4.12 shows that both AM and PM peak period models required a maximum of 6 runs to produce “statistically significant” results for the chosen MOEs. Based on this analysis, the model calibration results as well as the MOEs were prepared for the VISSIM models based on the average values from 10 simulation runs with varying random seeds.

TABLE 4.12: Required number of runs computation

Total Network Delay			
Peak Period	**Mean (hr)**	**Std. Deviation (hr)**	**No. of Runs Required**
AM	924.36	84.46	6
PM	1,369.75	118.81	6
Average Speed			
Peak Period	**Mean (mph)**	**Std. Deviation (mph)**	**No. of Runs Required**
AM	44.55	1.02	1
PM	40.08	1.15	1
Total Network Travel Time			
Peak Period	**Mean (hr)**	**Std. Deviation (hr)**	**No. of Runs Required**
AM	3,710.03	90.38	1
PM	4,194.86	114.19	1

4.9. Model Calibration

AM peak period microsimulation model was chosen for performing the calibration process. Both visual examination and evaluation of statistical model outputs were performed to accomplish the calibration process. The model calibration primarily focused on replicating the traffic volume data, travel time/operating speed data, and existing bottlenecks/congestion locations along I-295 based on field observations. This discussion provides a detailed comparison between model outputs and existing data within the context of the calibration criteria.

4.9.1. Hourly Flows (Modeled vs. Observed)

4.9.1.1. Individual Link Flows

As defined by FHWA's Traffic Analysis Toolbox Volume III (Dowling et al., 2004), a link falls under one of the three categories depending on the observed volumes on the roadway segments:

- Low volume links (flow less than 700 vph),
- Medium volume links (flow between 700 vph and 2700 vph), and
- High volume links (flow greater than 2700 vph).

The calibration criteria vary depending on these three volume categories, as summarized in Section 4.2 (Table 4.1). Based on these criteria, the volumes of all the freeway and arterial links within the VISSIM network are compared to the existing traffic volumes for each of the three hours within the peak periods. In addition, all of the individual links were checked to see if they are within the15% threshold of the field observed volume values as calibration criteria requires.

The first calibration check performed on the modeled volumes was to make sure that the modeled traffic volumes are within 15% threshold of the field observed traffic counts for at least 85% of the links within the VISSIM models. Table 4.13 provides a summary of the total number of links analyzed and the number of links that are complying with the identified criteria. This table illustrates that at least 98% of all links during the AM peak period model are within the allowable 15% range during all three hours of the peak period.

TABLE 4.13: Individual link flows – all links (within 15%) – AM peak period

Peak Period	Simulation Hour	Total No. of Links (Observed)	Links within Criteria (Modeled)	Percentage Compliant
AM	Hour 1	199	197	99%
	Hour 2	**199**	**196**	**98%**
	Hour 3	199	195	98%

In addition, low volume links (links with flow < 700 veh/hr) were checked to ensure that the modeled traffic volumes are within 100 veh/hr of the field observed traffic counts for 85% of these link types within the VISSIM model as specified in the FHWA's Traffic Analysis Toolbox (Section 4.2). Table 4.14 provides a summary of the total number of links that fall under this category during the AM peak period and the number of links that are complying with the criteria identified. This table shows that at least 98% of all links that fall under this volume category are within the allowable 100 veh/hr range during all three hours of the peak period.

TABLE 4.14: Individual link flows – low volume links (within 100 vph) – AM peak period

Peak Period	Simulation Hour	Total No. of Links (Observed)	Links within Criteria (Modeled)	Percentage Compliant
AM	Hour 1	73	73	100%
	Hour 2	**43**	**42**	**98%**
	Hour 3	55	54	98%

Medium volume links (links with flow >700 veh/hr and < 2,700 veh/hr) were also checked to ensure that the modeled traffic volumes are within 15% of the field observed traffic counts for 85% of these link types in the VISSIM models. Table 4.15 provides a summary of the total number of links that fall under this category and the number of links that are complying with the identified criteria. This table shows that all links (100%) that fall under this volume category are within the allowable 15% range during all three hours of the peak period.

TABLE 4.15: Individual link flows – medium volume links (within 15%) – AM peak period

Peak Period	Simulation Hour	Total No. of Links (Observed)	Links within Criteria (Modeled)	Percentage Compliant
AM	Hour 1	92	92	100%
	Hour 2	**92**	**92**	**100%**
	Hour 3	84	84	100%

In case of high volume links (links with flow >2,700 veh/hr), the modeled traffic volumes were checked to ensure that they fall within 400 veh/hr of the field observed traffic counts for 85% of these link types within the VISSIM models. Table 4.16 provides a summary of the total number of links that fall under this category during the AM peak period. More than 89% of all links falling under this category met the hourly flow criteria range. In conclusion, the calibrated model had hourly flow compliant percentage of 85% or better across the network for all three hours of the peak period.

TABLE 4.16: Individual link flows – high volume links (within 400 vph) – AM peak period

Peak Period	Simulation Hour	Total No. of Links (Observed)	Links within Criteria (Modeled)	Percentage Compliant
AM	Hour 1	34	34	100%
	Hour 2	**64**	**57**	**89%**
	Hour 3	60	60	100%

Tables A1 to A3 (Appendix A) present detailed link volume calibration information. Overall, the analysis of the modeled versus observed volumes presented above indicates that the individual link flows for each of the three hours of AM peak period met the volume criteria set.

4.9.1.2. Sum of All Link Flows

The sum of individual links flows for all the freeway and arterial links within the VISSIM network were compared to the sum of existing condition traffic volumes for these links. The AM peak period model followed similar trend in terms of percent difference within each hour of the peak period. Table 4.17 provides a summary of the sum of link flows for the AM peak period. At a network level, the percent difference between modeled and observed volume is within the FHWA's 5 percent calibration target criteria. As observed in the individual link flows, the percent difference of sum of all link flows (modeled versus observed) is slightly higher for hour 2 (3.99 percent) as compared to hours 1 and 3.The traffic less than the peak hour traffic was simulated in hour 1 and all of this traffic demand was served. During hour 2, more traffic is entering the system, and bottlenecks and speed drop zones were being developed within the network. Therefore, the amount of traffic simulated when compared to the observed volume is lesser during hour 2. In the third hour, the congestion was being relieved and the simulated volumes increased. These observations are reflective of the field conditions observed.

TABLE 4.17: Sum of link flows – AM peak period

Peak Period	Simulation Hour	Sum of all Link Flows (Observed)	Sum of all Link Flows (Modeled)	Difference	Difference Percentage (%)
AM	Hour 1	275,841	283,058	7,217	2.62%
	Hour 2	**429,790**	**412,648**	**-17,142**	**-3.99%**
	Hour 3	373,699	378,431	4,732	1.27%

4.9.1.3. GEH Statistic

The GEH statistic is an empirical formula used in traffic engineering to compare two sets of traffic volumes. The GEH statistic aids in avoiding pitfalls that occur when using simple percentages to compare two sets of traffic volumes. A GEH value of less than 5.0 is considered a good fit between the hourly input volumes and the modeled volumes. The calibrated model had GEH compliant percentage of 85% or better across the network. Table 4.18 summarizes the GEH statistic summary.

TABLE 4.18: GEH statistic summary – AM peak period

Peak Period	**Simulation Hour**	**Total Number of Links**	**Links with GEH < 5**	**Percentage (%)**	**Within 85%**
AM	Hour 1	199	196	98%	Yes
	Hour 2	**199**	**186**	**93%**	**Yes**
	Hour 3	199	198	99%	Yes

4.9.2. Travel Time and Speed Data

4.9.2.1. Travel Times (Modeled vs. Observed)

Travel time segments were defined in the calibrated model in compliance with the field data collection segments to generate the modeled travel time information for the comparison. To identify and quantify congestion along I-295 corridor, travel time data were collected in the field using probe vehicles to assist in the calibration process (Sub-sub-section 4.4.5.2). The modeled travel times along I-295 and SR 202 mainline were compared with the field collected travel time information. Table 4.19 shows the travel time calibration results compared with the field observation.

The criteria for travel time are met if the modeled travel times are within 15% of the field measured travel time information. However, based on field observations and travel time runs, the travel time variability ranges between 3 to 5 minutes in certain congested segments (between US-1 and SR 152). In order to reasonably account for the temporal variation of travel times, the modeled travel times were also checked to see if they fall within one standard deviation of the field observed travel time values.

TABLE 4.19: Travel time comparison – AM peak hour

I-295 Corridor

	No.	From	To	Distance (mi)	AM Travel Time: Observed Travel Time (sec): Average Value Sec	Average Value Min	15% Range Upper	15% Range Lower	1 SD Range Upper	1 SD Range Lower	Criteria Used Upper	Criteria Used Lower	Modeled Average Travel Sec	Modeled Average Travel Min	Threshold Met?
NB	1	Begin Project	US-1 On-Ramp	0.64	34	0.56	39	29	35	32	39	29	29	0.48	YES
	2	US-1 On-Ramp	SR 9B On-Ramp	1.63	102	1.69	117	86	121	82	121	82	109	1.82	YES
	3	SR 9B On-Ramp	SR 115 (Baymeadows Rd.) On-Ramp	2.79	294	4.90	338	250	407	181	407	181	362	6.03	YES
	4	SR 115 (Baymeadows Rd.) On-Ramp	Gate Pkwy. On-Ramp	1.38	88	1.46	101	75	104	72	104	72	74	1.23	YES
	5	Gate Pkwy. On-Ramp	SR 202 (JTB) WB On-Ramp	1.27	69	1.15	80	59	77	61	80	59	72	1.20	YES
	6	SR 202 (JTB) WB On-Ramp	End Project	1.12	67	1.11	77	57	71	63	77	57	66	1.10	YES
	7	Entire Corridor		8.83	653	10.89							673	11.22	

	No.	From	To	Distance (mi)	AM Travel Time: Observed Travel Time (sec): Average Value Sec	Average Value Min	15% Range Upper	15% Range Lower	1 SD Range Upper	1 SD Range Lower	Criteria Used Upper	Criteria Used Lower	Modeled Average Travel Sec	Modeled Average Travel Min	Threshold Met?
SB	8	Begin Project	Town Center Pkwy. On-Ramp	0.77	54	0.89	62	46	63	44	63	44	63	1.05	YES
	9	Town Center Pkwy. On-Ramp	SR 202 (JTB) EB On-Ramp	1.21	66	1.10	76	56	73	60	76	56	68	1.13	YES
	10	SR 202 (JTB) EB On-Ramp	Gate Pkwy. On-Ramp	1.14	65	1.09	75	55	73	57	75	55	68	1.13	YES
	11	Gate Pkwy. On-Ramp	SR 115 (Baymeadows Rd.) On-Ramp	1.41	73	1.22	84	62	77	69	84	62	87	1.45	NO
	12	SR 115 (Baymeadows Rd.) On-Ramp	SR 9B Off-Ramp	1.67	92	1.53	106	78	101	83	106	78	97	1.62	YES
	13	SR 9B Off-Ramp	End Project	2.55	127	2.11	146	108	130	124	146	108	137	2.28	YES
	14	Entire Corridor		8.75	477	7.95							518	8.63	

SR 202 (JTB) Corridor

	No.	From	To	Distance (mi)	AM Travel Time: Observed Travel Time (sec): Average Value Sec	Average Value Min	15% Range Upper	15% Range Lower	1 SD Range Upper	1 SD Range Lower	Criteria Used Upper	Criteria Used Lower	Modeled Average Travel Sec	Modeled Average Travel Min	Threshold Met?
EB	15	Begin Project	Gate Pkwy. On-Ramp	1.19	64	1.06	73	54	69	58	73	54	67	1.12	YES
	16	Gate Pkwy. On-Ramp	I-295 NB On-Ramp	1.11	59	0.99	68	50	61	58	68	50	62	1.03	YES
	17	I-295 NB On-Ramp	End Project	1.21	68	1.14	78	58	71	66	78	58	75	1.25	YES
	18	Entire Corridor		3.51	191	3.19							197	3.28	

	No.	From	To	Distance (mi)	AM Travel Time: Observed Travel Time (sec): Average Value Sec	Average Value Min	15% Range Upper	15% Range Lower	1 SD Upper	1 SD Lower	TT Range Upper	TT Range Lower	Modeled Average Travel Sec	Modeled Average Travel Min	Threshold Met?
WB	19	Begin Project	Kernan Blvd. On-Ramp	0.84	48	0.79	55	40	52	43	55	40	49	0.82	YES
	20	Kernan Blvd. On-Ramp	I-295 SB On-Ramp	1.23	63	1.05	72	54	64	62	72	54	70	1.17	YES
	21	I-295 SB On-Ramp	End Project	1.44	79	1.32	91	67	91	67	91	67	85	1.42	YES
	22	Entire Corridor		3.51	190	3.16							198	3.30	

Travel Times meet threshold for > 85% of times: 94%

The calibrated model for the AM peak period met the criteria for travel time as 94% of the freeway simulated sections evaluated were either within the 15% range or one standard deviation (for sections with significant travel time variability) of the average travel time observed in the field. The AM peak models reflected travel times that are within one minute of the field observed travel time on the section where the travel time calibration could not be achieved. Therefore, it can be concluded that the AM peak model accurately reflected the field observed travel times.

4.9.2.2. Speed (Modeled vs. Observed)

The speeds along I-295 were calculated utilizing the travel time data collected in the field using probe vehicles outlined in Sub-sub-section 4.4.5.2. The field observed speeds between various sections along I-295 were compared with the modeled speed data. The pre-peak hour, peak hour, and post-peak hour (Hour 1, 2, and 3, respectively) modeled speeds along mainline I-295 were compared with the field collected speed data during peak hour (Hour 2). The pre-peak hour (Hour 1) and post-peak hour (Hour 3) existing conditions data was not available for comparison. Figure 4.4 shows the speed chart comparing the modeled versus observed speeds for AM peak hour model.

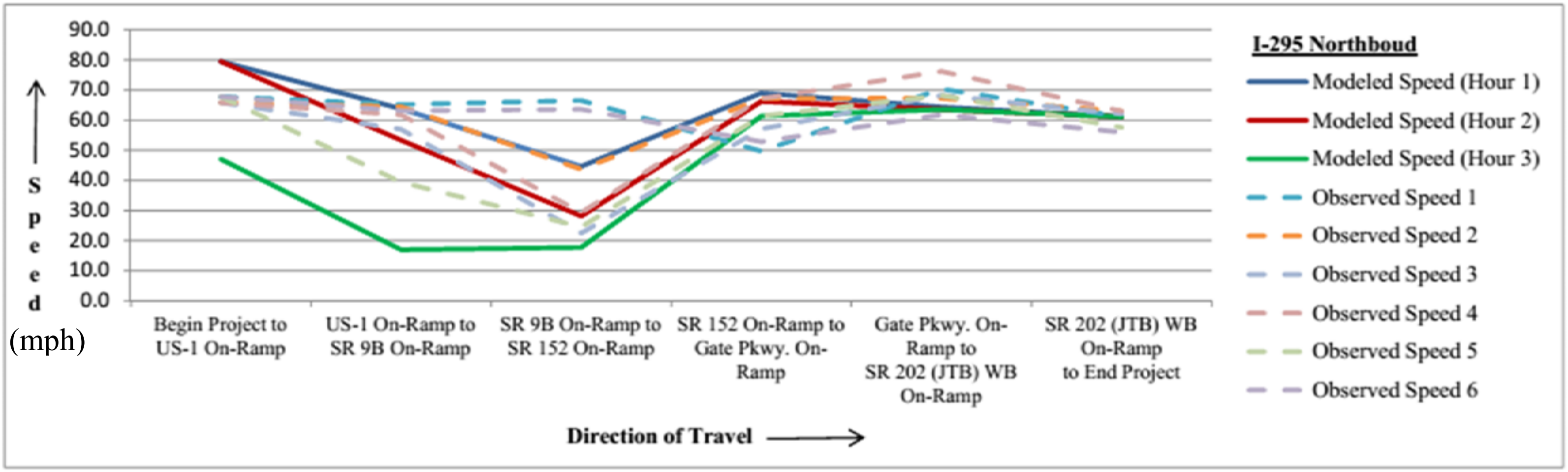
During the AM peak hour, along I-295 northbound direction, the modeled peak hour speeds and field speed data show similar variation trends. The simulation produced congestion between SR 5 and SR 152 accurately that was observed in the field. Along I-295 southbound direction, the modeled peak hour speeds shows similar trends as the field observed speed data between Town Center Parkway and Gate Parkway. However, the trend between Gate Parkway and SR 5 differs slightly from the field observed speed data.

By comparing the pre-peak hour, peak hour, and post-peak hour speeds during the AM peak period, it is noted that along the I-295 northbound direction, Hour 3 showed lower speeds than the peak hour (Hour 2) and the bottleneck areas are not completely cleared. This observation was checked with Google™ Travel Time information that was available online and was found to be consistent with the simulation model where the congestion was extending into the third hour. However, for the current modeling effort, the simulation time period was not extended beyond Hour 3 as no unmet demand was observed in the model along I-295. Along the southbound direction, the pre-peak hour (Hour 1) and the post-peak hour (Hour 3) showed higher speeds and lesser congestion than the peak hour (Hour 2), except between the SR 202 and Gate Parkway on-ramps.

4.9.3. Visual Audits for Bottlenecks

Visual audits of the simulation runs were performed to verify the formation of bottlenecks/queues in the AM peak period VISSIM model. Field visits were conducted to identify visual bottlenecks, speed drop zones, maximum back of queues along I-295 mainline, and queuing along I-295 off-ramps. An overview of the critical areas during the AM peak period is summarized below:

- I-295 Northbound direction: SR 9B at I-295 is a major merge location. At this location, the outside lane from SR 9B onto I-295 is dropped, followed in succession by an inside lane drop along I-295. This creates a major bottleneck in the AM peak hour, especially when the I-295 northbound traffic is heavy.
- I-295 Southbound direction: No major bottlenecks were observed.



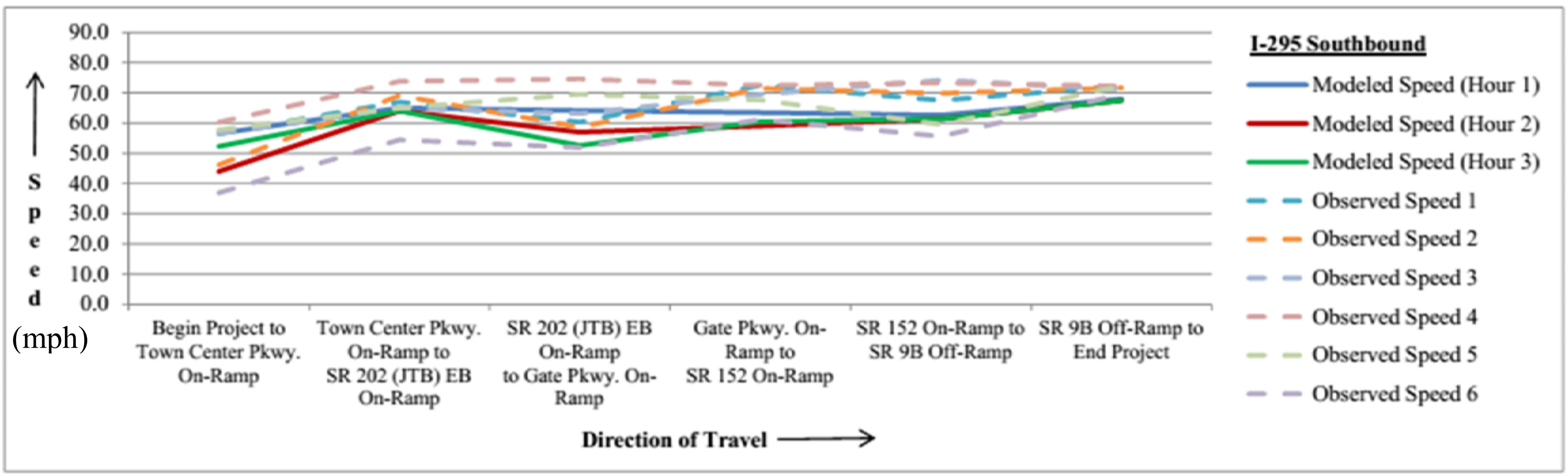


FIGURE 4.4: AM peak hour speed chart

Based on the visual audits performed, the AM peak period simulation model reasonably replicates the bottlenecks, speed drop zones, maximum back of queues along I-295 mainline, and queuing along I-295 off-ramps. Figure 4.5 shows the AM peak hour VISSIM simulation in relation to the field observed bottleneck locations.

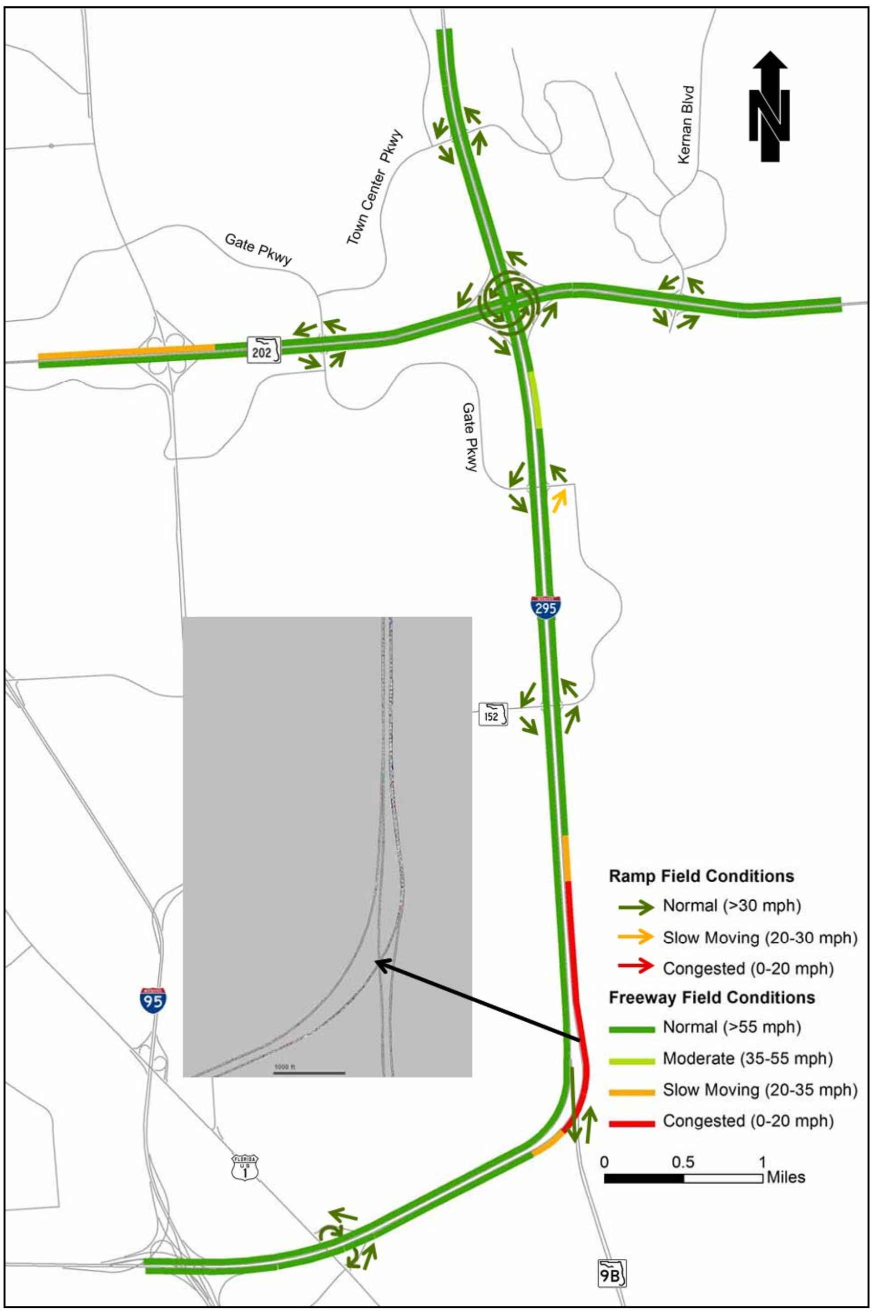


FIGURE 4.5: Visual audits – AM peak hour bottleneck (simulated vs. observed)

4.9.4. Ramp Terminal Queues

Node evaluation files were processed from the AM peak period model to produce the modeled queue length information. A queue study was performed by FDOT for all ramp terminal intersections within the study area as discussed in Sub-sub-section 4.4.5.4. Field visits were conducted to validate and verify the provided queue data. Adjustments were made to the intersection queue data near locations that showed considerable difference in queue lengths between the queue study and field observations. This field verified queue data were used for the calibration of the ramp terminal intersection queues.

The modeled queue lengths were compared to the average queue lengths near the off-ramp (diverging) areas. The modeled values are considered to be within the acceptable range of the observed queue length if they fall within the 95th percentile queue value. The 95th percentile queue value range was calculated as being equal to the average queue plus or minus 1.65 standard deviations. Tables 4.20 shows the ramp terminal intersections queue calibration results for AM peak hour model.

4.9.5. Calibration Summary

The FHWA's Traffic Analysis Toolbox Volume III (SPO, 2014) calibration criteria for traffic volumes, travel times, speeds, and ramp terminal queues for the AM peak period VISSIM model were met. The visual audits of the VISSIM simulation showed buildup and dissipation of congestion consistent with the field observations. The VISSIM model accurately reflected the existing traffic operations during the AM peak period along I-295.

TABLE 4.20: Off-ramp queue length comparison – AM peak

Number	Intersection ID	Intersection	Ramp	Movement	Observed Queue Length			Modeled Average Queue Length (ft)	Within 1.65 SD Range
					Average Value (ft)	1.65 SD Range (ft)			
						Upper Limit	Lower Limit		
1	T2	Node 102: US-1 & I-295 NB Ramps/Greenland Rd.	I-295 NB Off-Ramp	Left	270	385	155	265	YES
				Through	103	188	17	89	YES
				Right	43	136	0	2	YES
2	T1	Node 103: US-1 & I-295 SB Ramps	I-295 SB Off-Ramp	Left	177	266	88	106	YES
				Right	0	10	0	9	YES
3	T4	Node 202: Baymeadows Rd. & I-295 SB Ramps	I-295 SB Off-Ramp	Left	58	123	0	9	YES
				Right	46	184	0	4	YES
4	T5	Node 203: Baymeadows Rd. & I-295 NB Ramps	I-295 NB Off-Ramp	Left	79	163	0	30	YES
				Right	2	14	0	0	YES
5	T7	Node 302: Gate Pkwy. & I-295 SB Ramps	I-295 SB Off-Ramp	Left	110	196	23	64	YES
				Right	6	24	0	5	YES
6	T8	Node 303: Gate Pkwy. & I-295 NB Ramps	I-295 NB Off-Ramp	Left	800+	1000+	600+	1,104	YES
				Right	0	10	0	0	YES
7	T10	Node 402: Gate Pkwy. & SR 202 (JTB)	SR 202 (JTB) EB Off-Ramp	Left	92	190	0	45	YES
				Right	73	151	0	0	YES
8			SR 202 (JTB) WB Off-Ramp	Left	419	561	277	644	NO
				Right	0	10	0	424	NO
9	T12	Node 501: Kernan Blvd. & SR 202 (JTB)	SR 202 (JTB) EB Off-Ramp	Left	0	10	0	0	YES
				Right	0	10	0	0	YES
10	T11	Node 502: Kernan Blvd. & SR 202 (JTB)	SR 202 (JTB) WB Off-Ramp	Left	0	10	0	0	YES
				Right	23	71	0	8	YES
11	T9	Node 602: Town Center Pkwy. & I-295	I-295 NB Off-Ramp	Left	158	258	58	44	NO
				Right	17	49	0	0	YES
12			I-295 SB Off-Ramp	Left	54	106	2	12	YES
				Right	52	144	0	11	YES
Count of Values within range									**22**
Off-Ramp Queues (within range)									**88%**

Note: SD = Standard deviation

4.10. Validation of Calibration Parameters

For the validation purposes, the same base model developed for the AM peak period simulation model was used for PM peak period model. Roadway geometry, speed distributions, vehicular compositions, and control devices were kept identical to the calibrated model. All the calibrated parameters were kept identical to validate the parameters using the PM peak period data and by comparing the simulated and field observed hourly flows, travel times, and speeds.

Similar to the calibrated model, a 3.5 hour peak period VISSIM model that depicts buildup and dissipation of congestion along I-295 within the study area was created, which includes 30 minutes of seeding time period to load the network with traffic to reach equilibrium between the number of vehicles entering and exiting the network, an hour prior to the peak hour, the actual peak hour, and an hour after the peak hour to recover from congestion. The input volumes in PM peak period VISSIM model were developed from the peak hour volumes and off-peak volume distribution percentages from the DTM provided by FDOT, described in Sub-sub-section 4.4.5.1. Volumes were entered into VISSIM as 15-minute flow rates through the entry links.

4.10.1. Hourly Flows (Modeled vs. Observed)

4.10.1.1. Individual Link Flows

The first validation check performed on the validation model was to make sure that the modeled traffic volumes are within 15% threshold of the field observed traffic counts for at least 85% of the links within the VISSIM models. Table 4.21 provides a summary of the total number of links analyzed and the number of links that are complying with the criteria identified. Similar to the AM peak period calibrated model, this table also illustrates that at least 98% of all links in the PM peak period model (validation model) are within the allowable 15% range during all three hours of the peak period.

TABLE 4.21: Individual link flows – all links (within 15%) – PM peak period

Peak Period	**Simulation Hour**	**Total No. of Links (Observed)**	**Links within Criteria (Modeled)**	**Percentage Compliant**
PM	Hour 1	199	196	98%
	Hour 2	**199**	**195**	**98%**
	Hour 3	199	195	98%

In case of low volume links (links with flow < 700 veh/hr), Table 4.22 provides a summary of the total number of links that fall under this category during the PM peak period and the number of links that are complying with the criteria identified. This table shows that all links which fall under this volume category were within the allowable 100 veh/hr range during all three hours of the peak period.

Medium volume links (links with flow >700 veh/hr and < 2,700 veh/hr) were also validated to ensure that the modeled traffic volumes are within 15% of the field observed traffic counts for 85% of these link types within the VISSIM models. Table 4.23 provides a summary of the total number of links that fall under this category. This table shows that all links (100%) which fall under this volume category were within the allowable 15% range during all three hours of the peak period.

TABLE 4.22: Individual link flows – low volume links (within 100 vph) – PM peak period

Peak Period	**Simulation Hour**	**Total No. of Links (Observed)**	**Links within Criteria (Modeled)**	**Percentage Compliant**
PM	Hour 1	53	53	100%
	Hour 2	**41**	**41**	**100%**
	Hour 3	54	54	100%

TABLE 4.23: Individual link flows – medium volume links (within 15%) – PM peak period

Peak Period	Simulation Hour	Total No. of Links (Observed)	Links within Criteria (Modeled)	Percentage Compliant
PM	Hour 1	86	86	100%
	Hour 2	**90**	**90**	**100%**
	Hour 3	89	89	100%

In case of high volume links (links with flow >2,700 veh/hr), the modeled traffic volumes were checked to ensure that they fall within 400 veh/hr of the field observed traffic counts for 85% of these link types within the VISSIM models. Table 4.24 provides a summary of the total number of links that fall under this category and shows that more than 85% of all links falling under this category met the volume criteria range. In conclusion, the validation model had hourly flow compliant percentage of 85% or better across the network for all three hours of the peak period.

Tables A4 to A6 (Appendix A) present detailed link flow validation information. In conclusion, the analysis of the modeled versus observed hourly flows presented above indicates that the individual link flows for each of the three hours of PM peak period met the hourly flows collected from the field.

TABLE 4.24: Individual link flows – high volume links (within 400 vph) – PM peak period

Peak Period	Simulation Hour	Total No. of Links (Observed)	Links within Criteria (Modeled)	Percentage Compliant
PM	Hour 1	60	60	100%
	Hour 2	**68**	**58**	**85%**
	Hour 3	56	56	100%

4.10.1.2. Sum of All Link Flows

Similar to the calibration process, the sum of all individual link flows of the validation model for all the freeway and arterial links within the VISSIM network were compared to the sum of existing condition traffic flows for all these links. Table 4.25 provides a summary of the sum of link flows for the PM peak period. At a network level, the percent difference between modeled and observed volume was within the FHWA's 5 percent calibration target criteria. As observed in the individual link flows, the percent difference of sum of all link flows (modeled versus observed) is slightly higher for hour 2 (3.93 percent) as compared to hours 1 and 3. Similar to the calibrated model, traffic less than the peak hour traffic is simulated in hour 1 and all of this traffic demand was served. As hour 2 served more volume, bottlenecks and speed drop zones were developed within the network. Therefore, the amount of traffic simulated when compared to the observed volume is lesser during hour 2. In the third hour, the congestion was being relieved and the simulated volumes increased, which are reflective of the field conditions observed.

TABLE 4.25: Sum of link flows – PM peak period

Peak Period	Simulation Hour	Sum of all Link Flows (Observed)	Sum of all Link Flows (Modeled)	Difference	Difference Percentage (%)
PM	Hour 1	373,752	374,848	1,096	0.29%
	Hour 2	**440,950**	**423,626**	**-17,324**	**-3.93%**
	Hour 3	366,685	377,178	10,493	2.86%

4.10.1.3. GEH Statistics

Table 4.26 summarizes the GEH statistic for the validation model. The validation model had GEH compliant percentage of 93% or better across the network illustrating a good fit between the hourly input volumes and the modeled volumes.

TABLE 4.26: GEH statistic summary – PM peak period

Peak Period	**Simulation Hour**	**Total Number of Links**	**Links with GEH < 5**	**Percentage (%)**	**Within 85%**
AM	Hour 1	199	196	98%	Yes
	Hour 2	**199**	**186**	**93%**	**Yes**
	Hour 3	199	198	99%	Yes

4.10.2. Travel Time and Speed Data

4.10.2.1. Travel Times (Modeled vs. Observed)

Travel time segments were defined in the validation model in compliance with the calibrated model segments. As described in Sub-sub-section 4.4.5.2, the probe vehicle travel time data were collected and compared for the validation purpose. The modeled travel times along I-295 and SR 202 mainline were compared with the field collected travel time information. Table 4.27 shows the travel time calibration results compared to that of the field observation.

Approximately 94% of the freeway segments evaluated produced modeled values that were either within the 15% range or one standard deviation (for sections with significant travel time variability) of the average travel time observed in the field. The modeled travel times were within one minute of the field observed travel times on segments where the 15% range could not be met. In conclusion, the PM peak hour model accurately reflected the field observed travel times.

TABLE 4.27: Travel time comparison – PM peak hour

I-295 Corridor															
				PM Travel Time											
	No.	From	To	Distance (mi)	Observed Travel Time (sec)								Modeled Average Travel		Threshold Met?
					Average Value		15% Range		1 SD Range		Criteria Used				
					Sec	Min	Upper	Lower	Upper	Lower	Upper	Lower	Sec	Min	
NB	1	Begin Project	US-1 On-Ramp	0.64	33	0.54	38	28	34	31	38	28	29	0.48	YES
	2	US-1 On-Ramp	SR 9B On-Ramp	1.63	91	1.52	105	77	96	86	105	77	93	1.55	YES
	3	SR 9B On-Ramp	SR 115 (Baymeadows Rd.) On-Ramp	2.79	179	2.98	206	152	231	127	231	127	163	2.72	YES
	4	SR 115 (Baymeadows Rd.) On-Ramp	Gate Pkwy. On-Ramp	1.38	86	1.44	99	73	100	73	100	73	87	1.45	YES
	5	Gate Pkwy. On-Ramp	SR 202 (JTB) WB On-Ramp	1.27	121	2.01	139	102	181	60	181	60	175	2.92	YES
	6	SR 202 (JTB) WB On-Ramp	End Project	1.12	85	1.42	98	72	110	60	110	60	69	1.15	YES
	7	Entire Corridor		8.83	595	9.91							606	10.10	
				PM Travel Time											
	No.	From	To	Distance (mi)	Observed Travel Time (sec)								Modeled Average Travel		Threshold Met?
					Average Value		15% Range		1 SD Range		Criteria Used				
					Sec	Min	Upper	Lower	Upper	Lower	Upper	Lower	Sec	Min	
SB	8	Begin Project	Town Center Pkwy. On-Ramp	0.77	46	0.76	53	39	47	45	53	39	47	0.78	YES
	9	Town Center Pkwy. On-Ramp	SR 202 (JTB) EB On-Ramp	1.21	241	4.02	277	205	364	119	364	119	68	1.13	NO
	10	SR 202 (JTB) EB On-Ramp	Gate Pkwy. On-Ramp	1.14	272	4.54	313	231	338	206	338	206	84	1.40	NO
	11	Gate Pkwy. On-Ramp	SR 115 (Baymeadows Rd.) On-Ramp	1.41	122	2.04	141	104	156	89	156	89	84	1.40	NO
	12	SR 115 (Baymeadows Rd.) On-Ramp	SR 9B Off-Ramp	1.67	104	1.74	120	89	117	91	120	89	110	1.83	YES
	13	SR 9B Off-Ramp	End Project	2.55	129	2.15	148	110	137	121	148	110	140	2.33	YES
	14	Entire Corridor		8.75	915	15.24							529	8.82	
SR 202 (JTB) Corridor															
				PM Travel Time											
	No.	From	To	Distance (mi)	Observed Travel Time (sec)								Modeled Average Travel		Threshold Met?
					Average Value		15% Range		1 SD Range		Criteria Used				
					Sec	Min	Upper	Lower	Upper	Lower	Upper	Lower	Sec	Min	
EB	15	Begin Project	Gate Pkwy. On-Ramp	1.19	80	1.33	92	68	83	77	92	68	74	1.23	YES
	16	Gate Pkwy. On-Ramp	I-295 NB On-Ramp	1.11	69	1.15	79	58	75	62	79	58	70	1.17	YES
	17	I-295 NB On-Ramp	End Project	1.21	72	1.19	82	61	73	71	82	61	79	1.32	YES
	18	Entire Corridor		3.51	220	3.67							217	3.62	
				PM Travel Time											
	No.	From	To	Distance (mi)	Observed Travel Time (sec)								Modeled Average Travel		Threshold Met?
					Average Value		15% Range		1 SD		TT Range				
					Sec	Min	Upper	Lower	Upper	Lower	Upper	Lower	Sec	Min	
WB	19	Begin Project	Kernan Blvd. On-Ramp	0.84	53	0.88	61	45	54	51	61	45	49	0.82	YES
	20	Kernan Blvd. On-Ramp	I-295 SB On-Ramp	1.23	63	1.05	72	54	64	62	72	54	68	1.13	YES
	21	I-295 SB On-Ramp	End Project	1.44	84	1.40	97	72	89	80	97	72	81	1.35	YES
	22	Entire Corridor		3.51	200	3.33							193	3.22	
Travel Times meet threshold for > 85% of times															85%

4.10.2.2. Speed (Modeled vs. Observed)

The pre-peak hour, peak hour, and post-peak hour (Hour 1, 2, and 3, respectively) modeled speeds between different sections along mainline I-295 were compared with the field collected speed data during peak hour (Hour 2). Figure 4.6 shows the speed chart comparing the modeled versus observed speeds for PM peak period model.

The PM peak period model shows similar trend during peak hour in variation of speed along the southbound direction between SR 202 and SR 5. In addition, along the northbound direction the model shows similar trends as the field observed speeds between Gate Parkway and Town Center Parkway.

4.11. Summary

The AM peak period model was calibrated based on criteria provided by FHWA's Traffic Analysis Toolbox Volume III (SPO, 2014). The validation of the calibrated parameters was performed by implementing the calibrated parameters on the PM peak period model and comparing them with field observed data. The PM peak period model closely resemblances the field condition, which validates the calibration process.

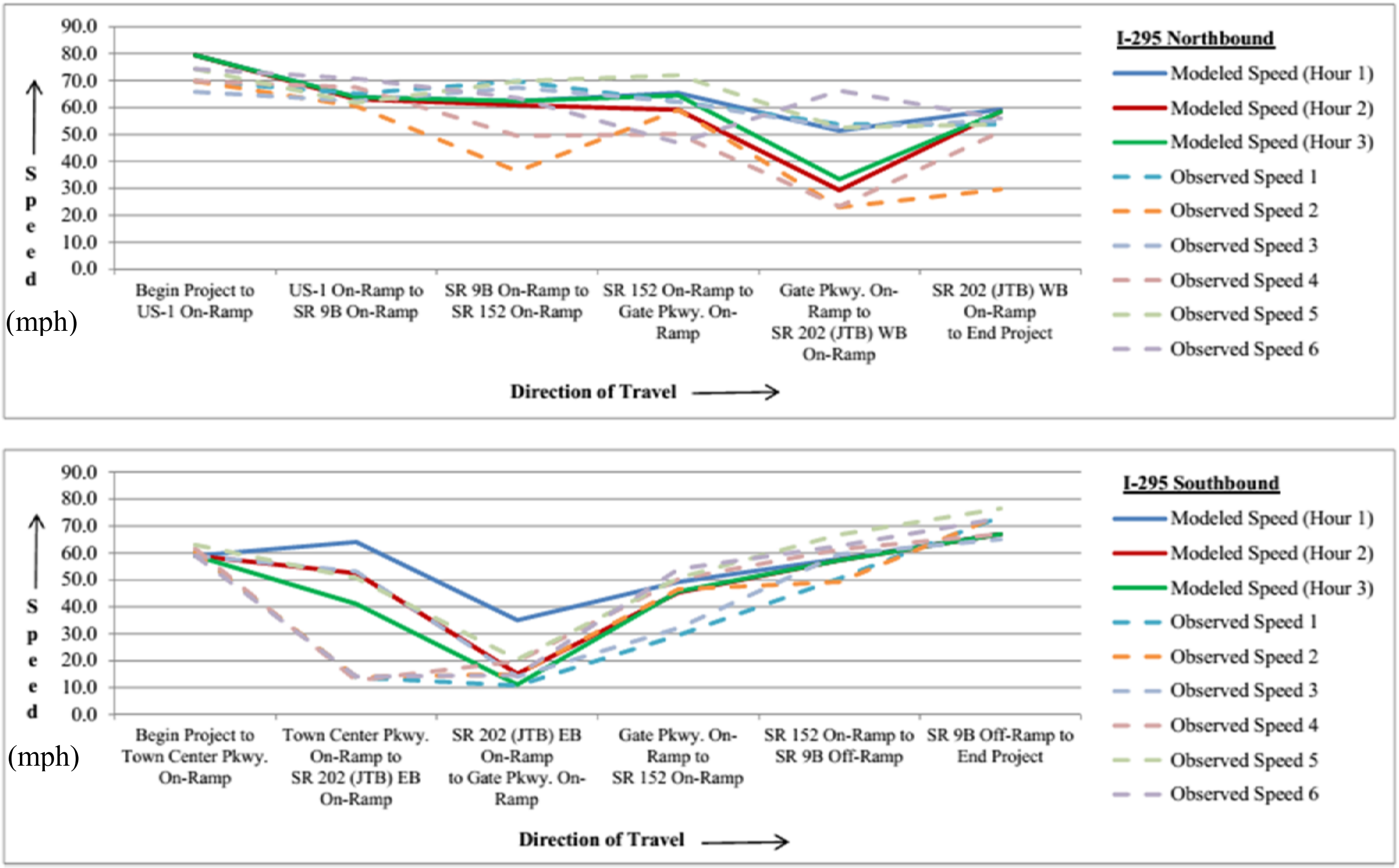


FIGURE 4.6: PM peak hour speed chart

CHAPTER 5: DENSITY – TRAVEL TIME BASED METRICS: RELATIONSHIPS AND ANALYSES

5.1. Introduction

The development, calibration, and validation of microscopic simulation model were discussed in the previous chapter. This chapter focuses on the implementation of the calibrated parameters using hypothetical freeway sections to establish density – travel time and density – travel time reliability relationships for different freeway posted speed limits.

5.2. Hypothetical Freeway Section Description

Several hypothetical sections were considered for the establishment of density – travel time and density – travel time reliability relationships. The simulations were performed based on different speed limit criteria on the freeway. The three freeway section types (basic freeway section, weaving section, and merging/diverging area) were evaluated based on demand fluctuations, different lane configurations, and lengths. Figures 5.1 and 5.2 illustrate the VISSIM hypothetical section representations for basic freeway section and merging/diverging area, and weaving section, respectively.

There are three types of geometric configurations for weaving sections defined in HCM (TRB, 2010).

- Type A: Weaving vehicles in both directions must make one lane change to successfully complete a weaving maneuver.

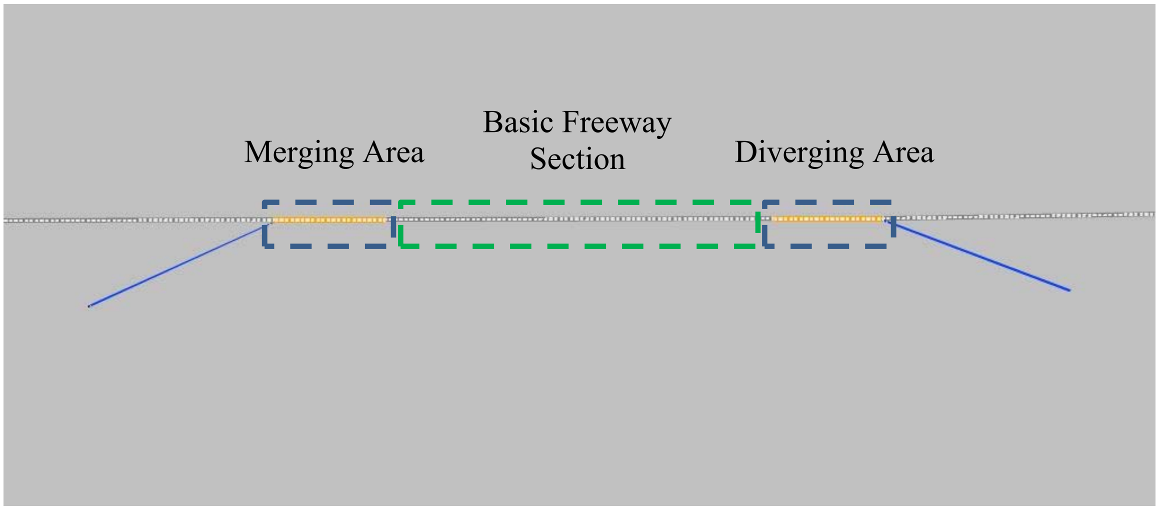


FIGURE 5.1: VISSIM basic freeway section and merging/diverging area illustration

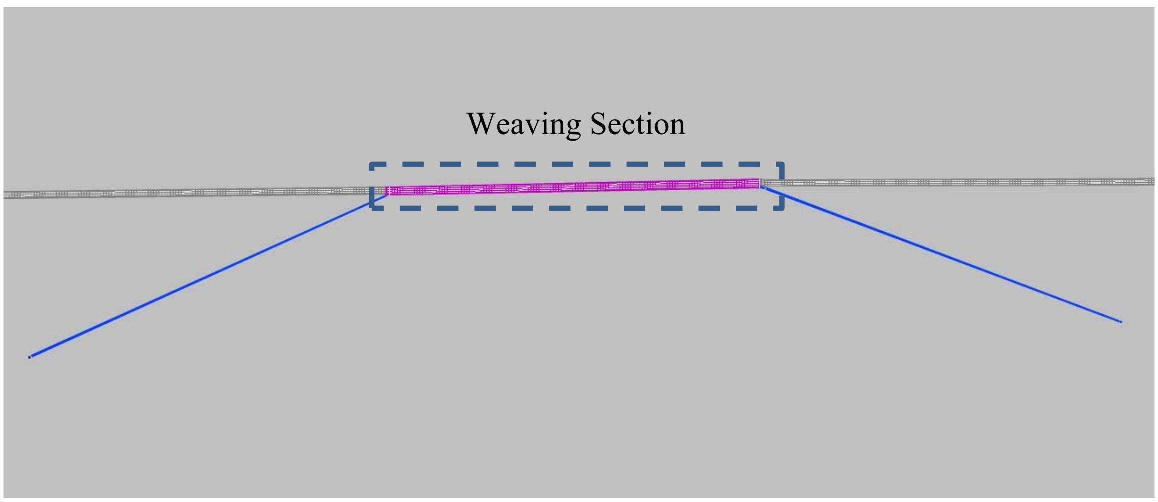


FIGURE 5.2: VISSIM weaving section illustration

- Type B: Weaving vehicles in one direction may complete a weaving maneuver without making a lane change, whereas other vehicles in the weaving segment must make one lane change to successfully complete a weaving maneuver.
- Type C: Weaving vehicles in one direction may complete a weaving maneuver without making a lane change, whereas other vehicles in the weaving segment must make two or more lane changes to successfully complete a weaving maneuver.

In this research, type A weaving section configuration was considered as this type of weaving section is most commonly used on field and involves complex weaving movements. The number of lane is another important factor. Several lane configurations were used for all freeway sections types. Single lane ramp configuration was used in the hypothetical section development. The length of the freeway section can also play an important role in the travel time analysis. Therefore, different length configurations were also evaluated for basic freeway section and weaving section. However, according to Geometric Design of Roads Handbook (Wolhuter, 2015), the influence areas of merging and diverging areas are, respectively, 1,500 feet downstream of a merge and 1,500 feet upstream of a diverge measured from where the edges meet. Therefore, the merging/diverging areas were restricted at 1,500 feet in the analysis. Table 5.1 provides a summary of the network configuration considered.

TABLE 5.1: Hypothetical sections characteristics considered

Factors	Basic Freeway Section	Weaving Section	Merging/Diverging Area
No. of Lanes	4	5	5
	3	4	4
	2	3	3
Length	2,640 feet (0.50 mile)	2,640 feet (0.50 mile)	1,500 feet (0.28 mile)
	5,280 feet (1.00 mile)	3,000 feet (0.55 mile)	
	10,560 feet (2.00 miles)	5,280 feet (1.00 mile)	

Four freeway posted limits (70 mph, 65 mph, 60 mph, and 55 mph) were considered in the analysis with a ±12% tolerance limit. The on- and off- ramp speed limits were kept at 45 mph with a ±12% tolerance limit as the speed on the ramps are of relatively little relevance to the analysis. The road users generally pick up the freeway posted speed limit while entering a merging area or stays at freeway posted speed limit on a diverging area. Therefore, the speed limit at the ramps does not influence the traffic operation at merging/diverging areas. Hence, different types of ramp configuration were also not considered. Demand on the freeway and on- and off-ramp were fluctuated to obtain the randomness, in general. Three different mainline demand conditions (off-peak, mid-day, and peak) were evaluated based on 15-min flow rate for each combination. The ramp demands were considered 2, 5, and 10% of the mainline demand for different combinations. Table 5.2 describes the summary of the factors considered to achieve the aforementioned relationships.

TABLE 5.2: Factors considered for hypothetical freeway sections

Factors	Basic Freeway Section	Weaving Section	Merging/Diverging Area
Speed Profile	70 mph (Upper Bound: 78.4 mph, Lower Bound: 61.6 mph)		
	65 mph (Upper Bound: 72.8 mph, Lower Bound: 57.2 mph)		
	60 mph (Upper Bound: 67.2 mph, Lower Bound: 52.8 mph)		
	55 mph (Upper Bound: 61.6 mph, Lower Bound: 48.4 mph)		
Ramp Speed Profile	45 mph (Upper Bound: 50.40 mph, Lower Bound: 39.60 mph)		
Demand	Off-Peak, Mid-Day, Peak		
Ramp Demand	2%, 5%, 10% of Off-Peak Mainline Demand		
	2%, 5%, 10% of Mid-Day Mainline Demand		
	2%, 5%, 10% of Peak Mainline Demand		
Heavy Vehicles	3%		

5.3. Density Comparisons

A statistical analysis was performed to examine the correlation between the density values obtained from the microscopic simulation software (VISSIM) and HCM method based macroscopic software (HCS) for each combination. Table 5.3 indicates that there is a strong correlation exists between the two densities, which further validates and strengthens the applicability of the research method to develop proposed LOS thresholds.

TABLE 5.3: VISSIM densities vs. HCM densities – correlation coefficients

Section Type		Speed Limit - 70 mph			
		4 Lanes	3 Lanes	2 Lanes	Overall
Basic Freeway Section		0.84	0.84	0.79	0.81
		5 Lanes	4 Lanes	3 Lanes	Overall
Weaving Section		0.87	0.89	0.93	0.89
Merging / Diverging Area	Merging	0.87	0.88	0.87	0.78
	Diverging	0.68	0.71	0.76	0.71
	Combined	0.77	0.74	0.64	0.69
Section Type		Speed Limit - 65 mph			
		4 Lanes	3 Lanes	2 Lanes	Overall
Basic Freeway Section		0.81	0.83	0.82	0.81
		5 Lanes	4 Lanes	3 Lanes	Overall
Weaving Section		0.91	0.92	0.97	0.93
Merging / Diverging Area	Merging	0.88	0.91	0.90	0.80
	Diverging	0.68	0.70	0.76	0.71
	Combined	0.77	0.76	0.65	0.70
Section Type		Speed Limit - 60 mph			
		4 Lanes	3 Lanes	2 Lanes	Overall
Basic Freeway Section		0.77	0.83	0.78	0.78
		5 Lanes	4 Lanes	3 Lanes	Overall
Weaving Section		0.95	0.96	0.98	0.96
Merging / Diverging Area	Merging	0.90	0.92	0.92	0.81
	Diverging	0.67	0.73	0.79	0.72
	Combined	0.77	0.74	0.65	0.70
Section Type		Speed Limit - 55 mph			
		4 Lanes	3 Lanes	2 Lanes	Overall
Basic Freeway Section		0.74	0.83	0.79	0.78
		5 Lanes	4 Lanes	3 Lanes	Overall
Weaving Section		0.96	0.96	0.98	0.96
Merging / Diverging Area	Merging	0.92	0.94	0.94	0.82
	Diverging	0.71	0.78	0.79	0.74
	Combined	0.80	0.77	0.65	0.70

P-Value < 0.001

5.4. Density – Travel Time per Mile Relationship Development

The hourly average travel times were collected from the VISSIM software and paired with their respective hourly densities from VISSIM. The travel times were converted to per mile travel times to have a meaningful relationship as different section lengths were used in the experiment.

The relationship between density and travel time, or density and travel time reliability metrics could be linear or non-linear in nature. Therefore, several trends were examined. Based on the coefficient of determination (R-squared or R^2) (in figures), it was observed that a non-linear exponential model better fits the travel time per mile data with their respective density values than any other linear or non-linear distribution considered in this dissertation. In case of reliability metrics, a polynomial relationship better suits the relationship between PTI and BTI with their respective density values. Therefore, results based on exponential and polynomial distributions are only presented in this dissertation.

5.4.1. Basic Freeway Section

Figures 5.3 to 5.6 illustrate the relationship between the density (pc/mi/ln) and travel time/mile (sec) for 70, 65, 60, and 55 mph speed limits on a basic freeway section, respectively. The relationships were developed based on the available lane exposure on the freeway. A non-linear (exponential) relationship was observed between the two variables. The R-squared value ranges from 0.75 to 0.96.

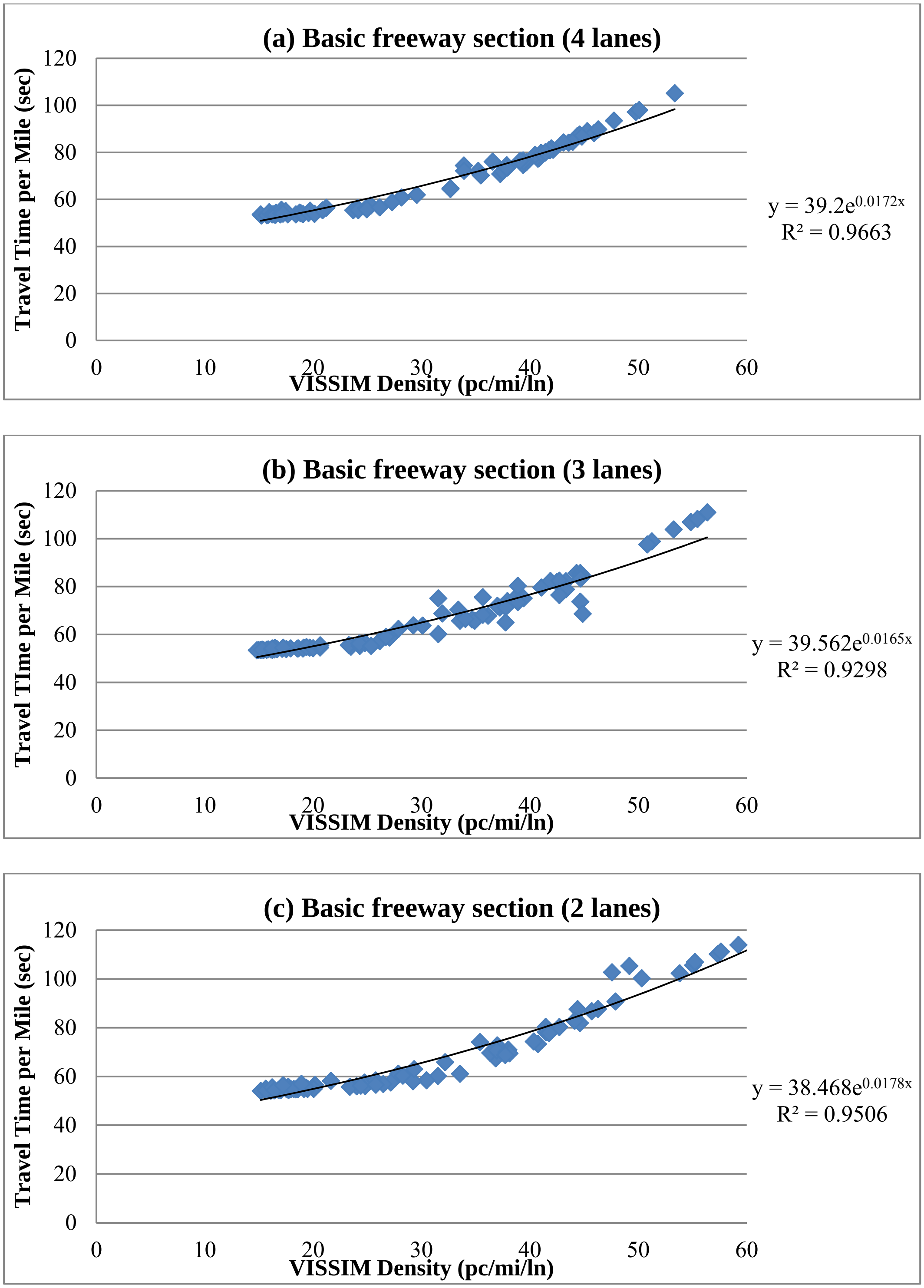


FIGURE 5.3: Density – travel time per mile relationship for 70 mph speed limit on basic freeway section

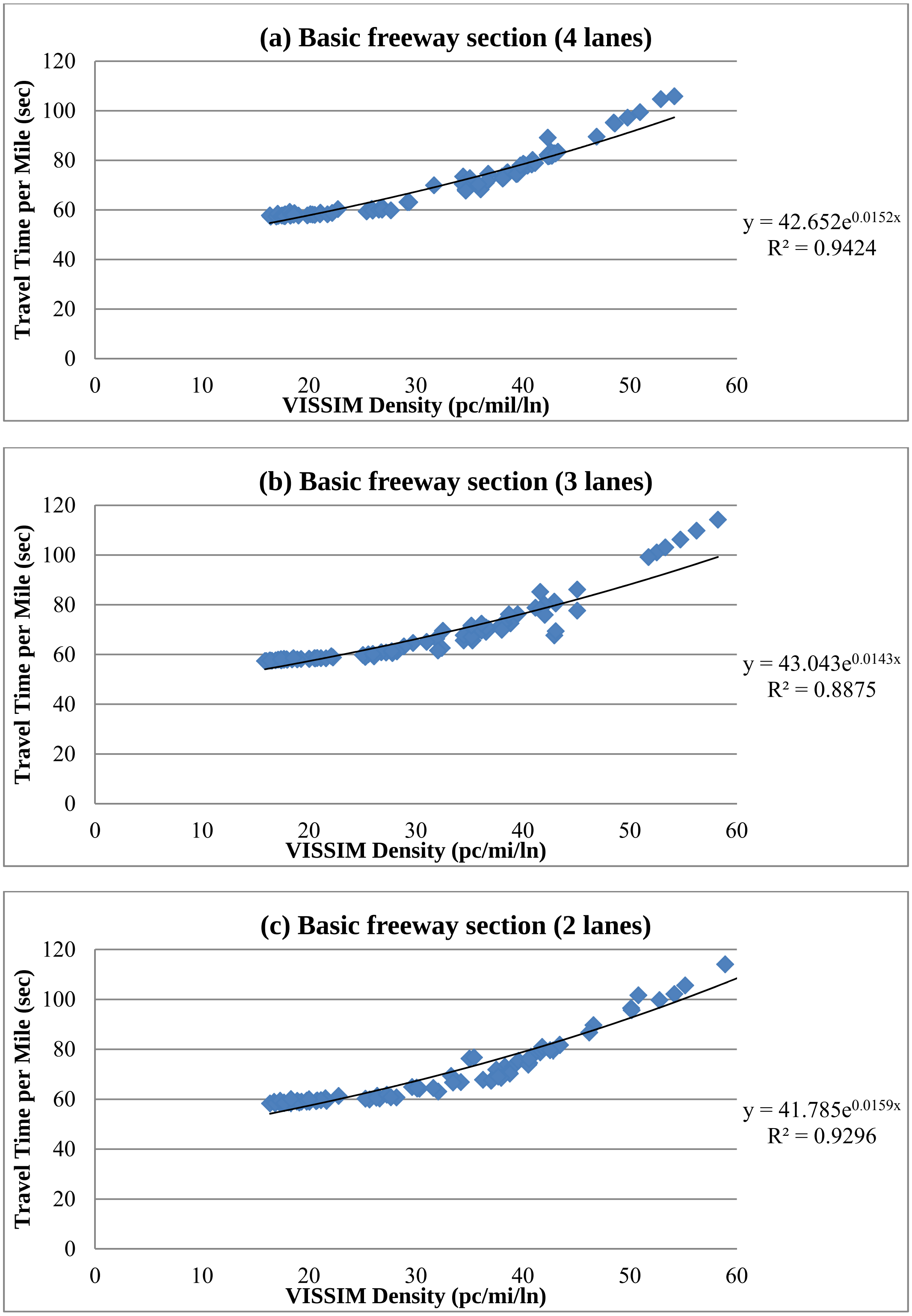


FIGURE 5.4: Density – travel time per mile relationship for 65 mph speed limit on basic freeway section

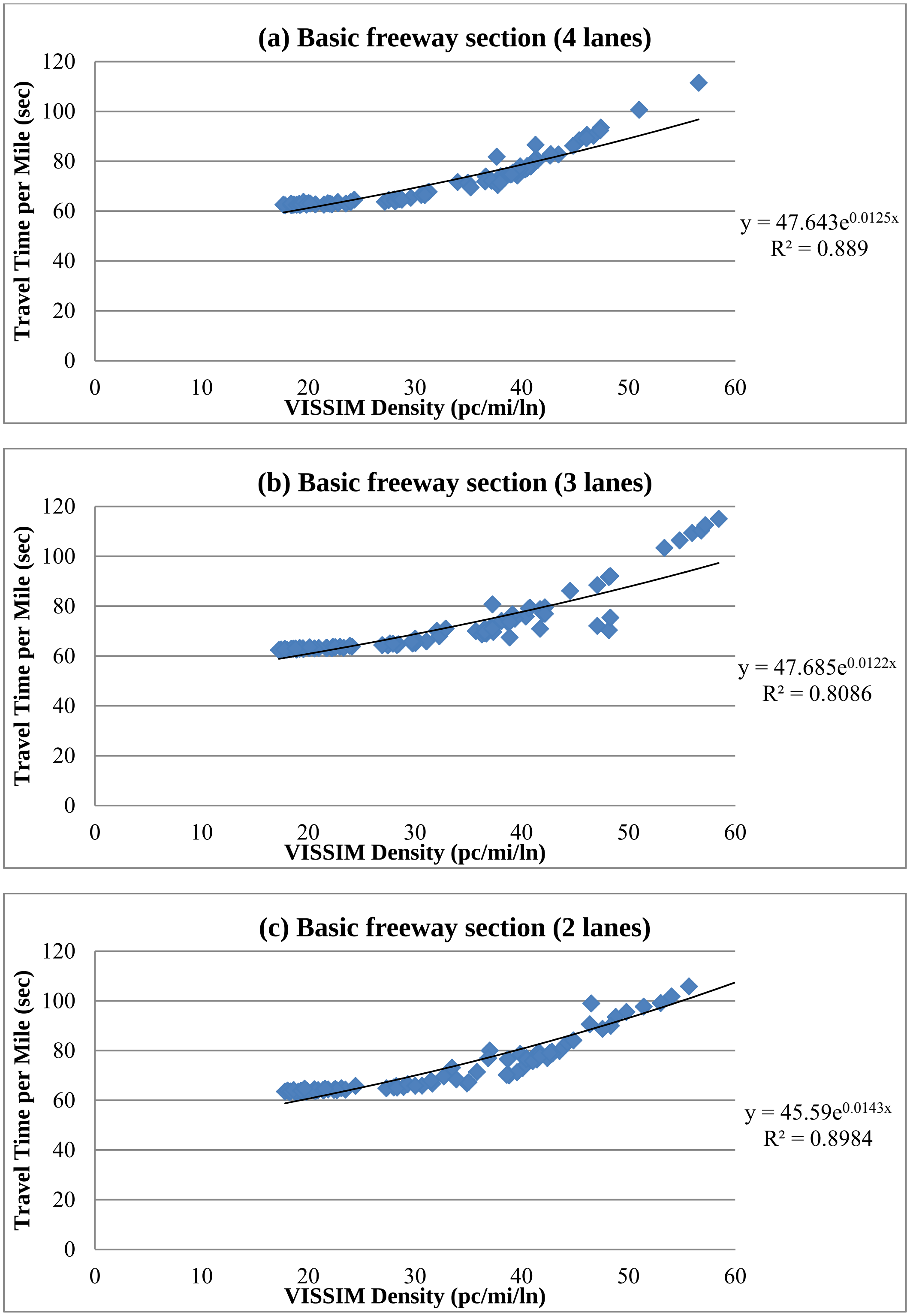


FIGURE 5.5: Density – travel time per mile relationship for 60 mph speed limit on basic freeway section

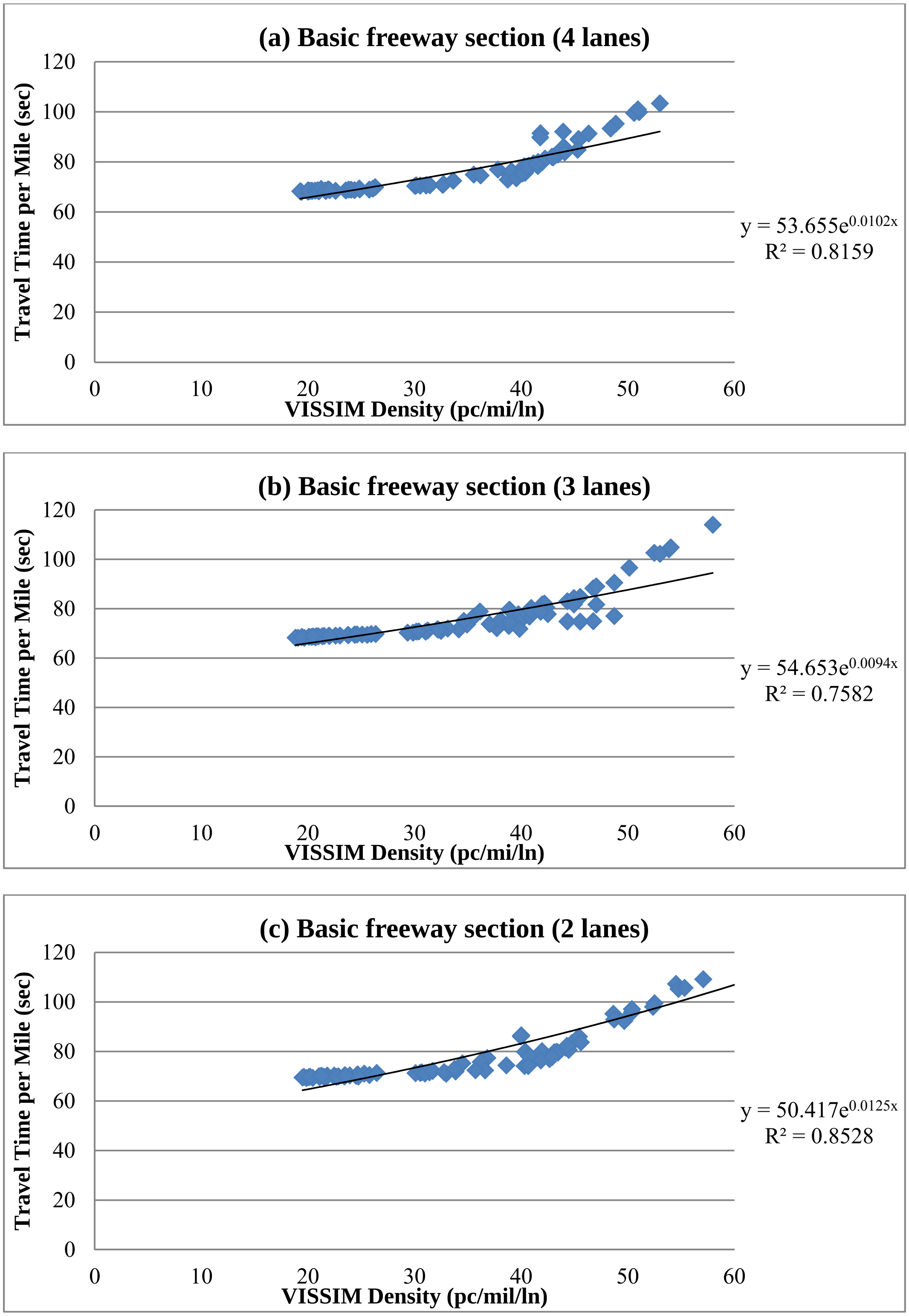


FIGURE 5.6: Density – travel time per mile relationship for 55 mph speed limit on basic freeway section

5.4.2. Weaving Section

Figures 5.7 to 5.10 provide the scatter plots between the density (pc/mi/ln) and travel time/mile (sec) for 70, 65, 60, and 55 mph speed limits on a weaving section, respectively. The relationships were developed based on the available lane exposure on the weaving section. An exponential relationship was observed between the two variables. The R-squared value ranges from 0.76 to 0.88. It can be noted that the lower the speed limit, the lesser the variability in the travel times.

5.4.3. Merging/Diverging Area

The relationship between the density (pc/mi/ln) and travel time/mile (sec) for 70, 65, 60, and 55 mph speed limits on a merging/diverging area, are shown in Figures 5.11, to 5.14, respectively, based on the available lane exposure. An exponential relationship was observed between the two variables. The R-squared value ranges from 0.92 to 0.98. However, as the maneuverability in merging and diverging areas is not similar in nature, both of the areas are further analyzed separately. Figures 5.15 to 5.18 show the relationship between density and travel time per mile in merging areas and Figures 5.19 to 5.22 for diverging areas.

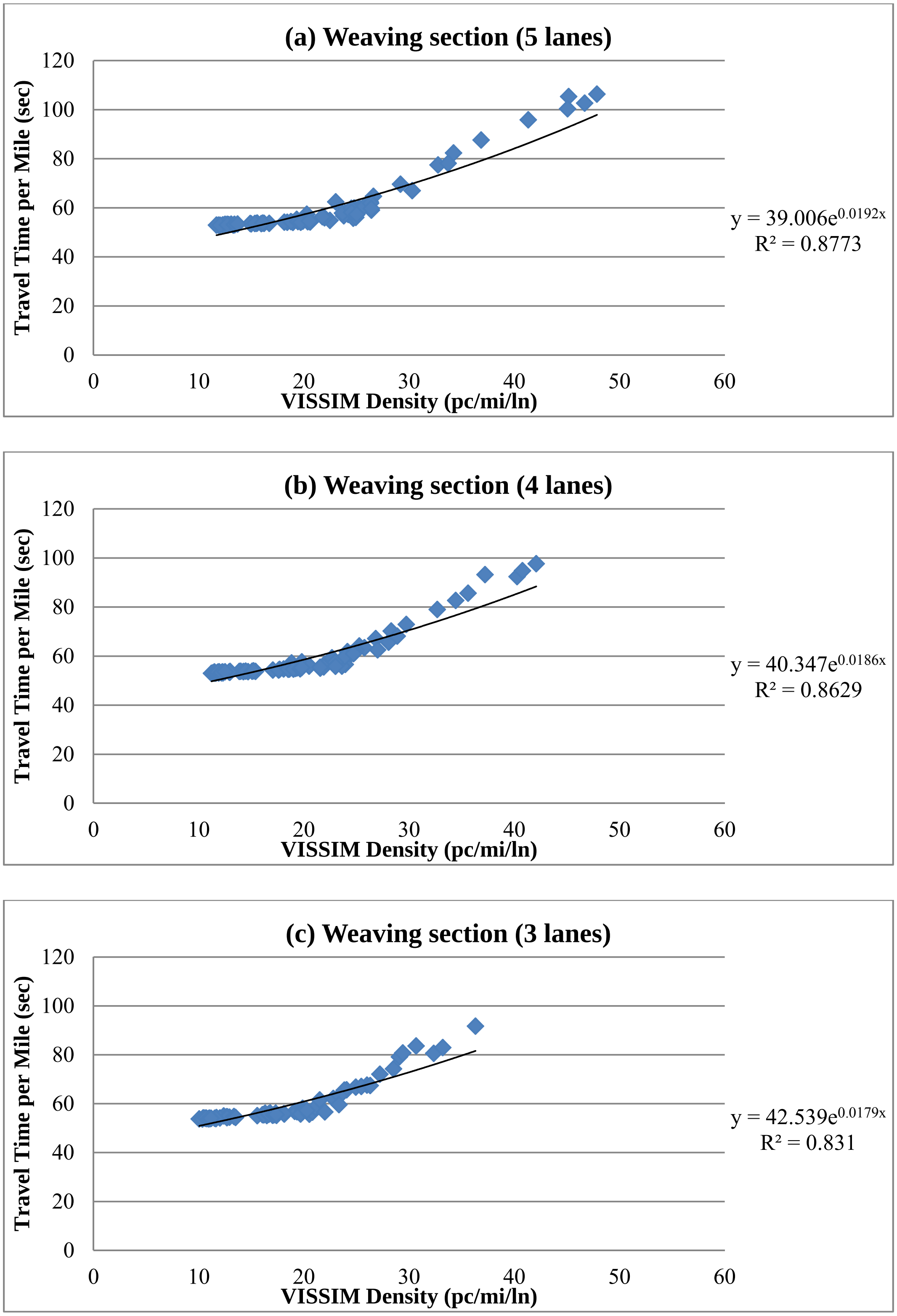


FIGURE 5.7: Density – travel time per mile relationship for 70 mph speed limit on weaving section

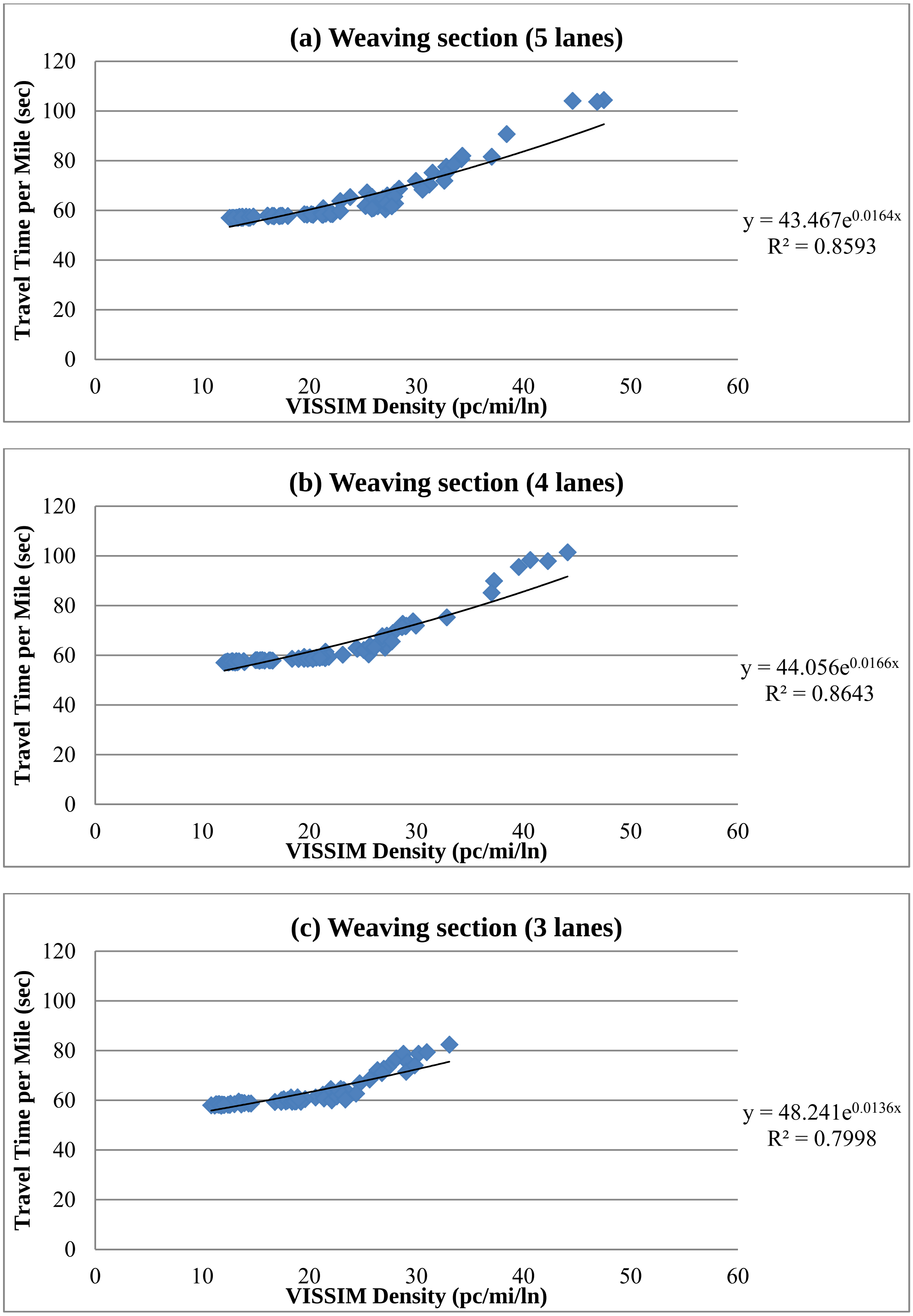


FIGURE 5.8: Density – travel time per mile relationship for 65 mph speed limit on weaving section

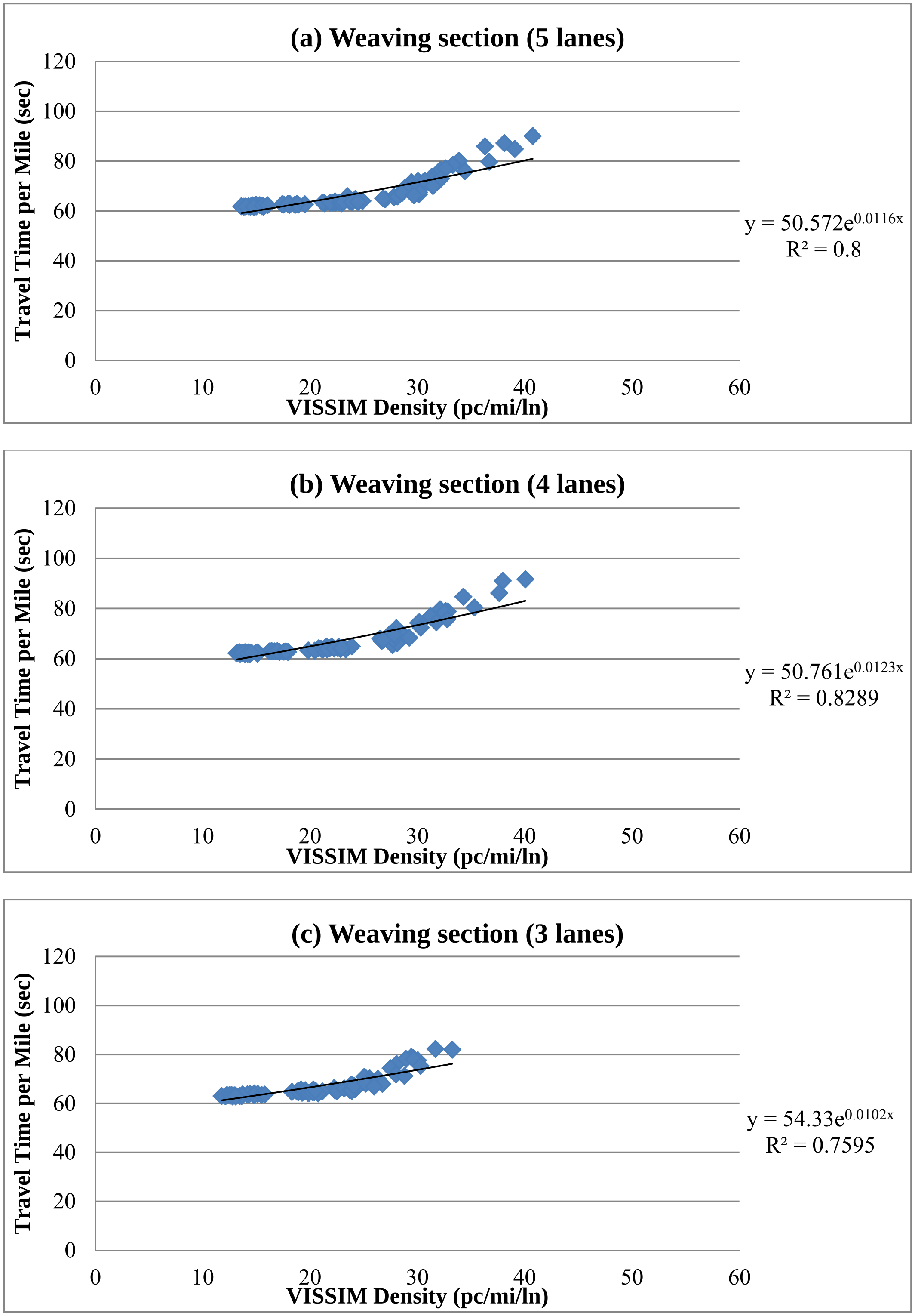


FIGURE 5.9: Density – travel time per mile relationship for 60 mph speed limit on weaving section

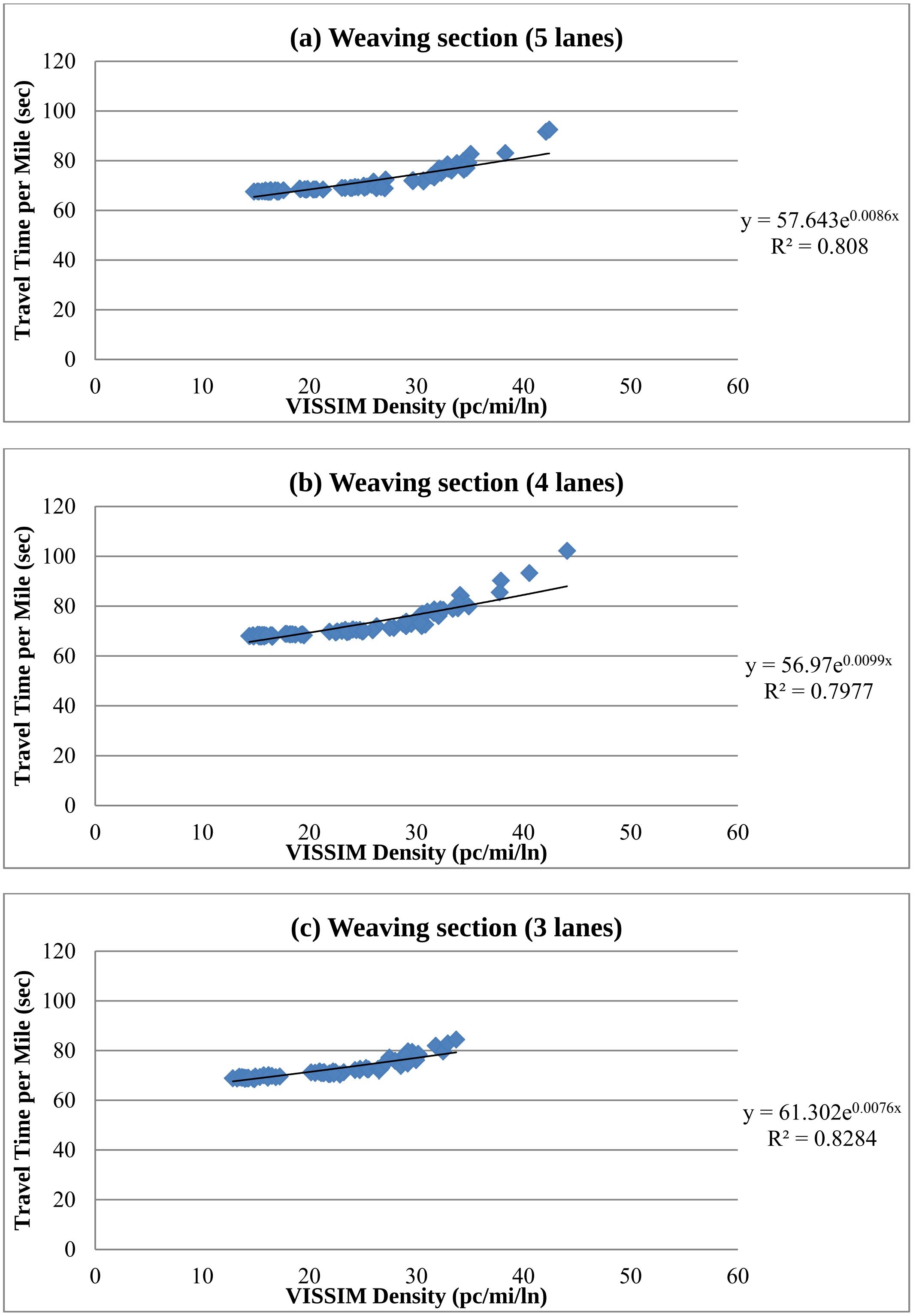


FIGURE 5.10: Density – travel time per mile relationship for 55 mph speed limit on weaving section

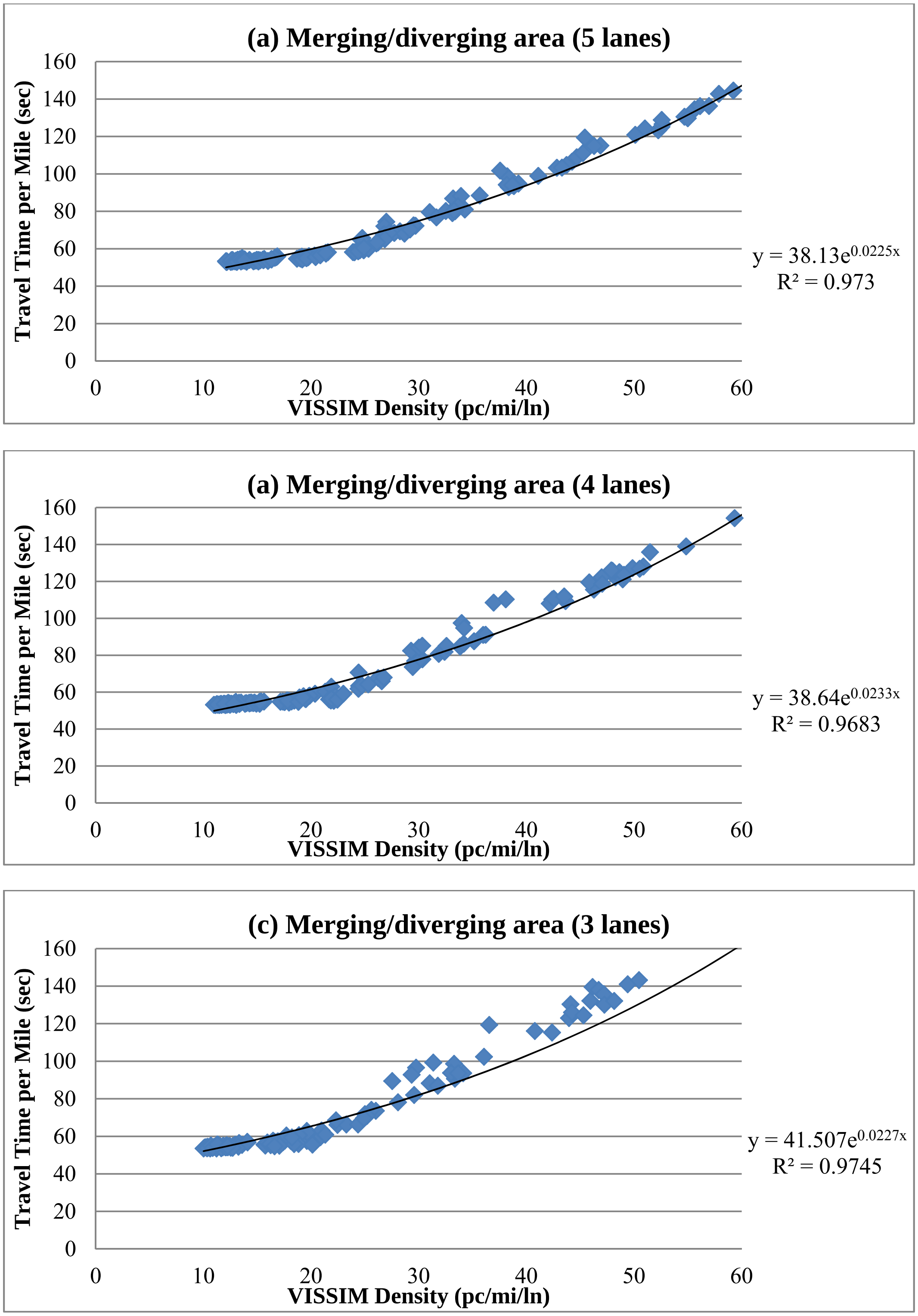


FIGURE 5.11: Density – travel time per mile relationship for 70 mph speed limit on merging/diverging area

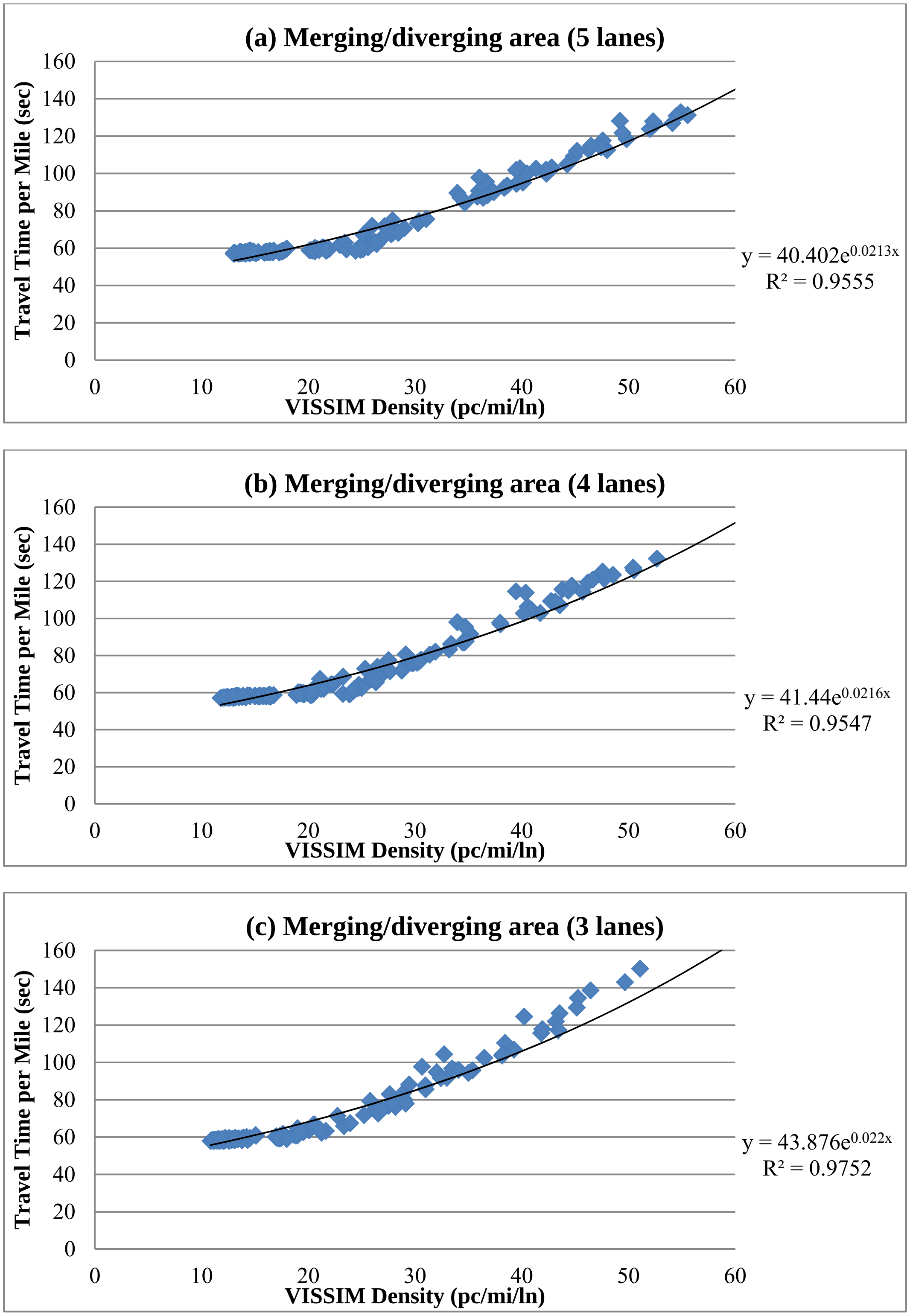


FIGURE 5.12: Density – travel time per mile relationship for 65 mph speed limit on merging/diverging area

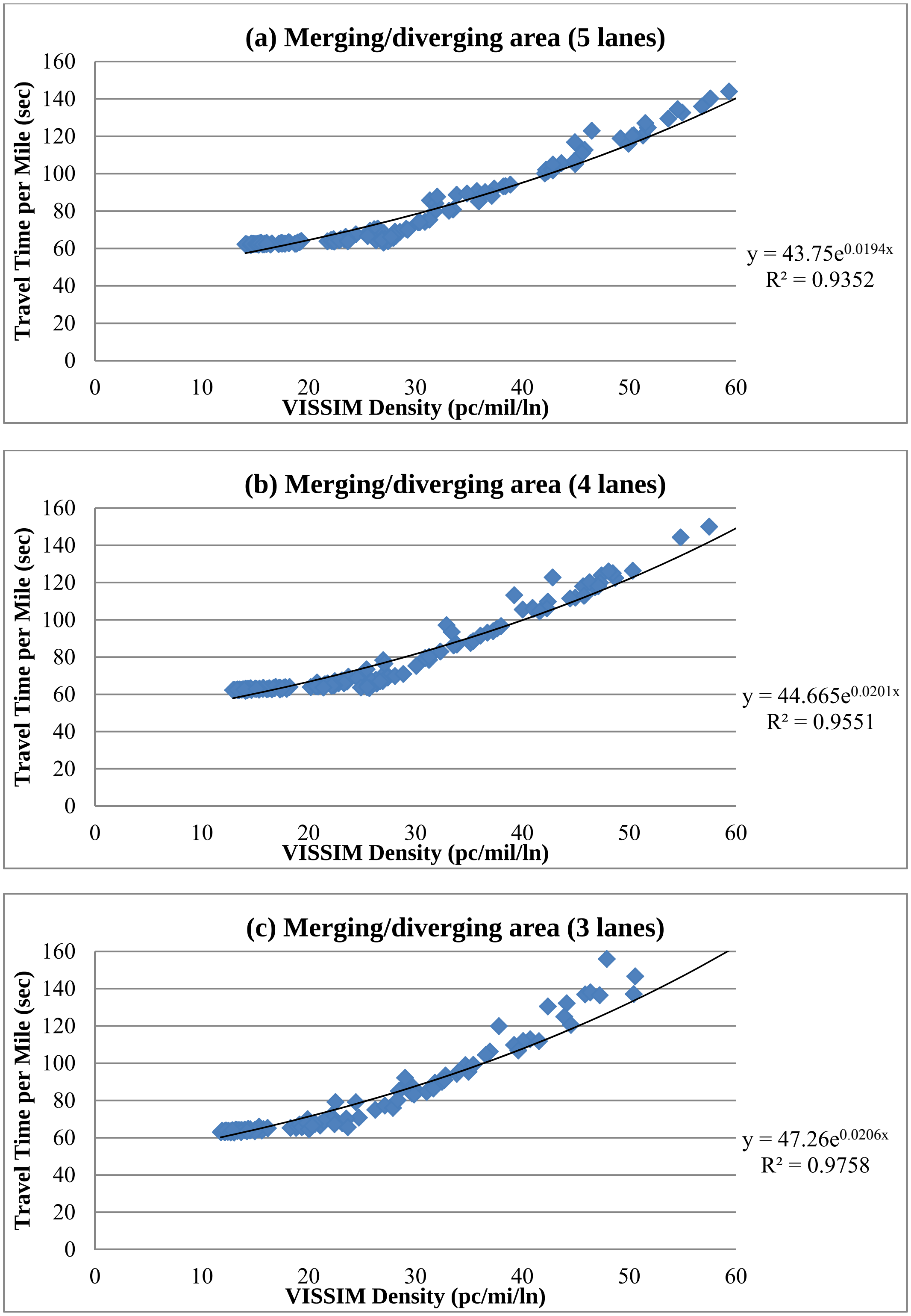


FIGURE 5.13: Density – travel time per mile relationship for 60 mph speed limit on merging/diverging area

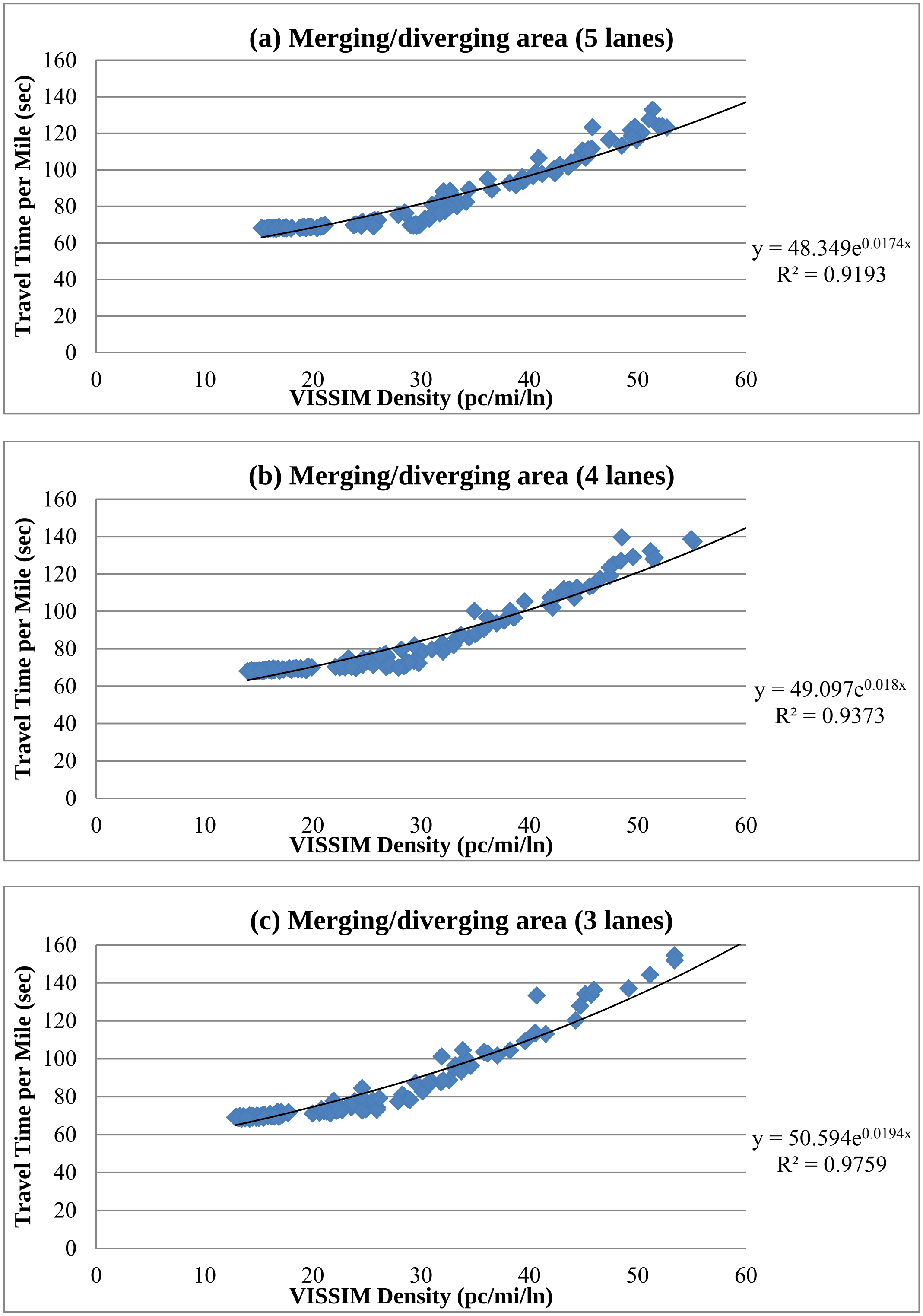


FIGURE 5.14: Density – travel time per mile relationship for 55 mph speed limit on merging/diverging area

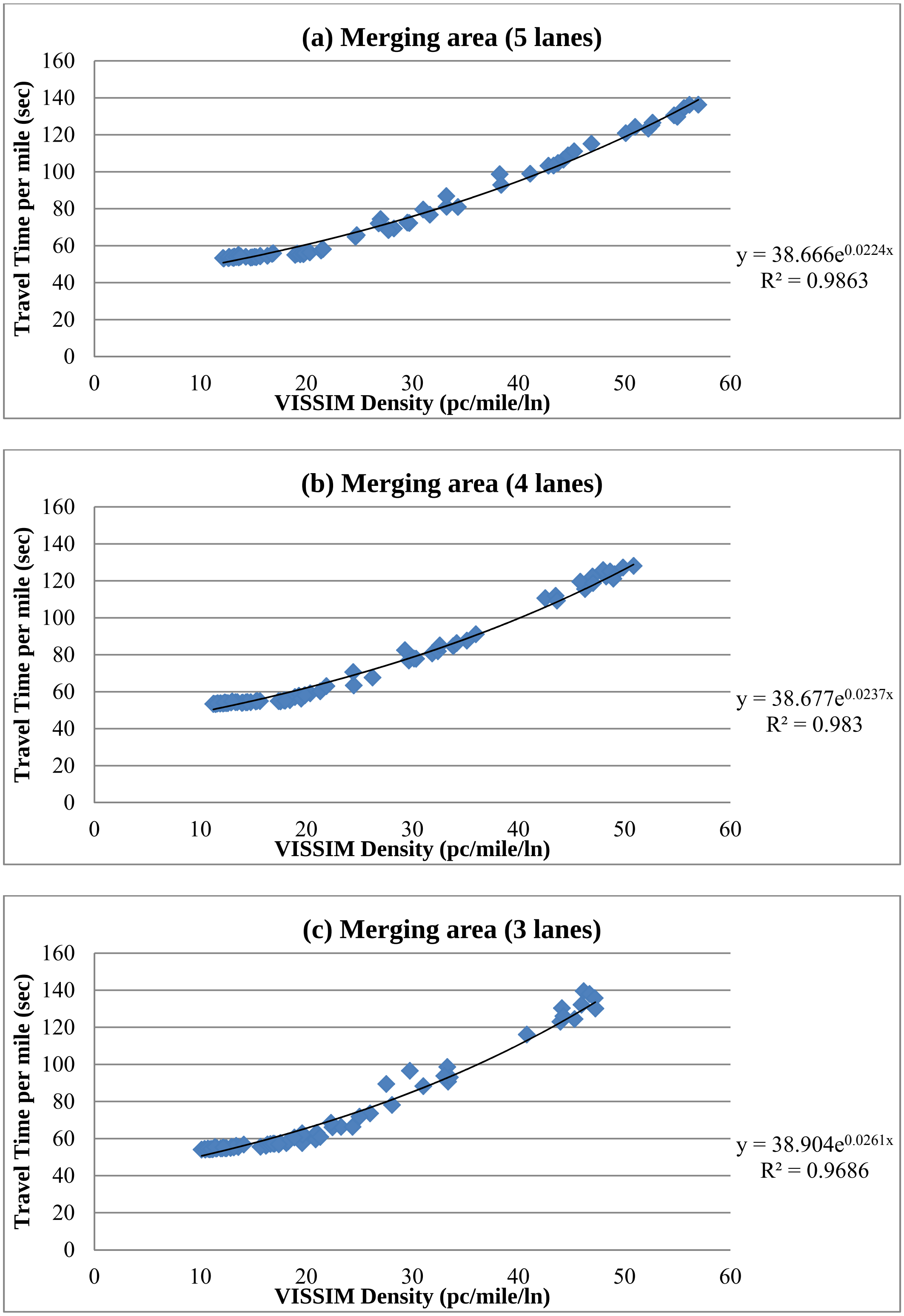


FIGURE 5.15: Density – travel time per mile relationship for 70 mph speed limit on merging area

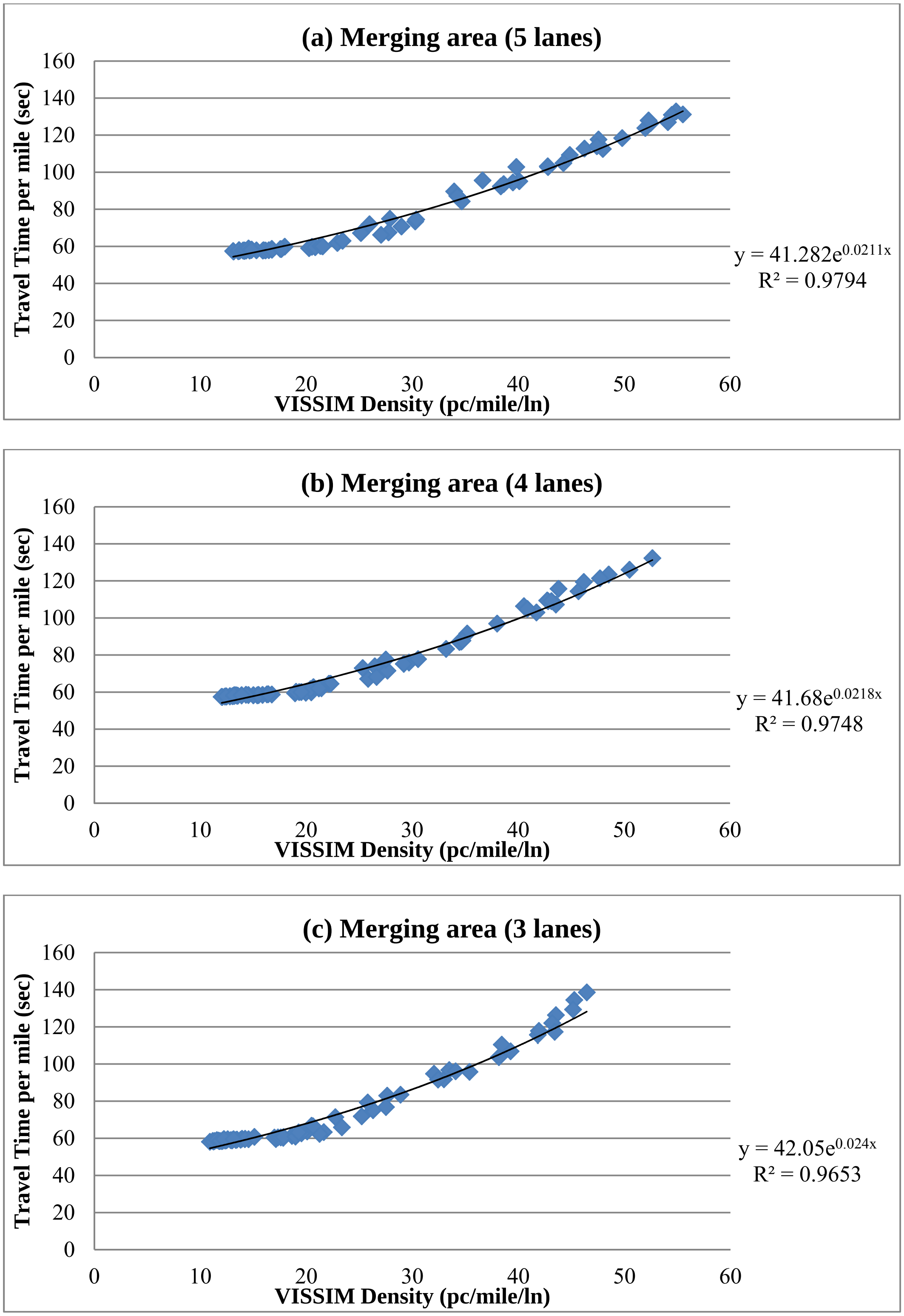


FIGURE 5.16: Density – travel time per mile relationship for 65 mph speed limit on merging area

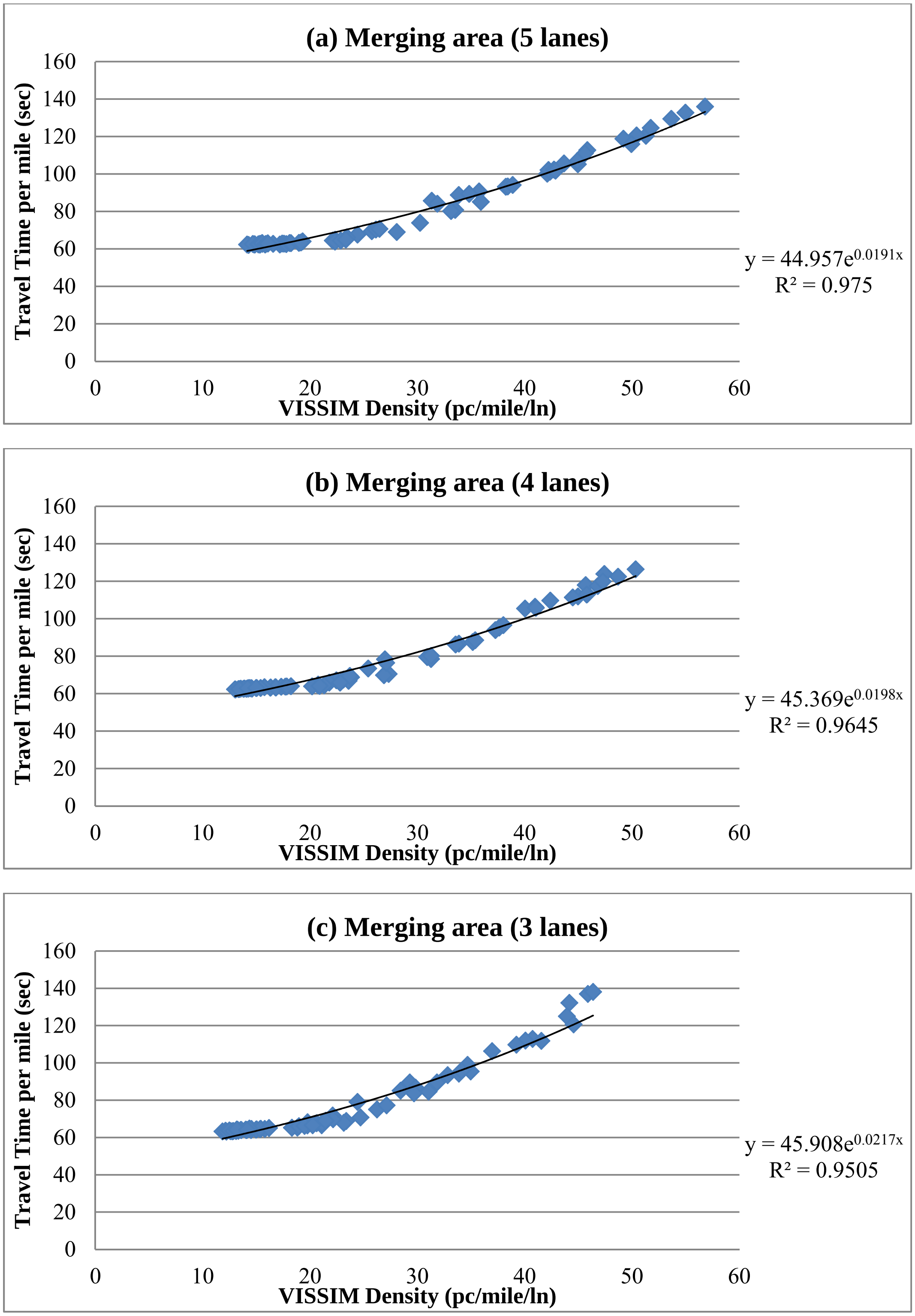


FIGURE 5.17: Density – travel time per mile relationship for 60 mph speed limit on merging area

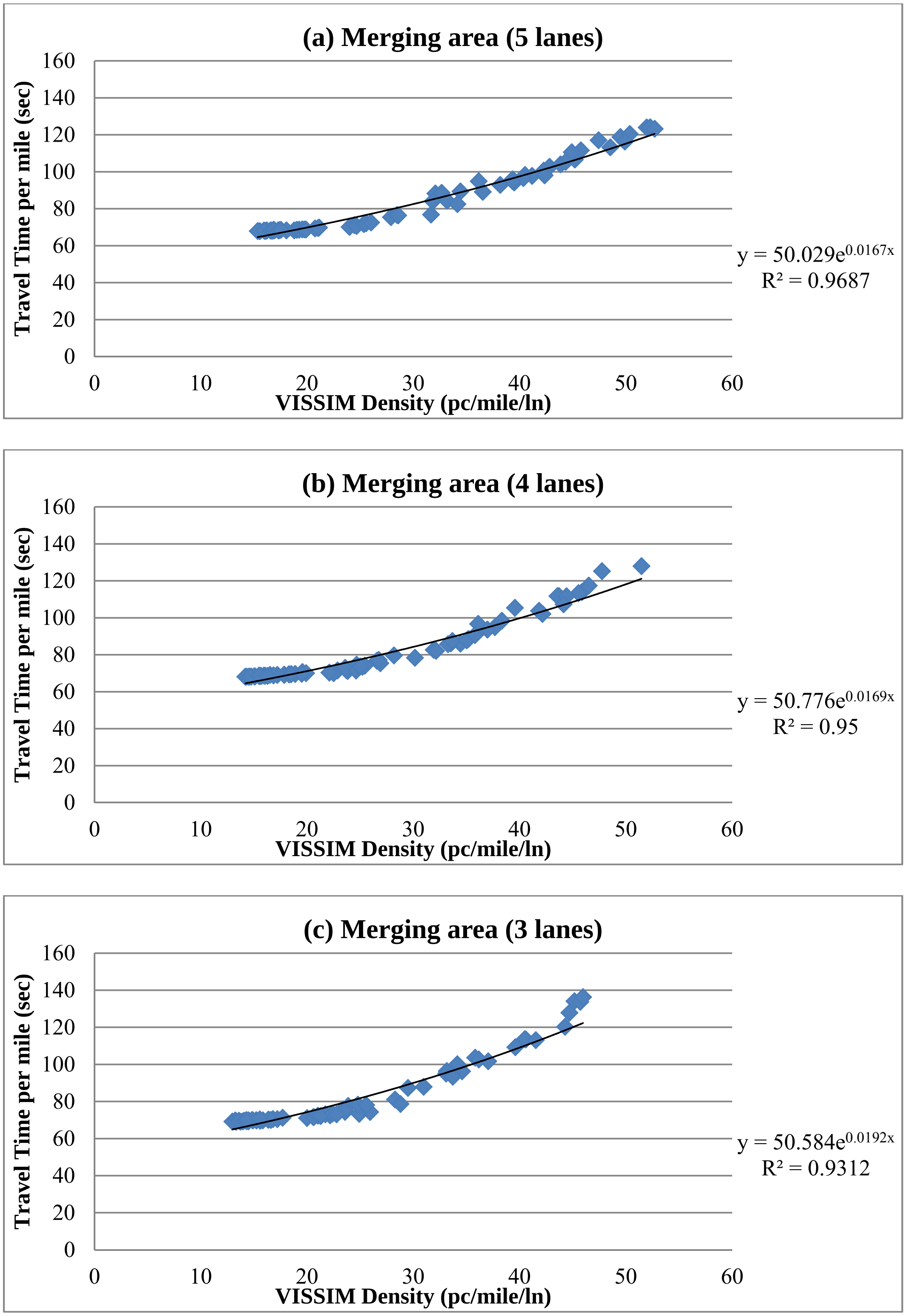


FIGURE 5.18: Density – travel time per mile relationship for 55 mph speed limit on merging area

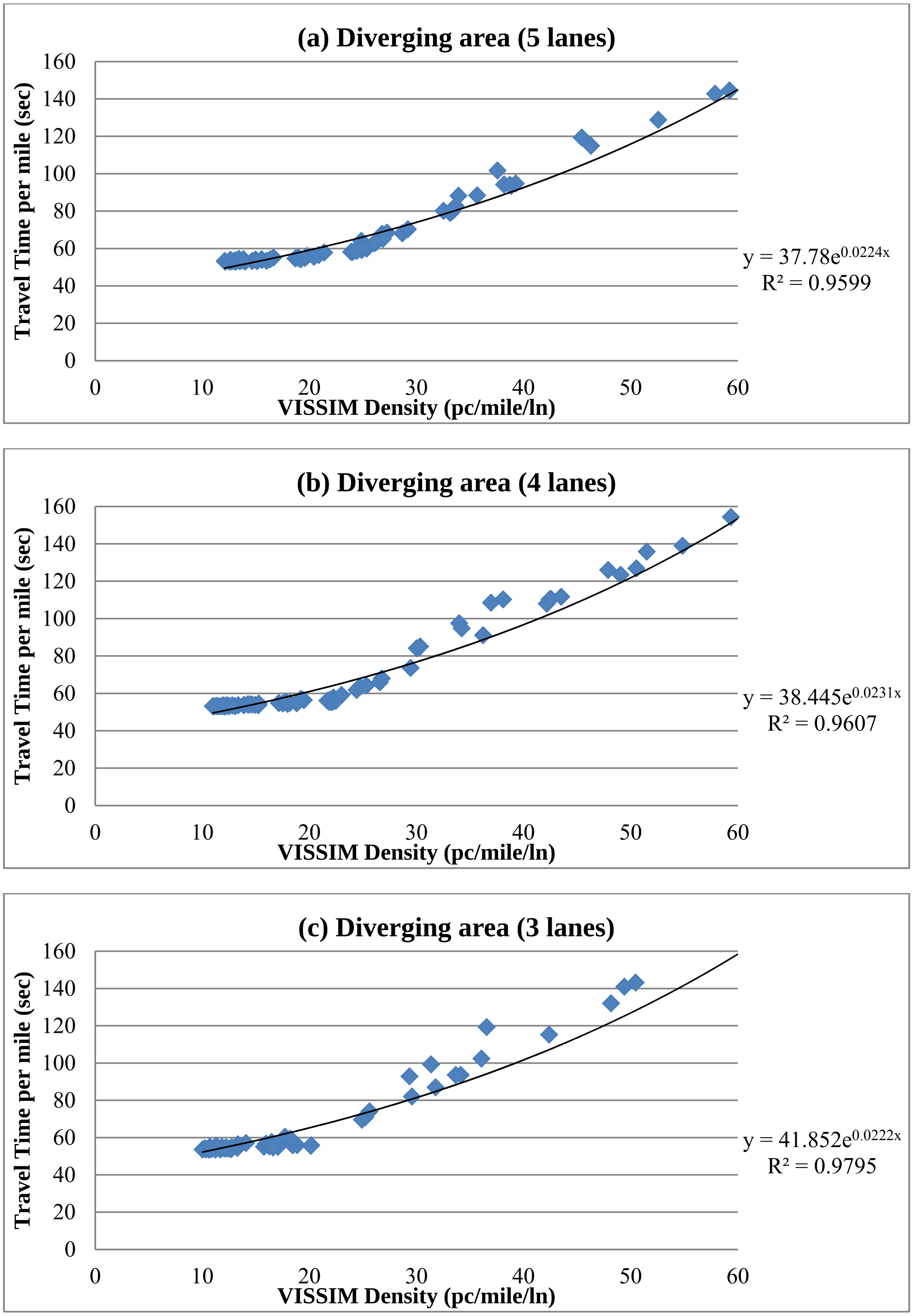


FIGURE 5.19: Density – travel time per mile relationship for 70 mph speed limit on diverging area

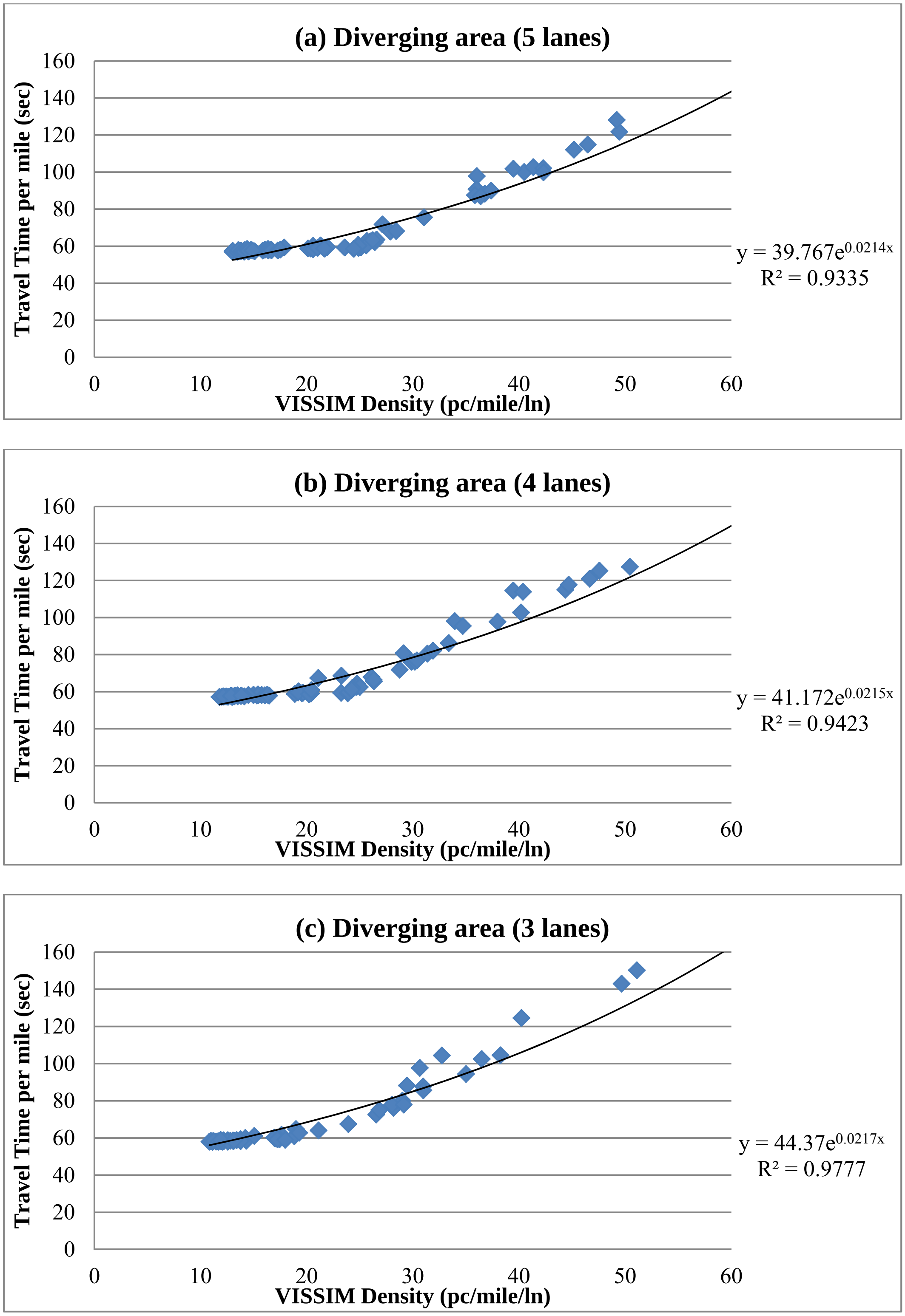


FIGURE 5.20: Density – travel time per mile relationship for 65 mph speed limit on diverging area

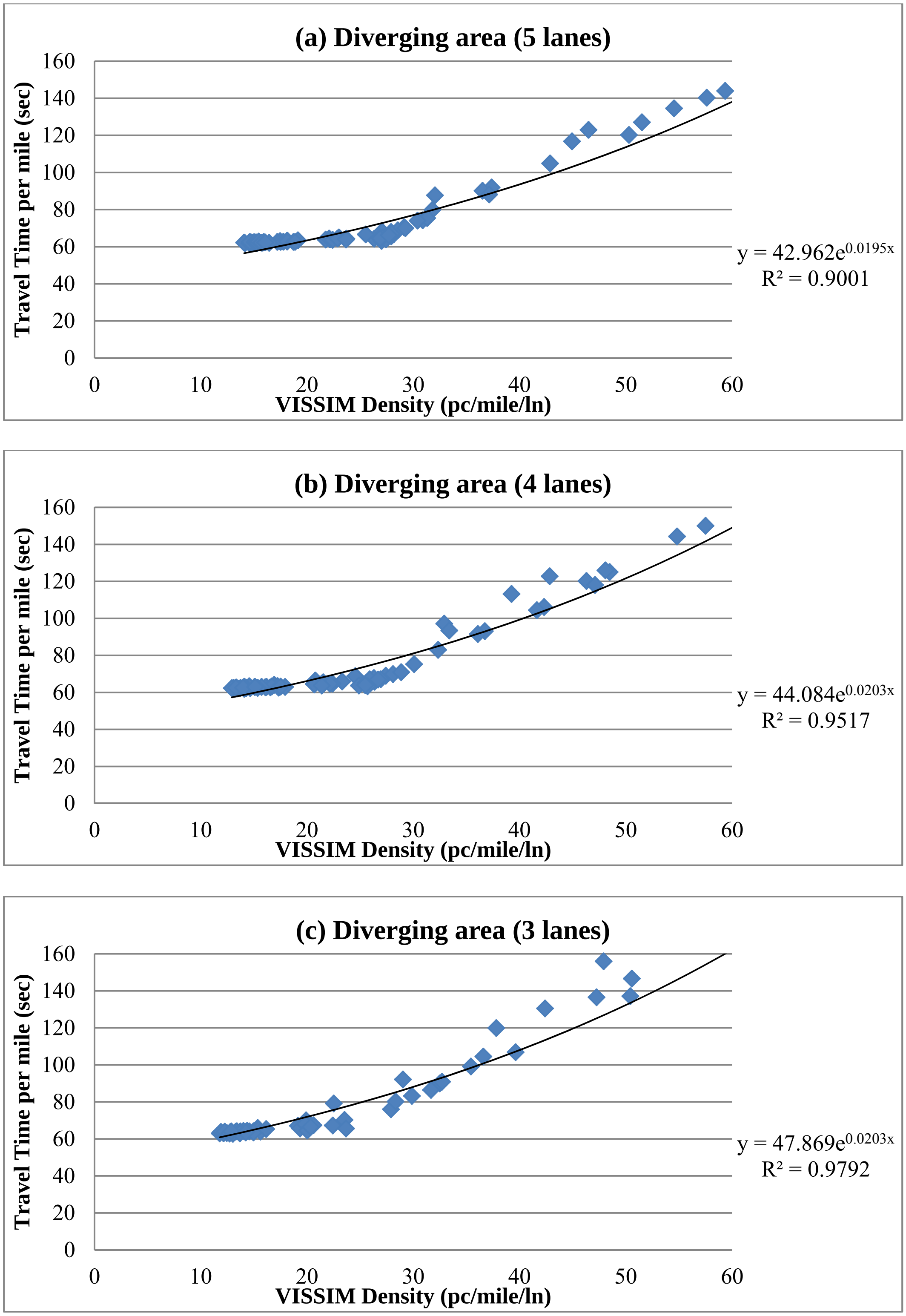
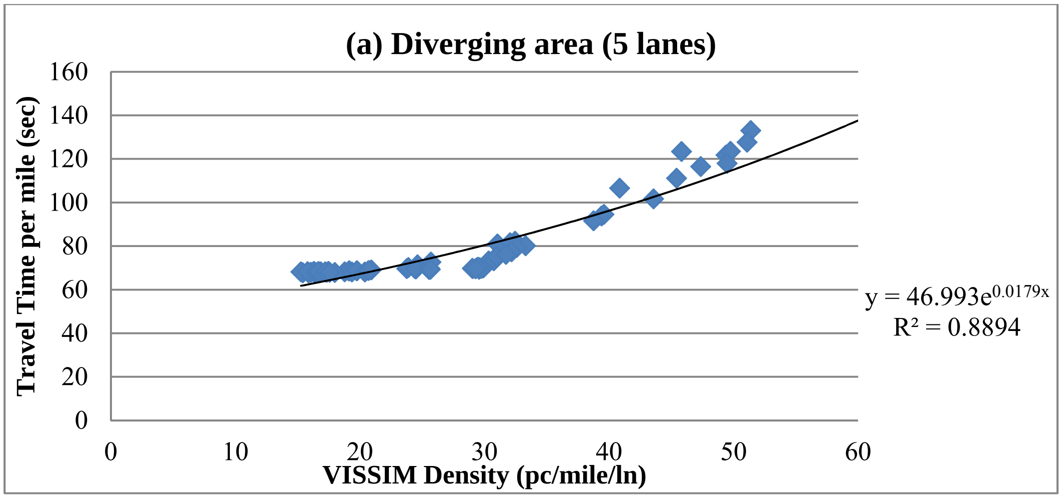
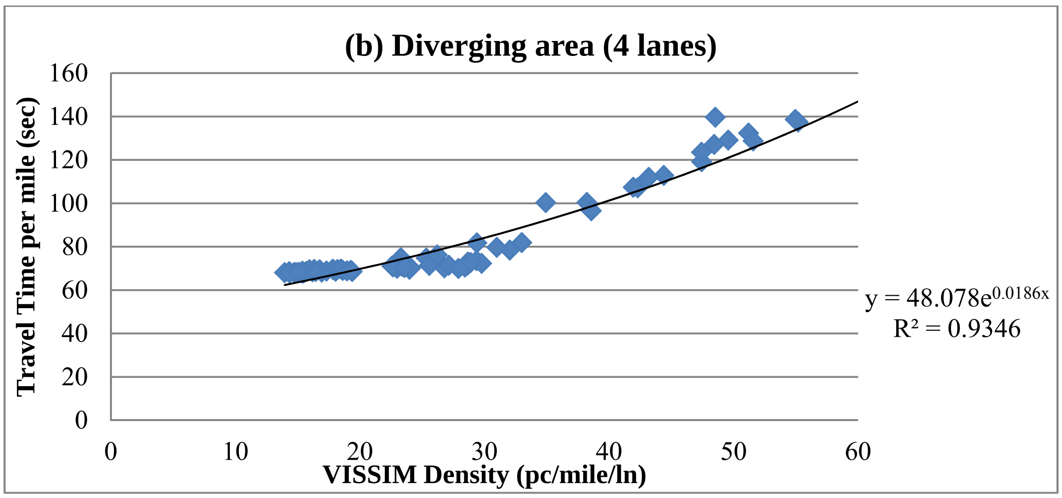


FIGURE 5.21: Density – travel time per mile relationship for 60 mph speed limit on diverging area





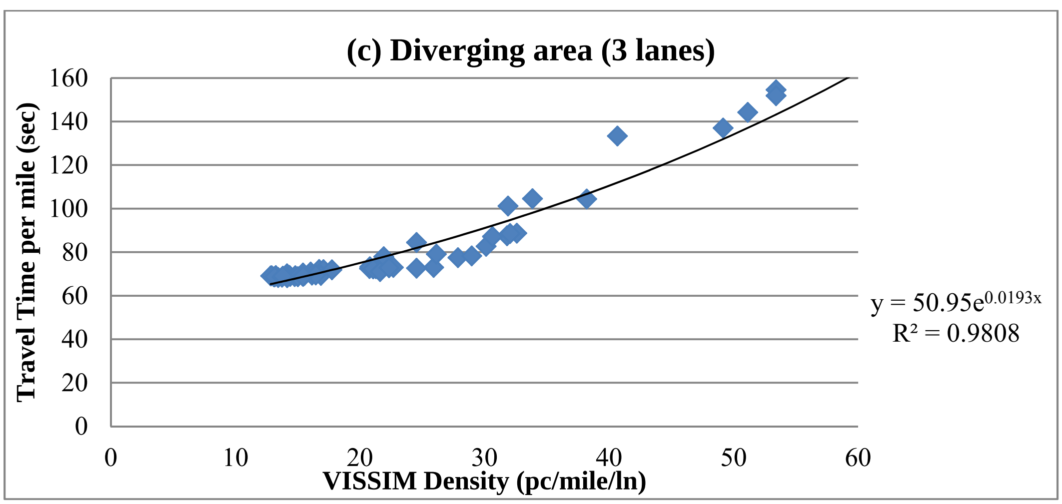


FIGURE 5.22: Density – travel time per mile relationship for 55 mph speed limit on diverging area

5.5. Density – Travel Time Reliability Indices Relationship Development

5.5.1. Planning Time Index

The 95th and 5th percentile travel time information were also computed for each hour using one-minute average travel time data from VISSIM. The information was used to compute the PTI values for each hour using Equation 3.1 and later compared with their respective hourly density values from VISSIM.

5.5.1.1. Basic Freeway Section

Figures 5.23 to 5.26 illustrate the relationship between the density and PTI for 70, 65, 60, and 55 mph speed limits on a basic freeway section, respectively. The relationships were developed based on the available lane exposure on the freeway. A non-linear (polynomial) relationship was observed between the two variables.

5.5.1.2. Weaving Section

Figures 5.27 to 5.30 provide the scatter plots between the density and PTI for 70, 65, 60, and 55 mph speed limits on a weaving section, respectively. The relationships were developed based on the available lane exposure on the weaving section. A polynomial relationship was observed between the two variables as well.

5.5.1.3. Merging/Diverging Area

The relationship between the density and PTI for 70, 65, 60, and 55 mph speed limits on a merging/diverging area are shown in Figures 5.31 to 5.34, respectively, based on the available lane exposure. A non-linear (polynomial) relationship was observed between the two variables. Both merging and diverging areas are also evaluated separately to identify if the maneuverability difference in those two areas have any influence on PTI and their respective densities. Figures 5.35 to 5.38 illustrate the condition for merging areas, while Figures 5.39 to 5.42 illustrate that of diverging areas.

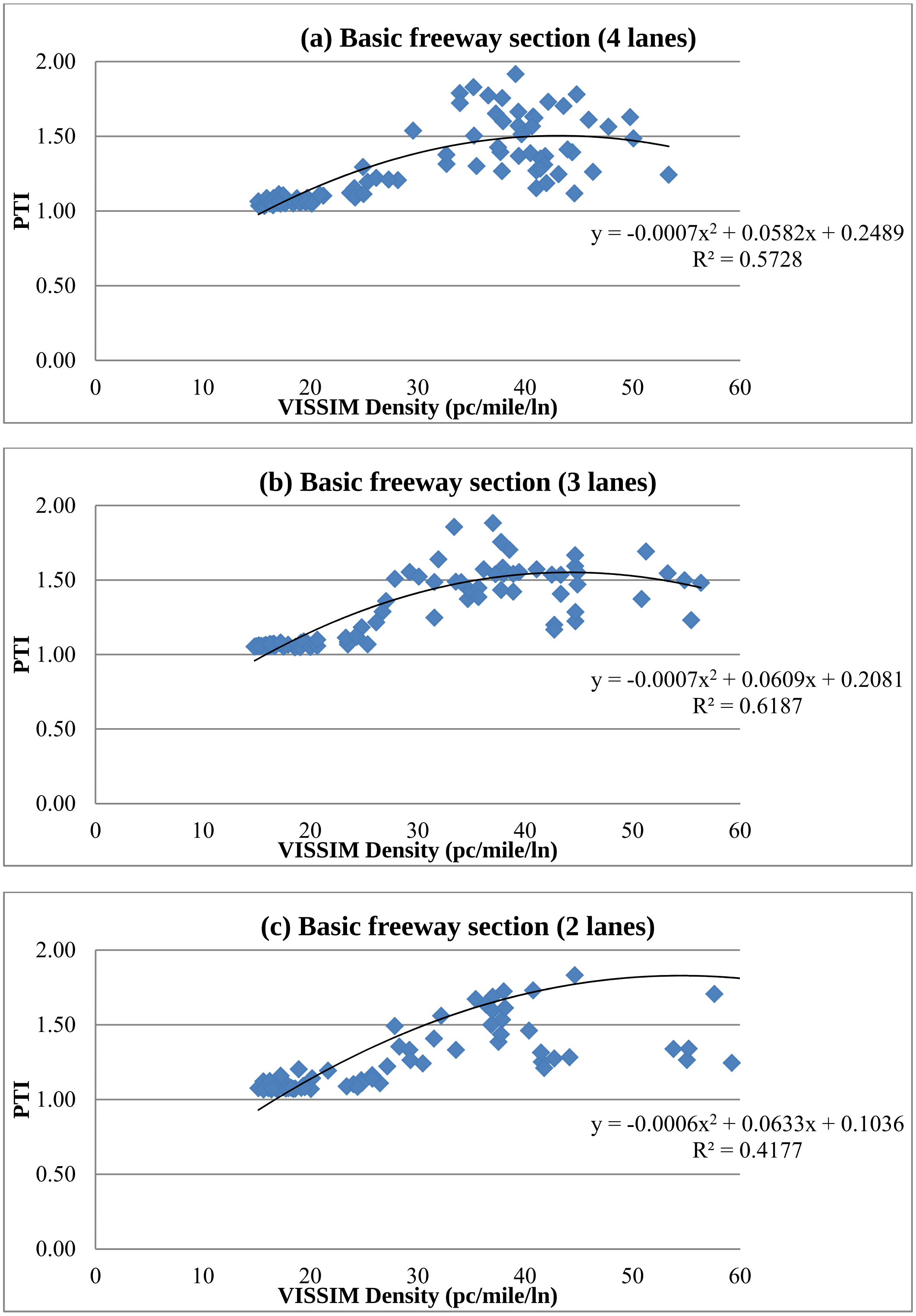


FIGURE 5.23: Density – PTI relationship for 70 mph speed limit on basic freeway section

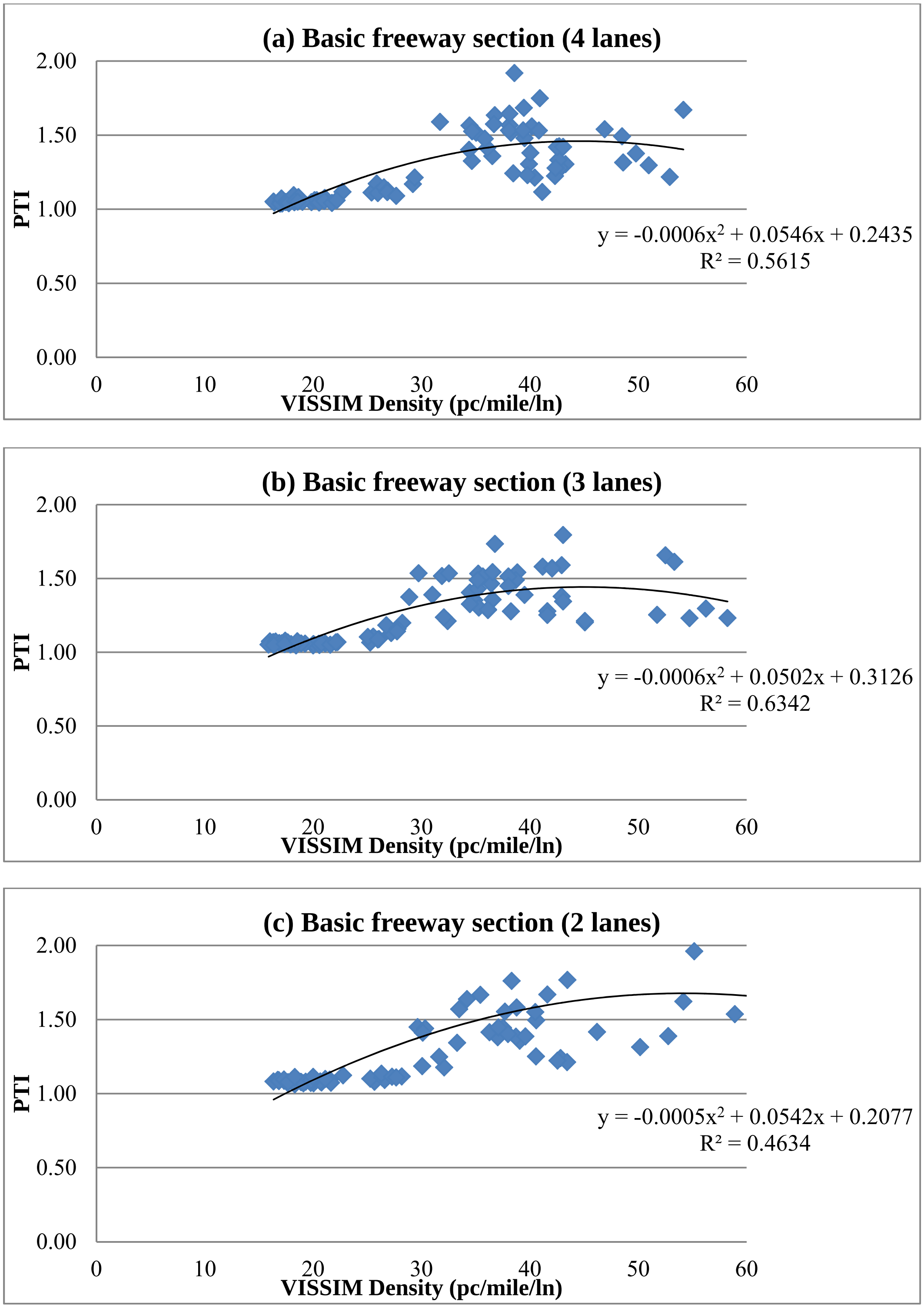


FIGURE 5.24: Density – PTI relationship for 65 mph speed limit on basic freeway section

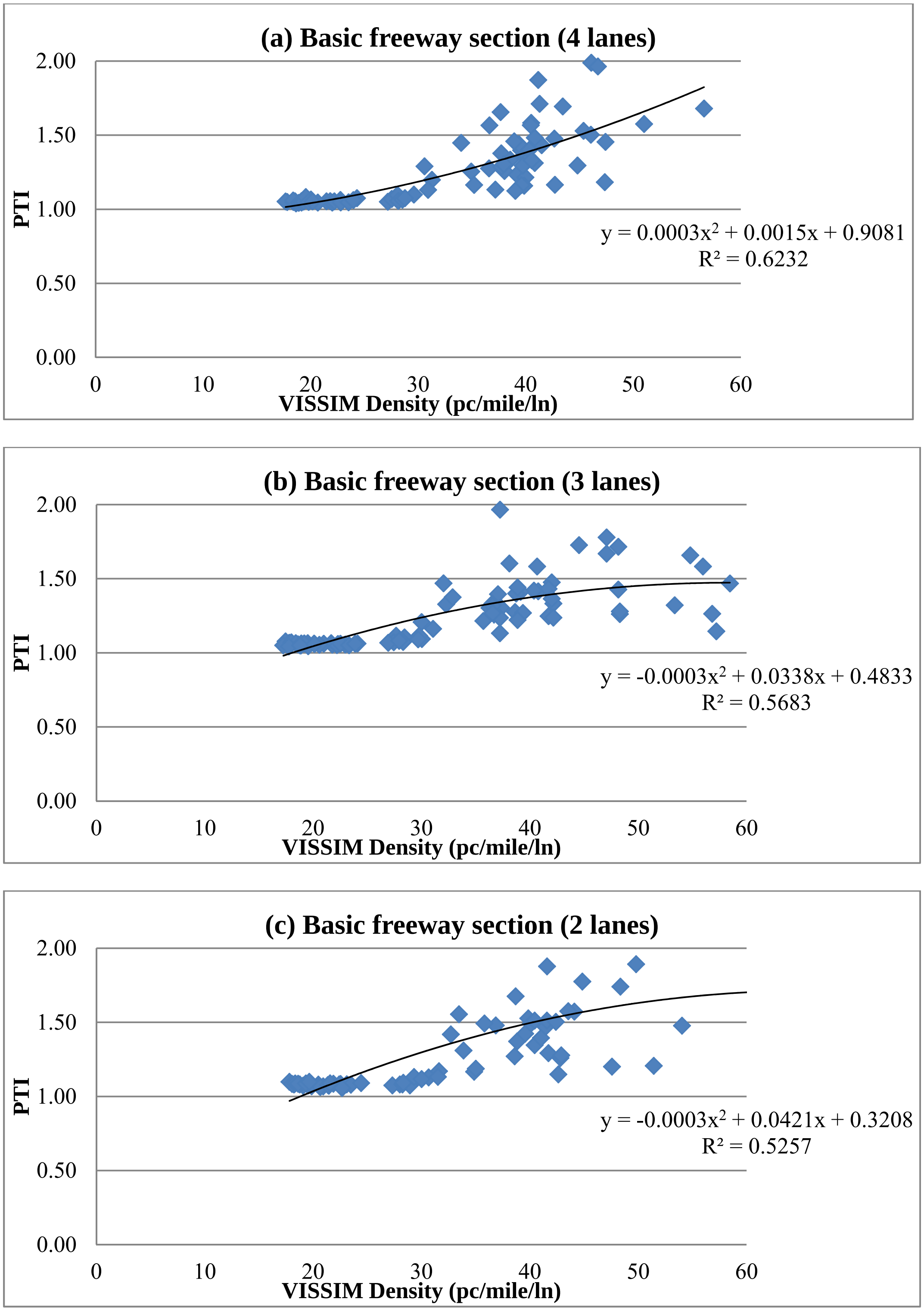


FIGURE 5.25: Density – PTI relationship for 60 mph speed limit on basic freeway section

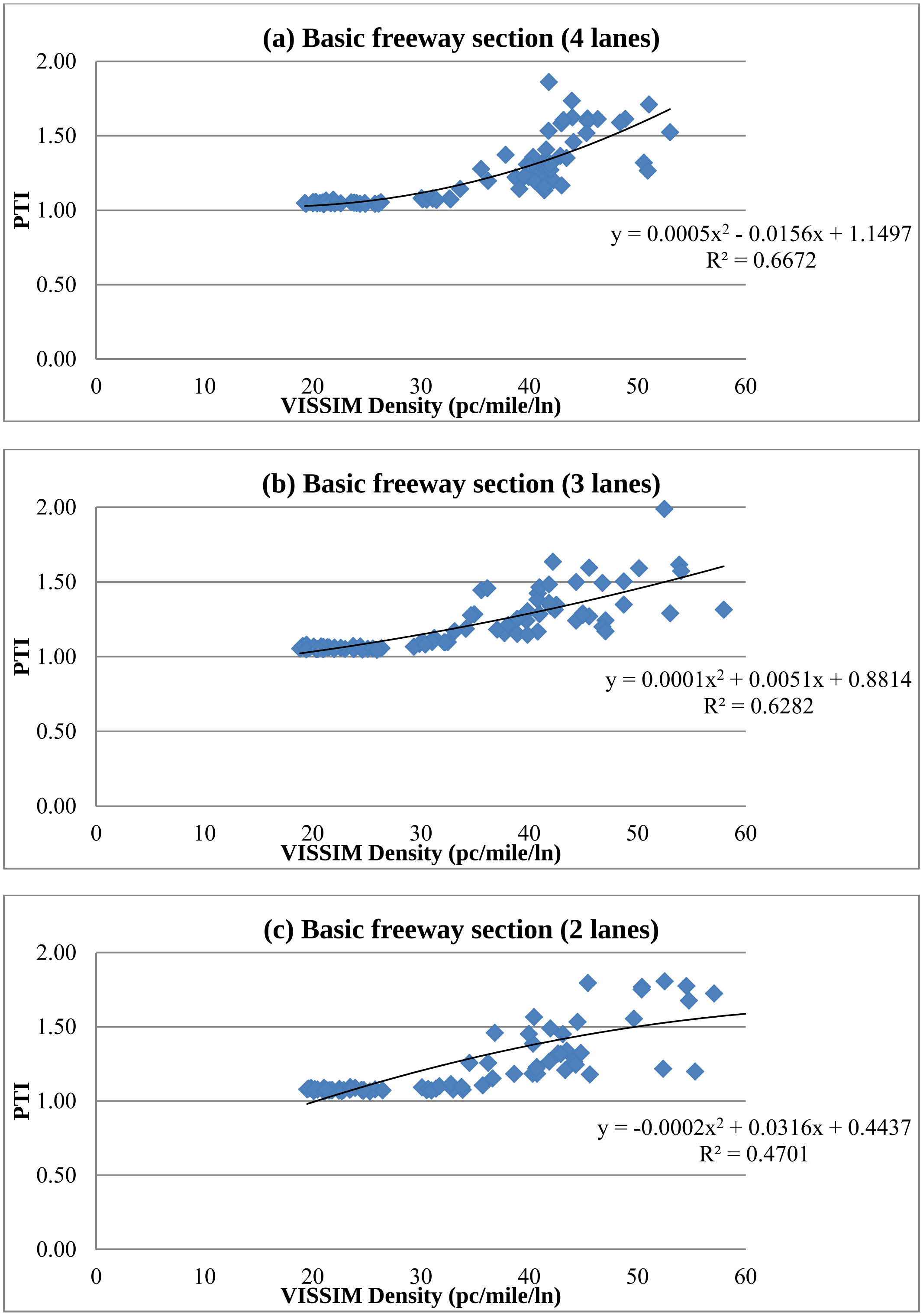


FIGURE 5.26: Density – PTI relationship for 55 mph speed limit on basic freeway section

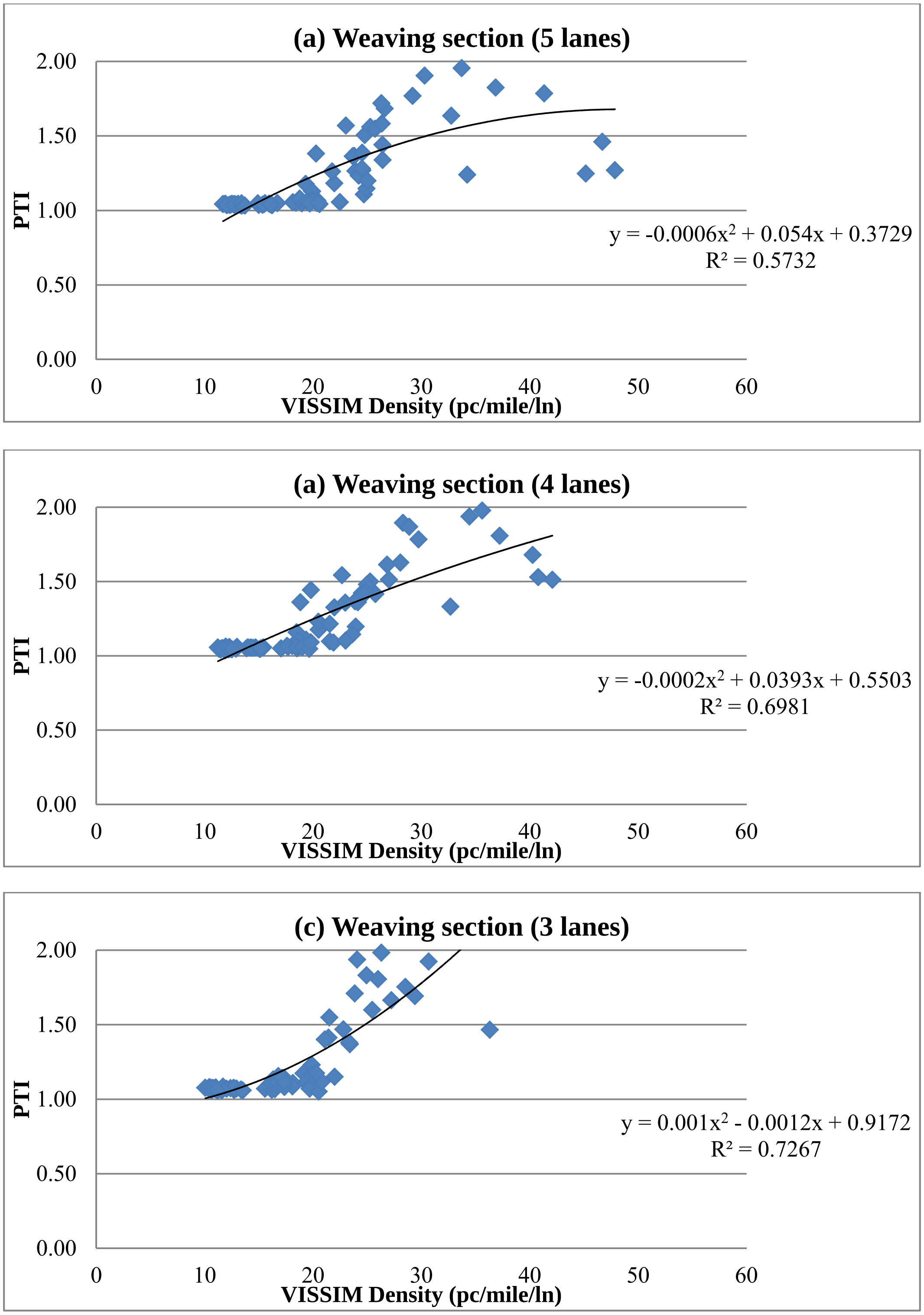


FIGURE 5.27: Density – PTI relationship for 70 mph speed limit on weaving section

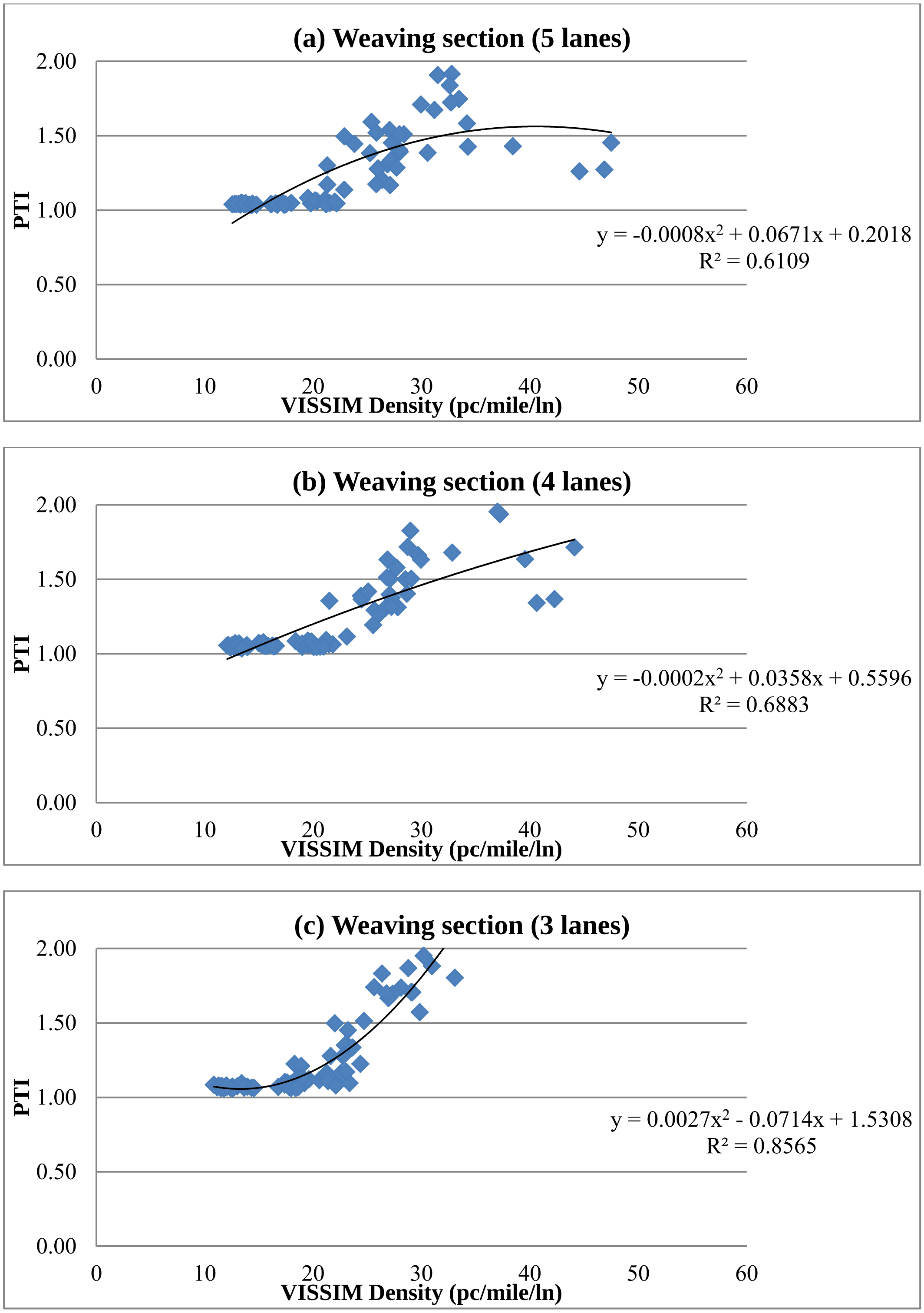
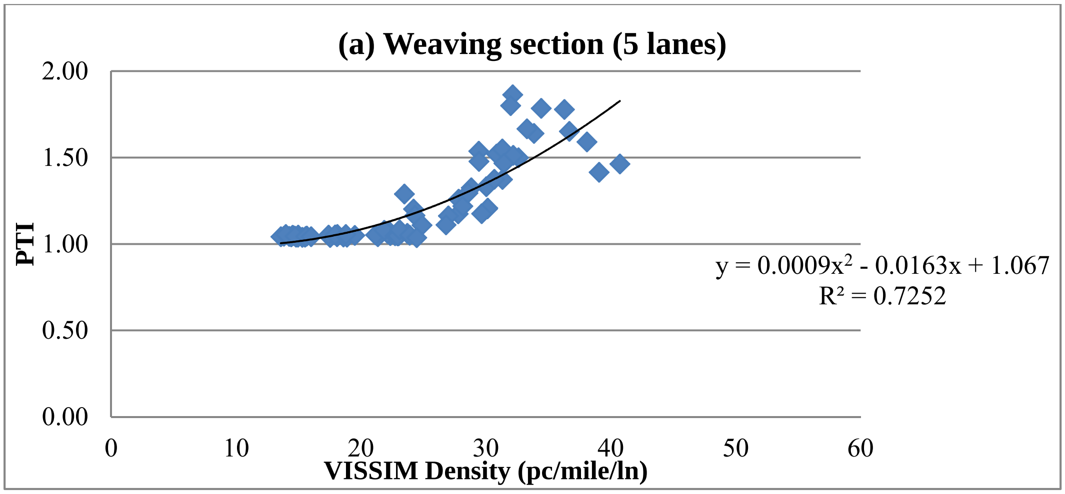
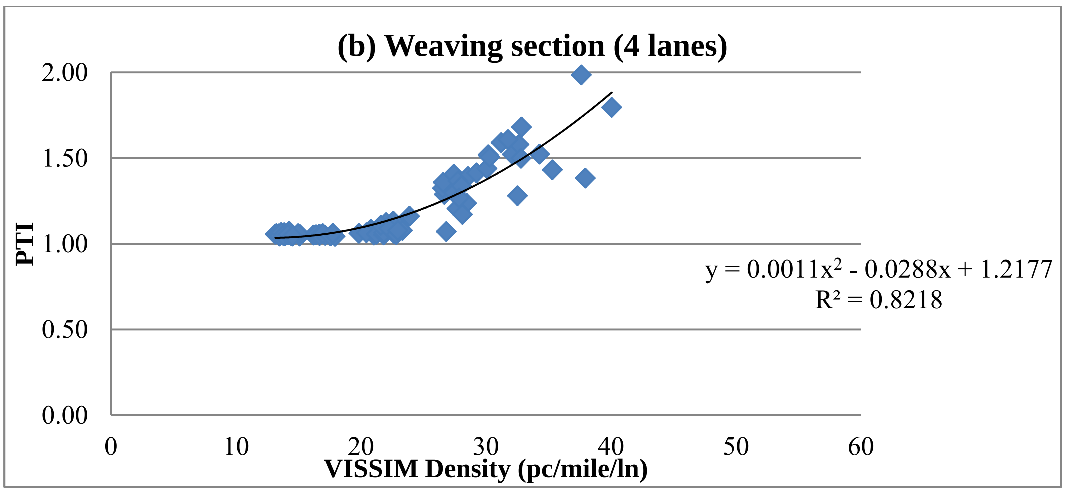


FIGURE 5.28: Density – PTI relationship for 65 mph speed limit on weaving section





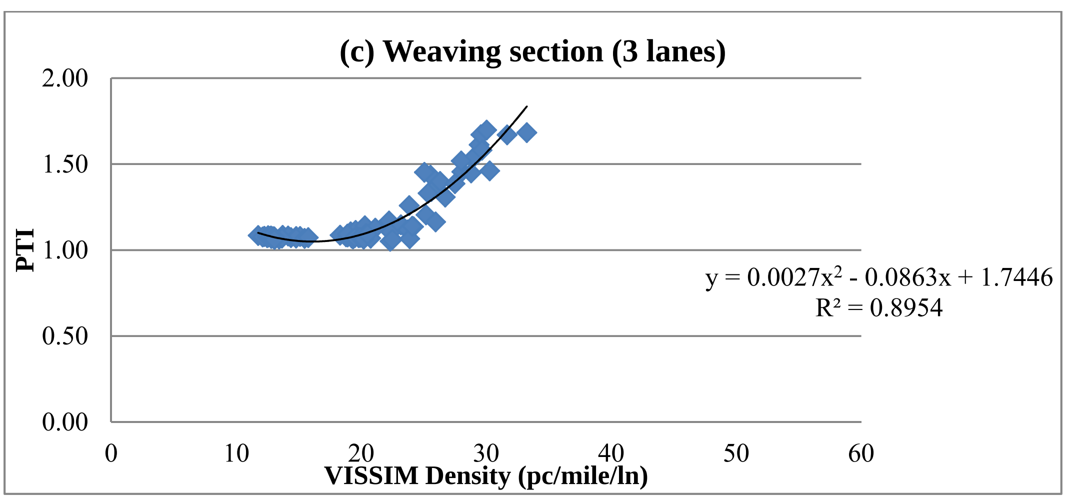


FIGURE 5.29: Density – PTI relationship for 60 mph speed limit on weaving section

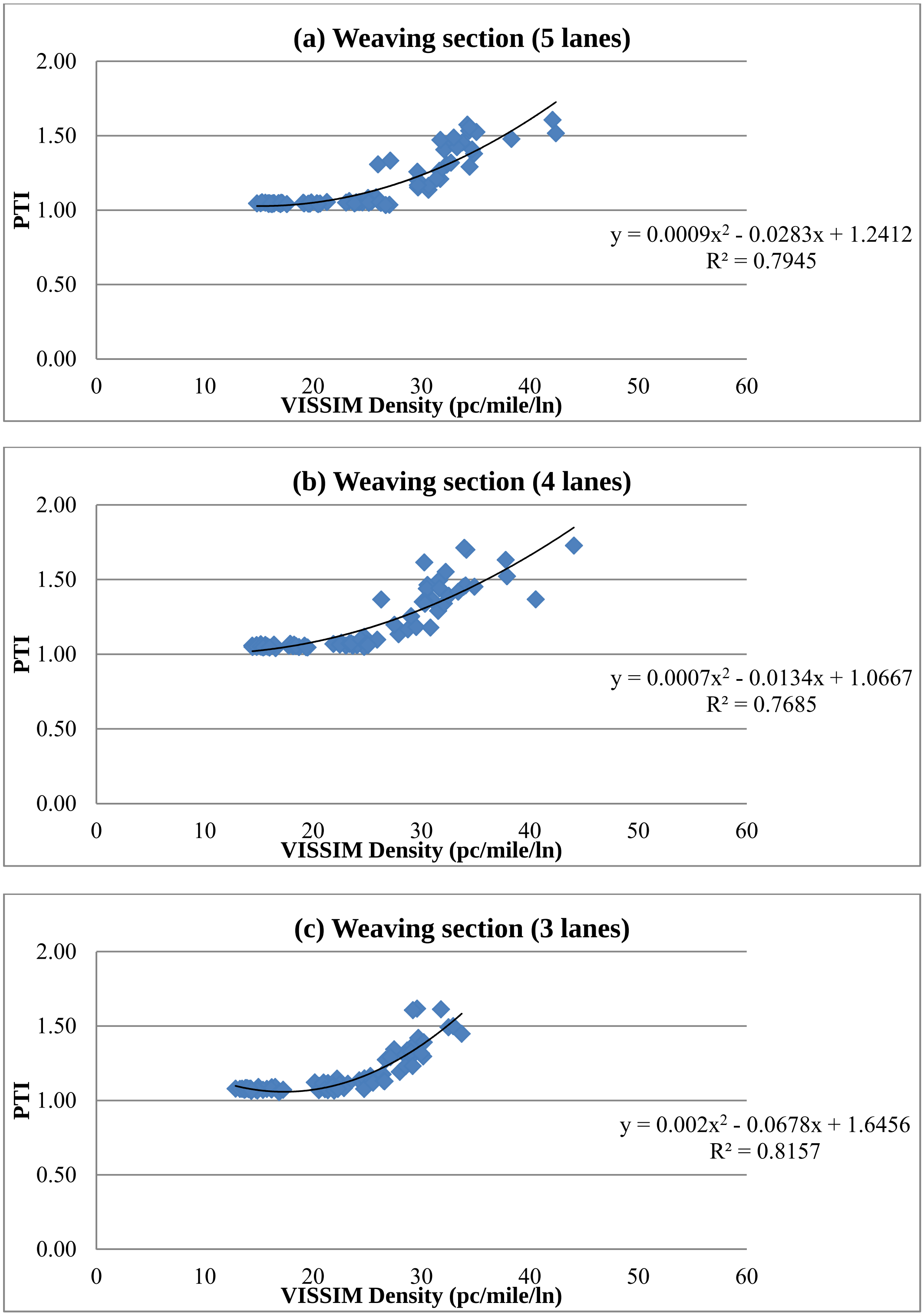


FIGURE 5.30: Density – PTI relationship for 55 mph speed limit on weaving section

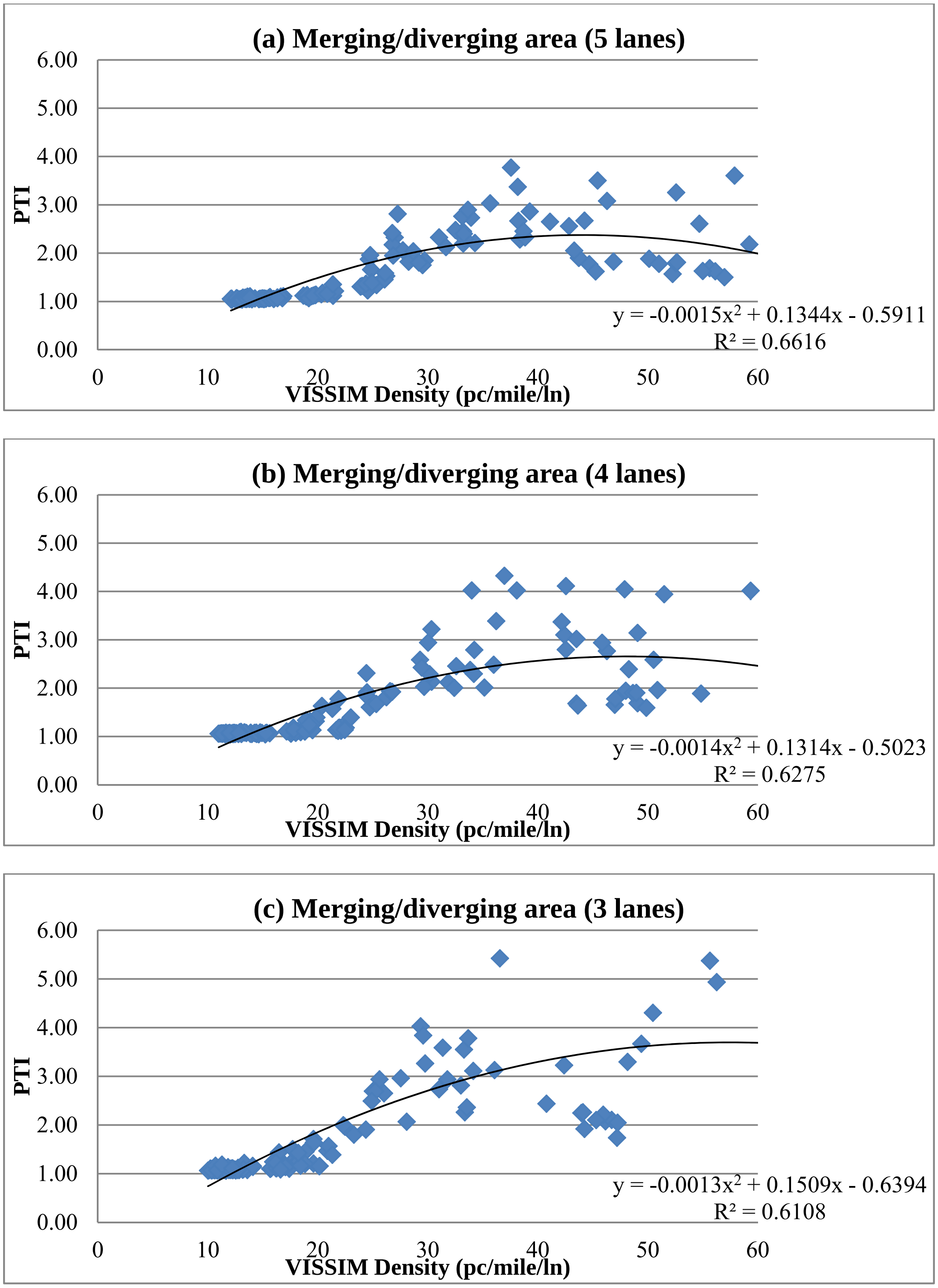
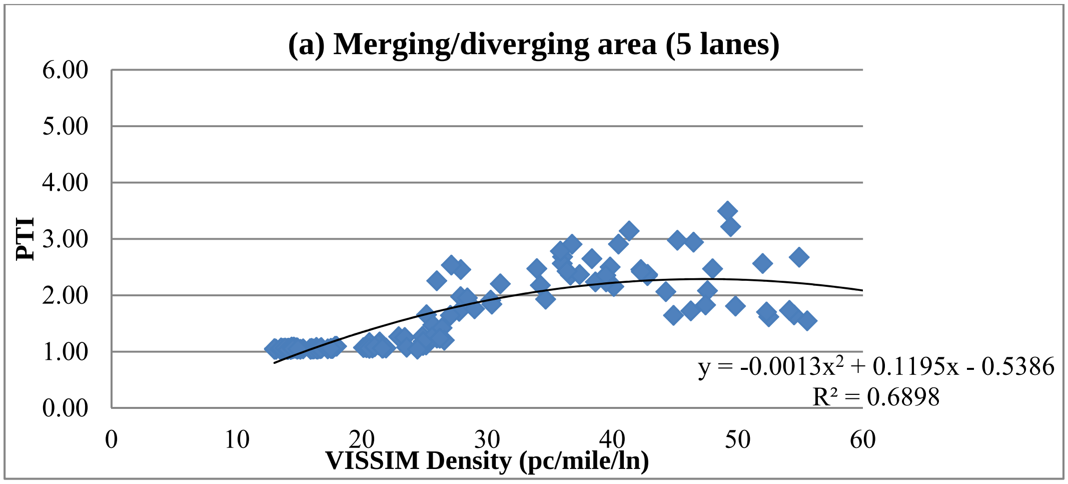
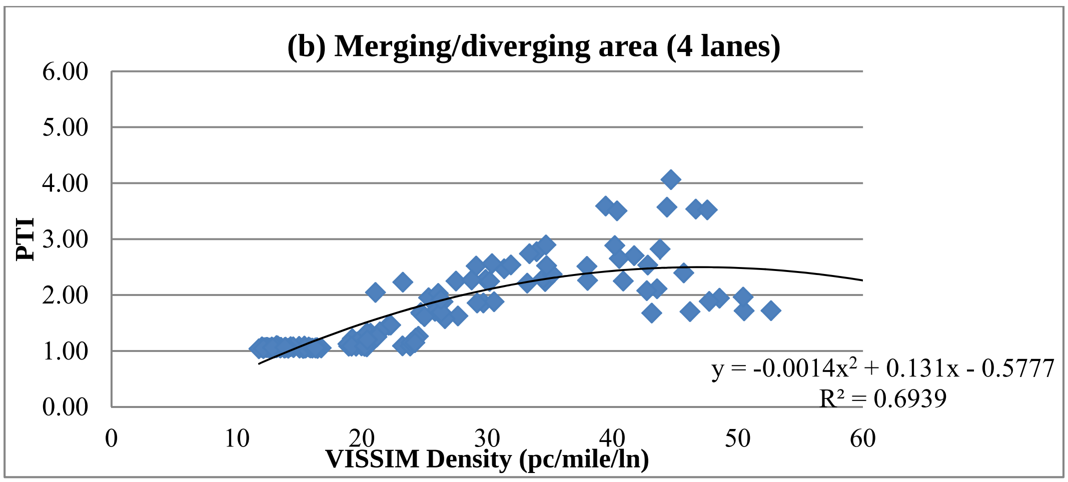


FIGURE 5.31: Density – PTI relationship for 70 mph speed limit on merging/diverging area





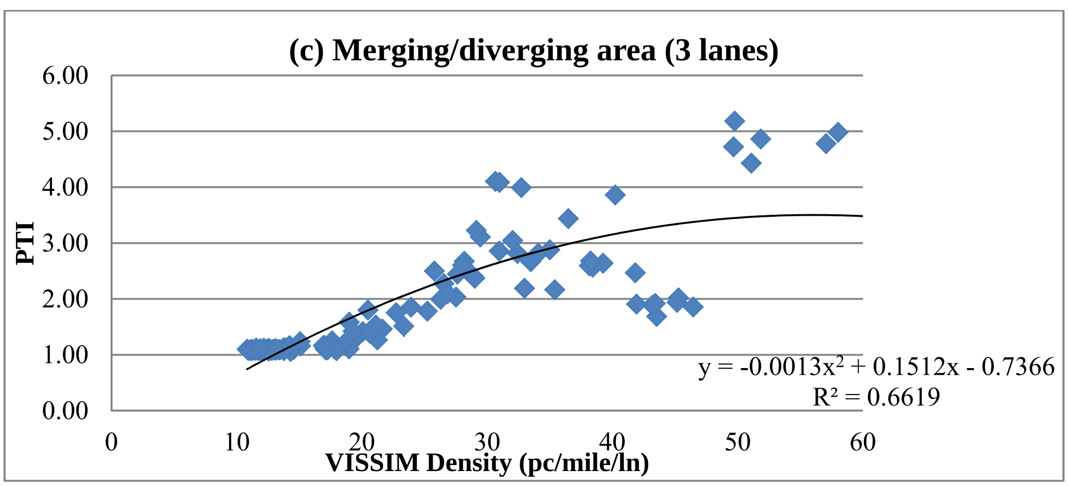


FIGURE 5.32: Density – PTI relationship for 65 mph speed limit on merging/diverging area

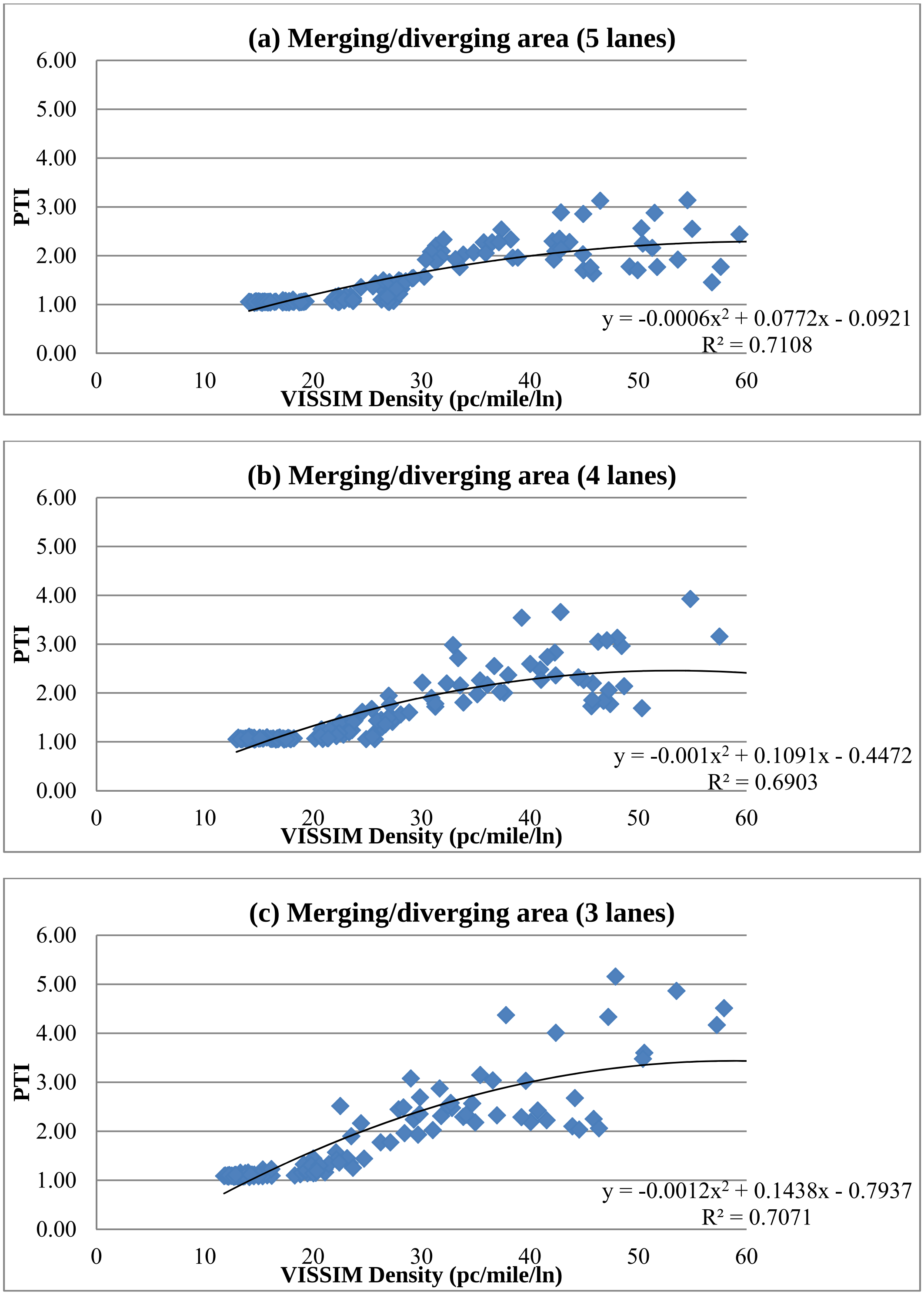
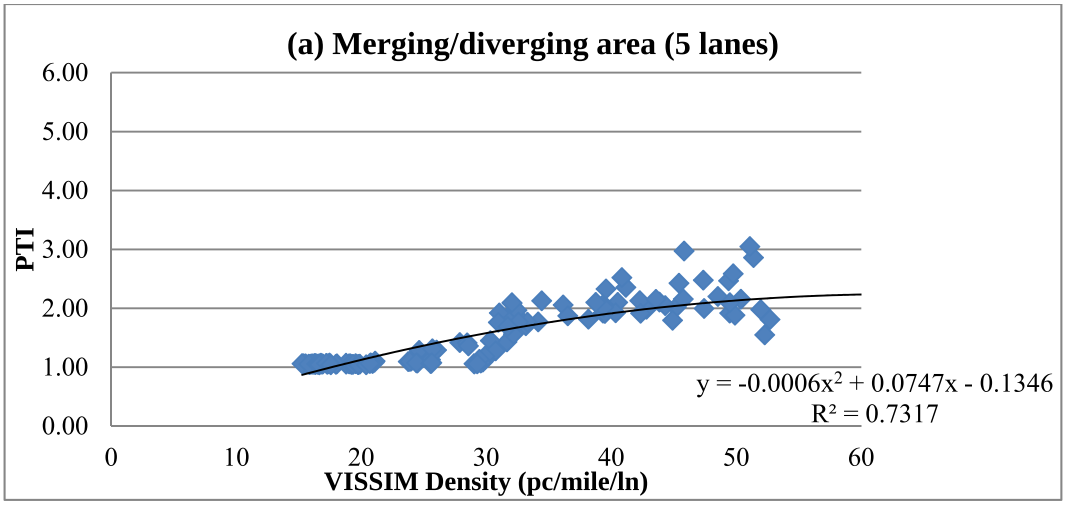
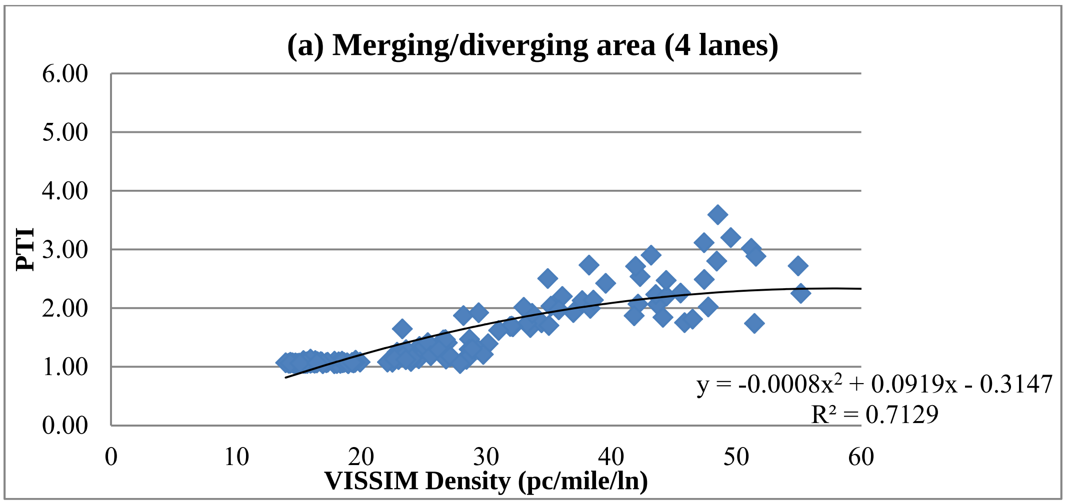


FIGURE 5.33: Density – PTI relationship for 60 mph speed limit on merging/diverging area





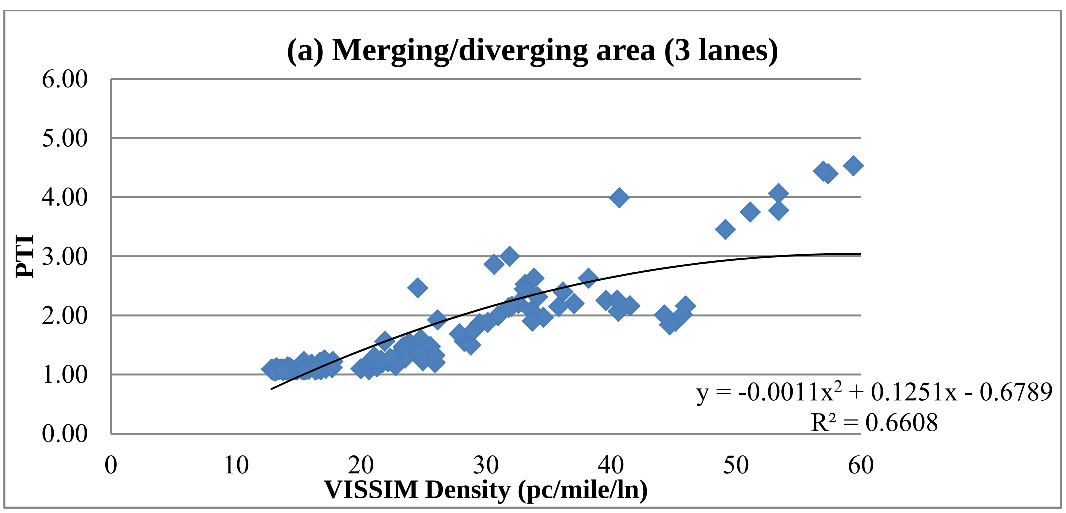


FIGURE 5.34: Density – PTI relationship for 55 mph speed limit on merging/diverging area

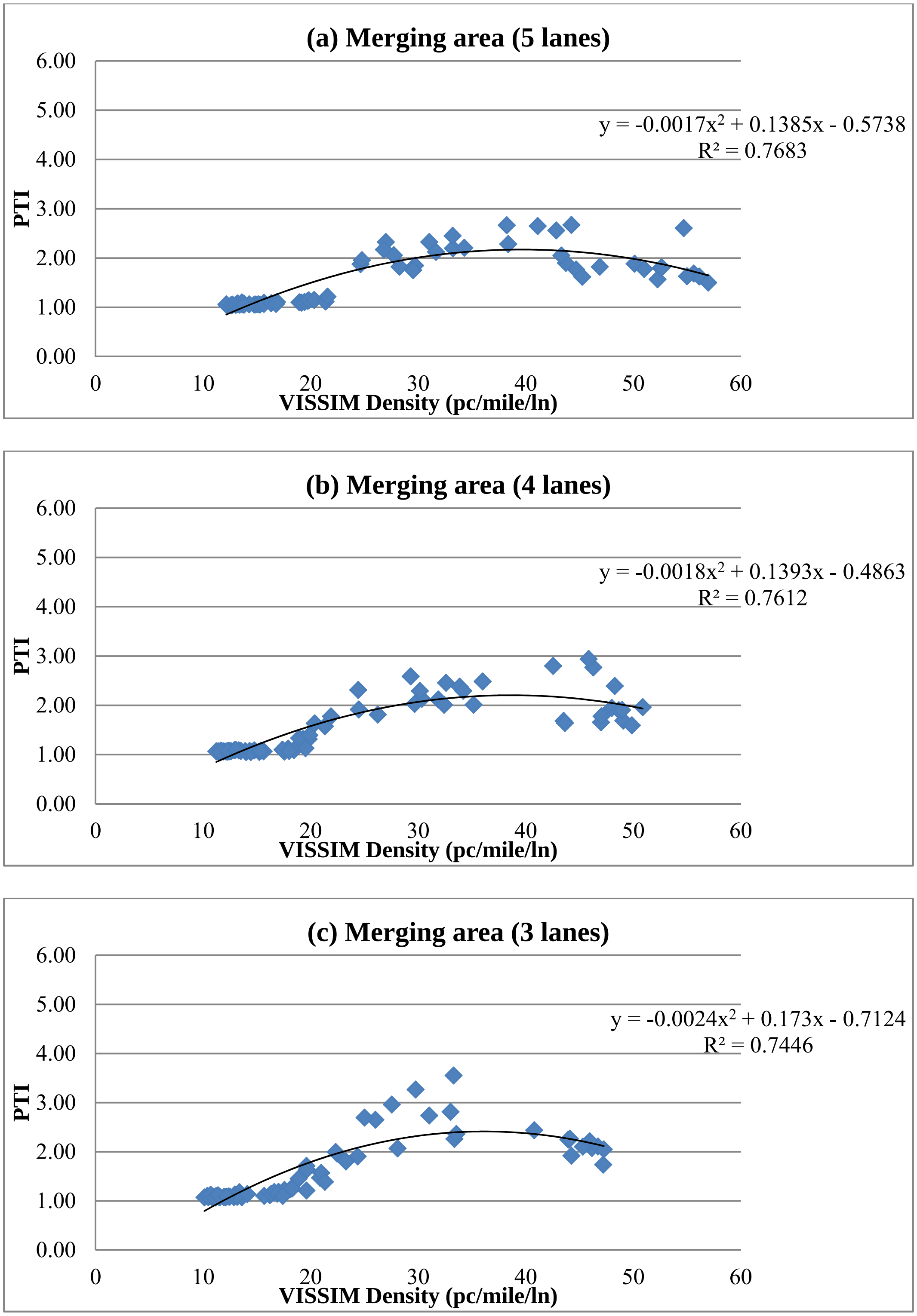


FIGURE 5.35: Density – PTI relationship for 70 mph speed limit on merging area

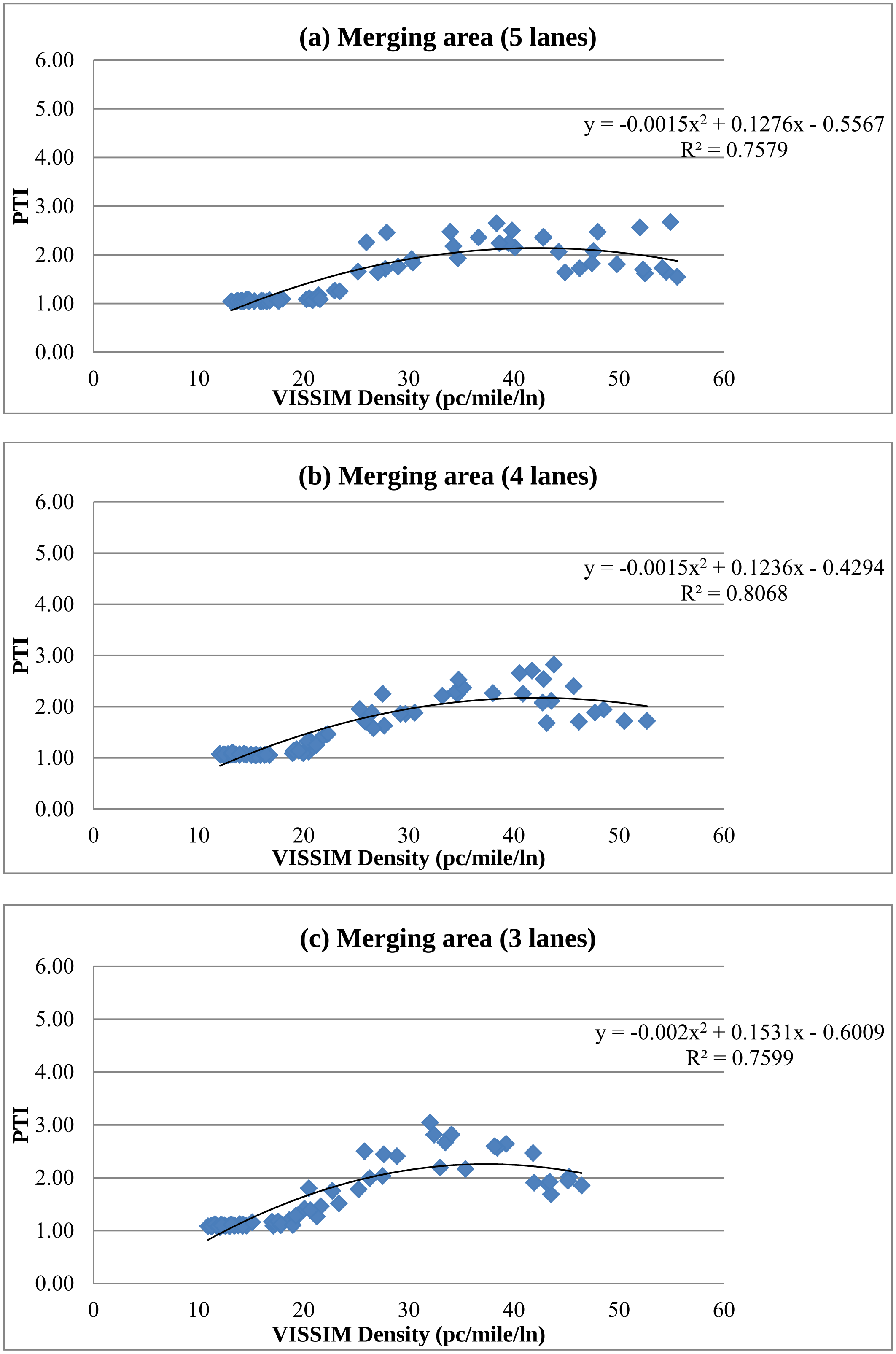


FIGURE 5.36: Density – PTI relationship for 65 mph speed limit on merging area

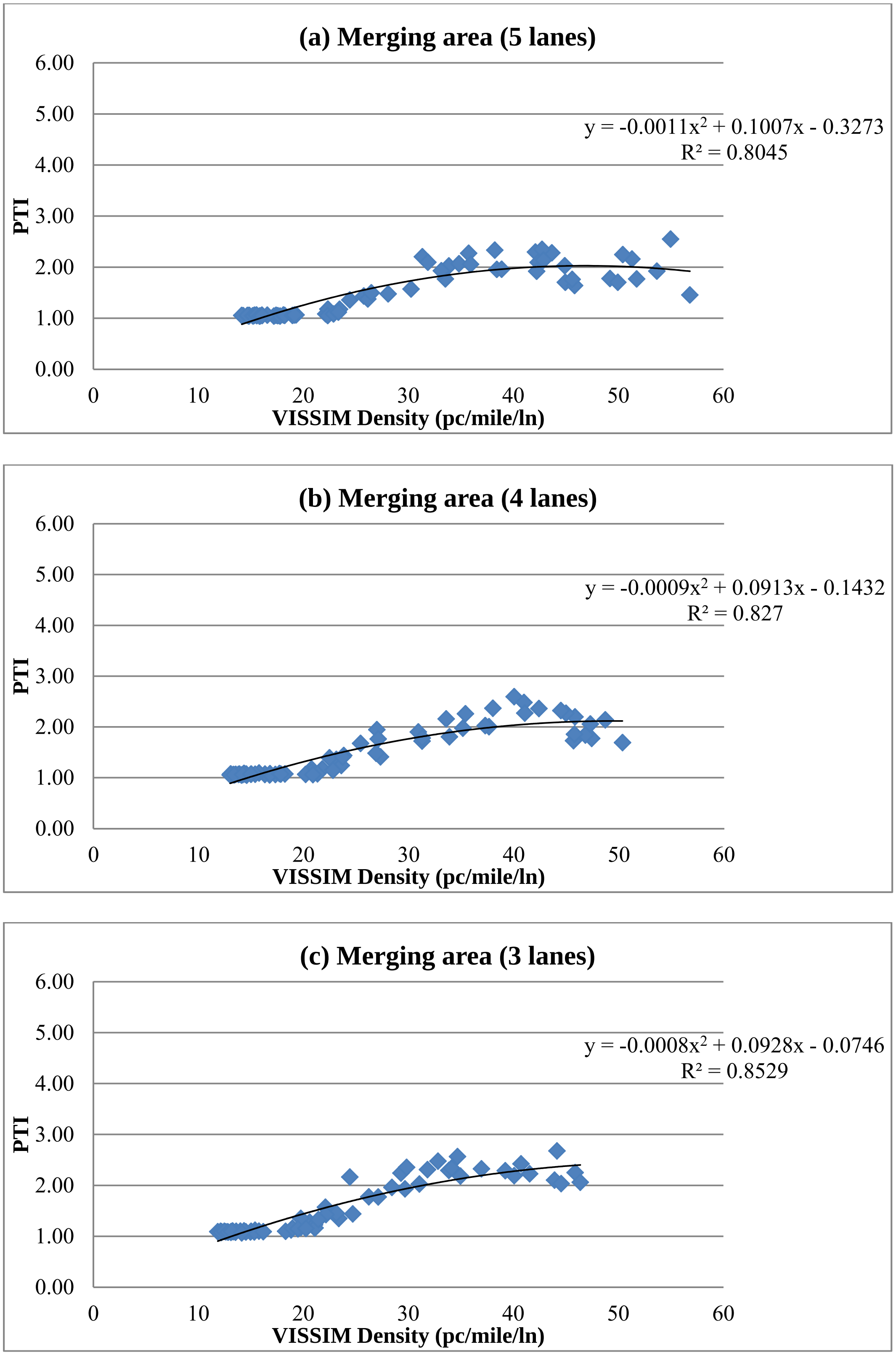


FIGURE 5.37: Density – PTI relationship for 60 mph speed limit on merging area

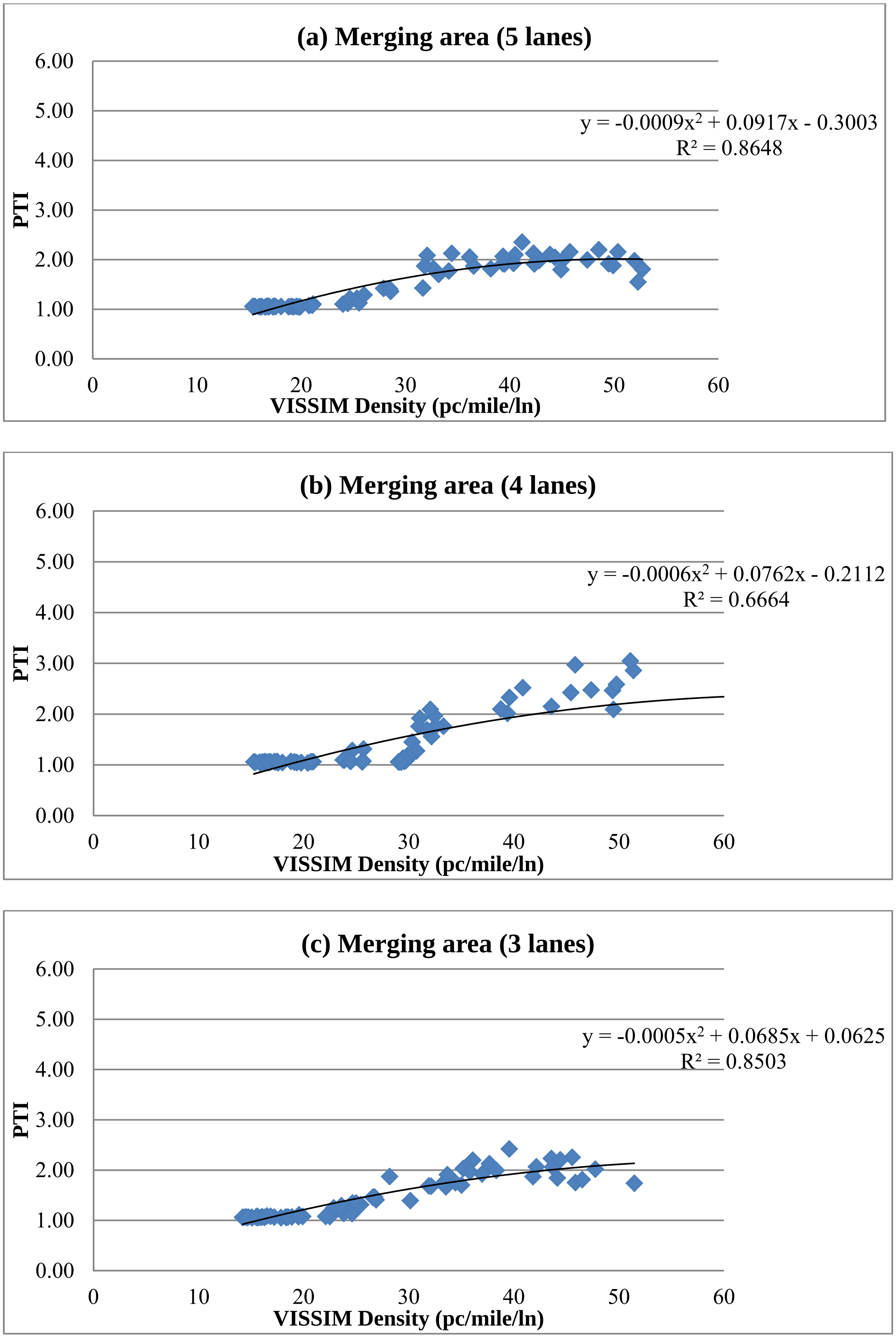


FIGURE 5.38: Density – PTI relationship for 55 mph speed limit on merging area

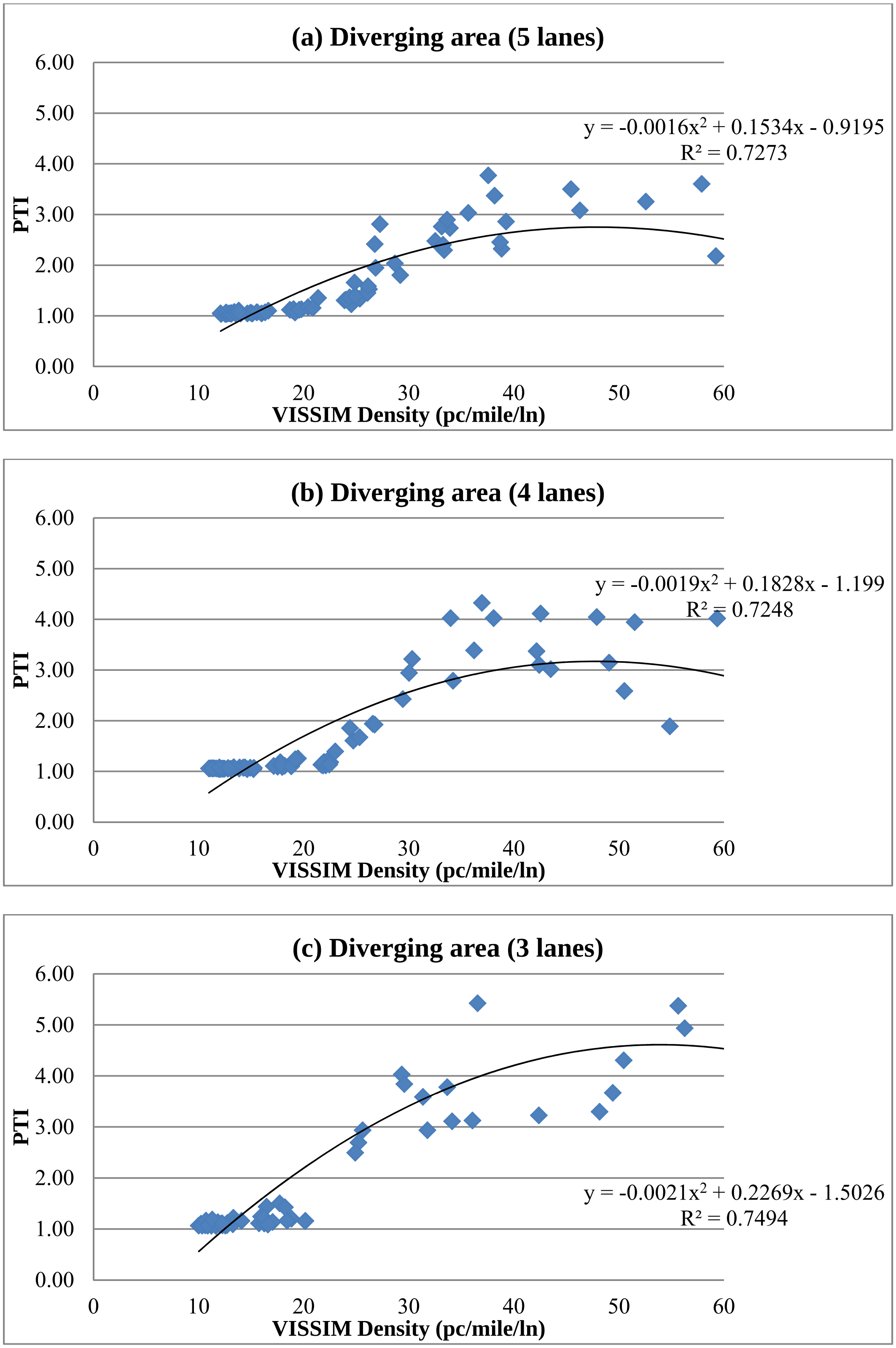


FIGURE 5.39: Density – PTI relationship for 70 mph speed limit on diverging area

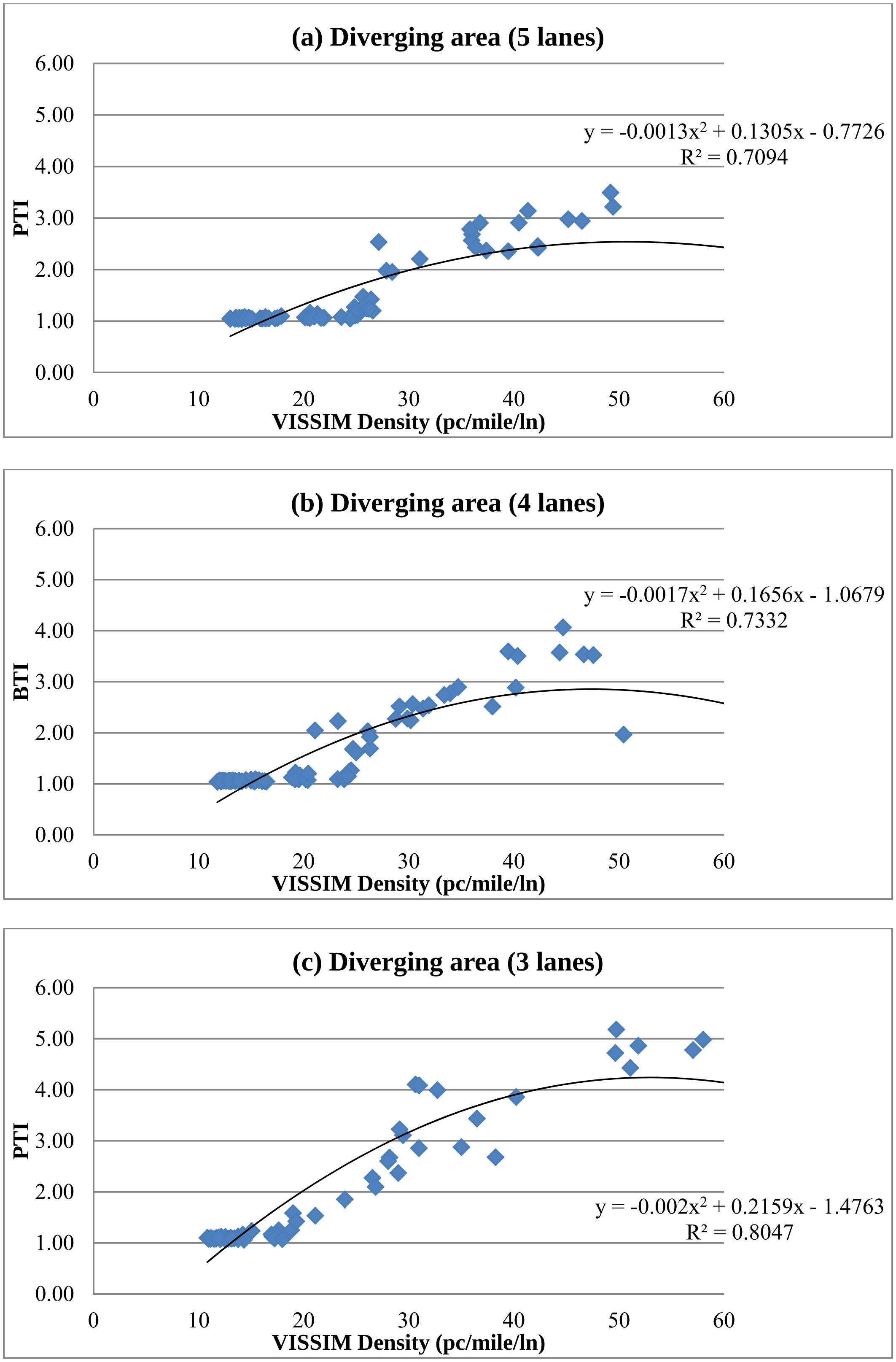


FIGURE 5.40: Density – PTI relationship for 65 mph speed limit on diverging area

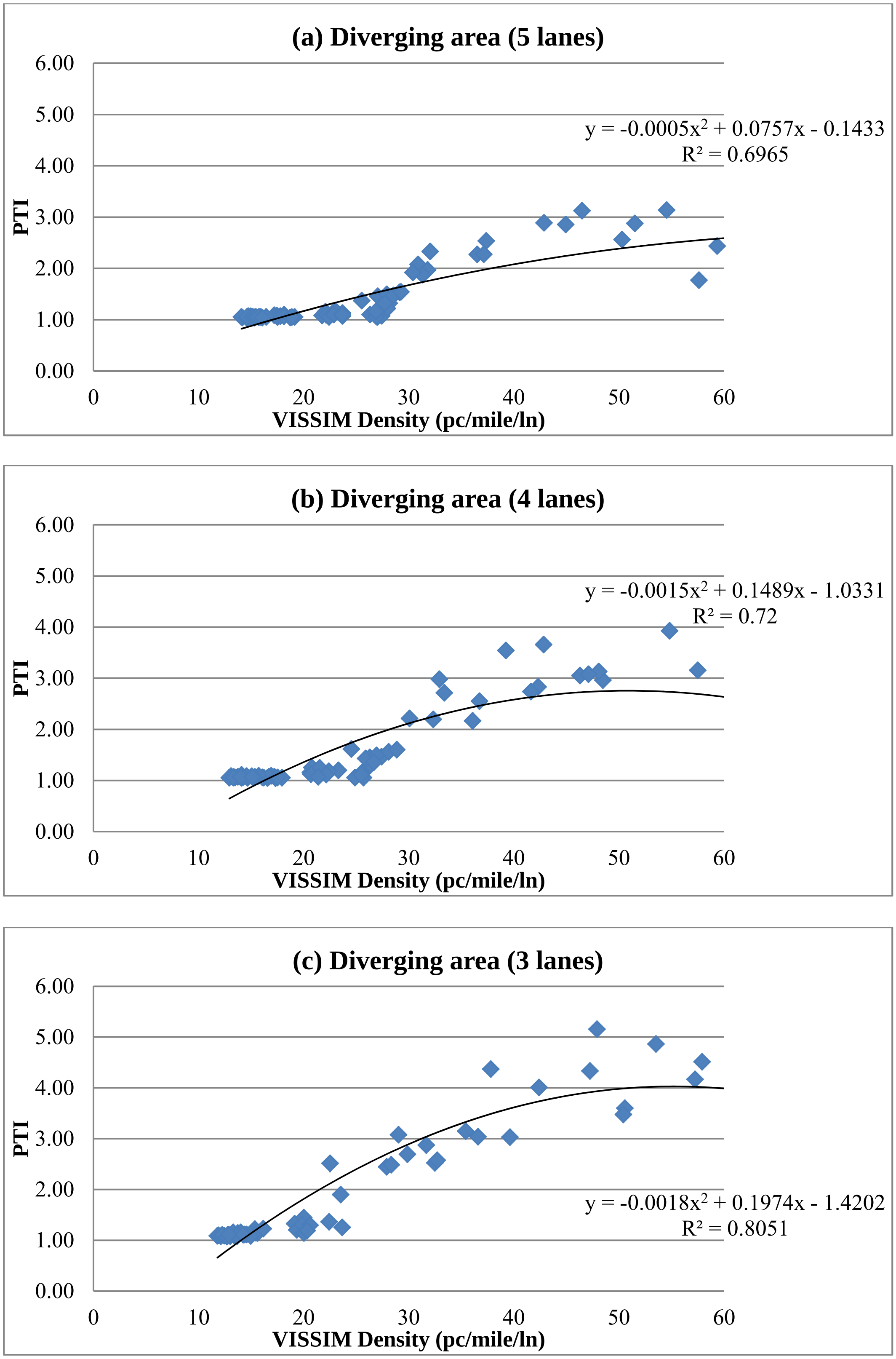


FIGURE 5.41: Density – PTI relationship for 60 mph speed limit on diverging area

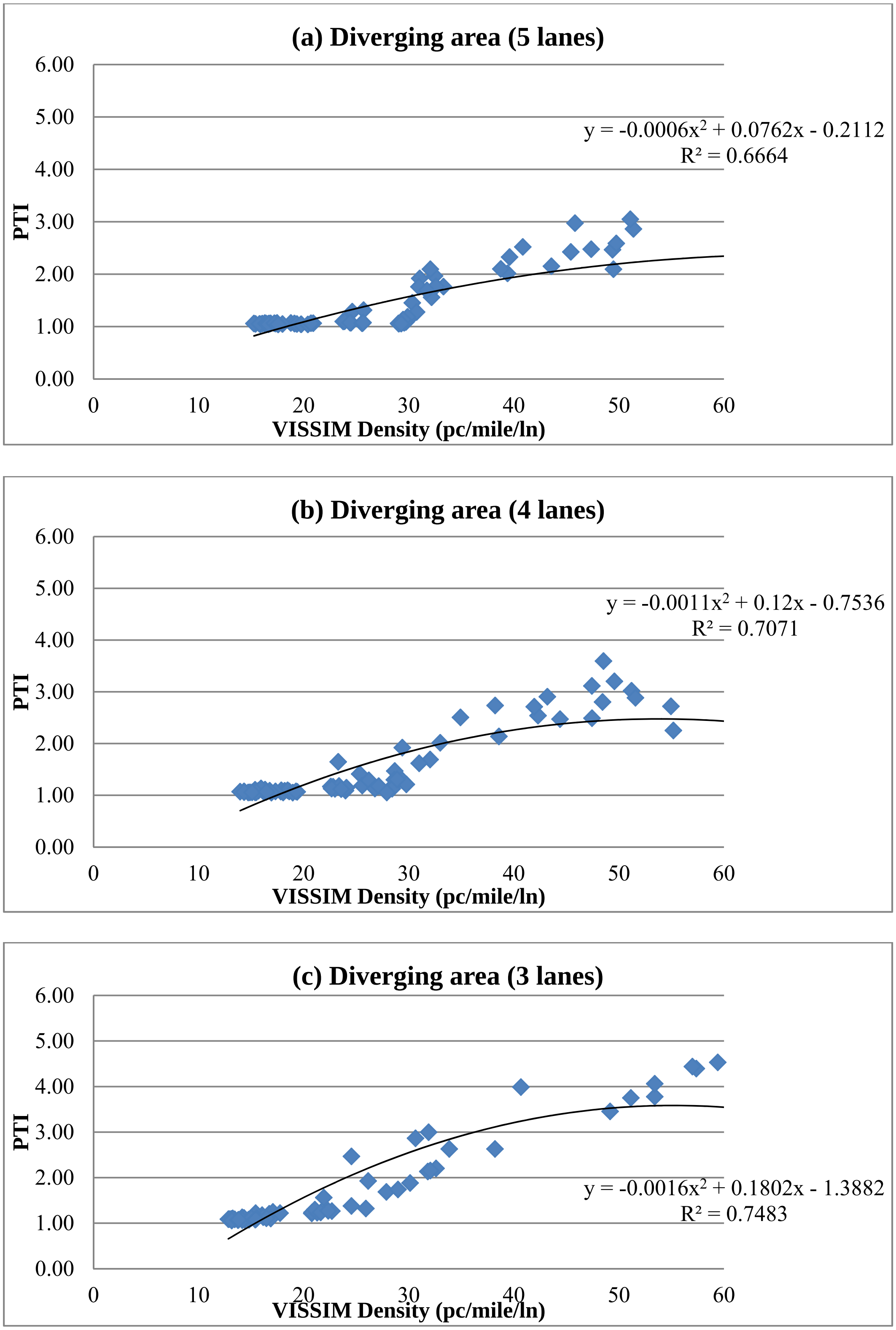


FIGURE 5.42: Density – PTI relationship for 55 mph speed limit on diverging area

5.5.2. Buffer Time Index

From VISSIM, 95th and 50th percentile travel time information were computed for each hour and used to compute the BTI values for each hour using Equation 3.2. They are coupled with their respective density values from VISSIM to establish the relationships between them.

5.5.2.1. Basic Freeway Section

Figures 5.43 to 5.46 illustrate the relationship between the density and BTI for 70, 65, 60, and 55 mph speed limits on a basic freeway section, respectively. The relationships were developed based on the available lane exposure on the freeway. A non-linear (polynomial) relationship was observed between the two variables.

5.5.2.2. Weaving Section

Figures 5.47 to 5.50 provide the scatter plots between the density and BTI for 70, 65, 60, and 55 mph speed limits on a weaving section, respectively, based on the available lane exposure. A polynomial relationship was observed between the two variables.

5.5.2.3. Merging/Diverging Area

The relationship between the density and BTI for 70, 65, 60, and 55 mph speed limits on a merging/diverging area, are shown in Figures 5.51 to 5.54, respectively. A polynomial relationship was observed between the two variables. However, Figures 5.55 to 5.58 illustrate the relationship between BTI and their respective density values in merging areas, while Figures 5.59 to 5.62 illustrate for the diverging areas to identify any potential effect on BTI by the difference in maneuverability in these two areas.

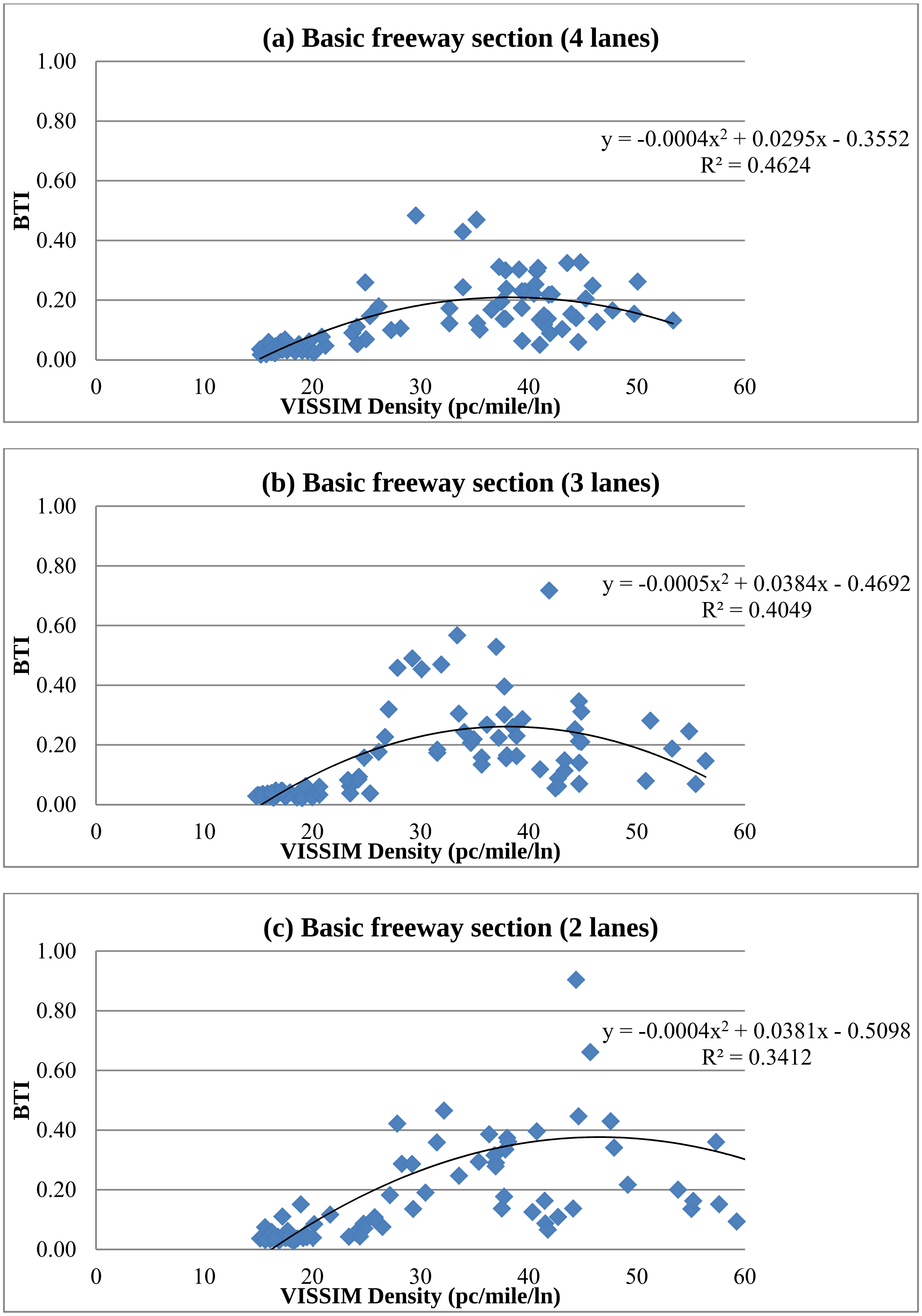


FIGURE 5.43: Density – BTI relationship for 70 mph speed limit on basic freeway section

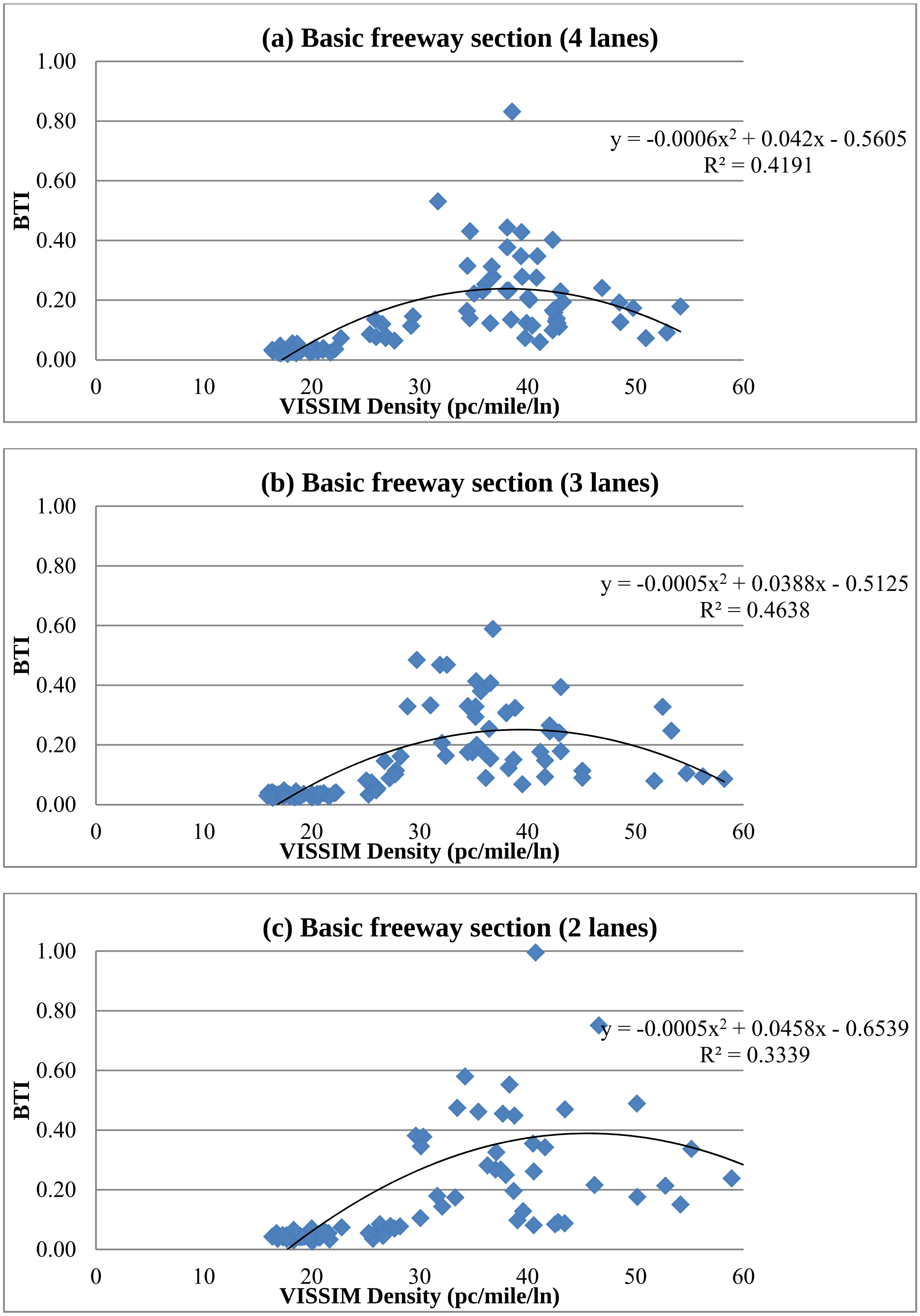


FIGURE 5.44: Density – BTI relationship for 65 mph speed limit on basic freeway section

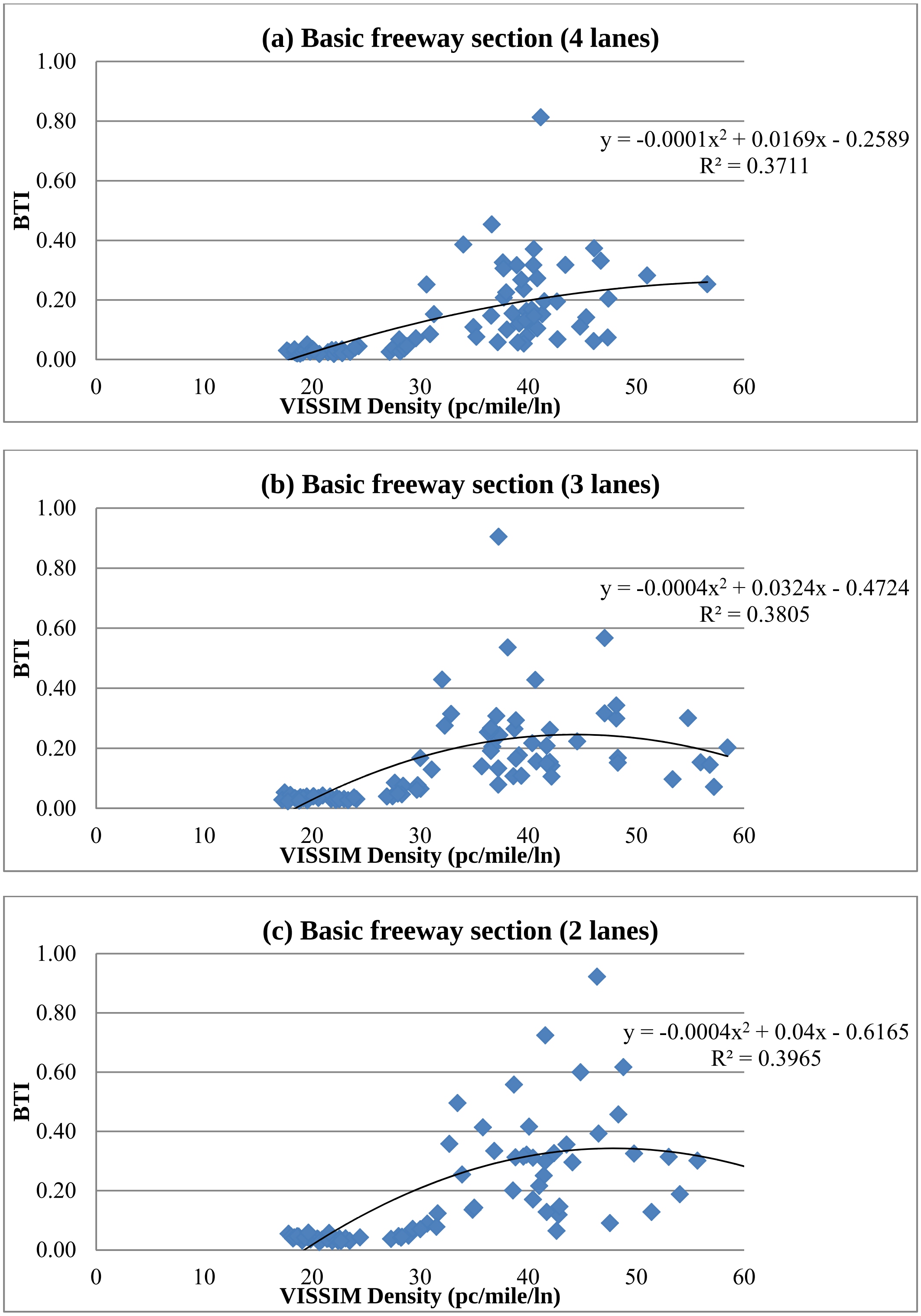


FIGURE 5.45: Density – BTI relationship for 60 mph speed limit on basic freeway section

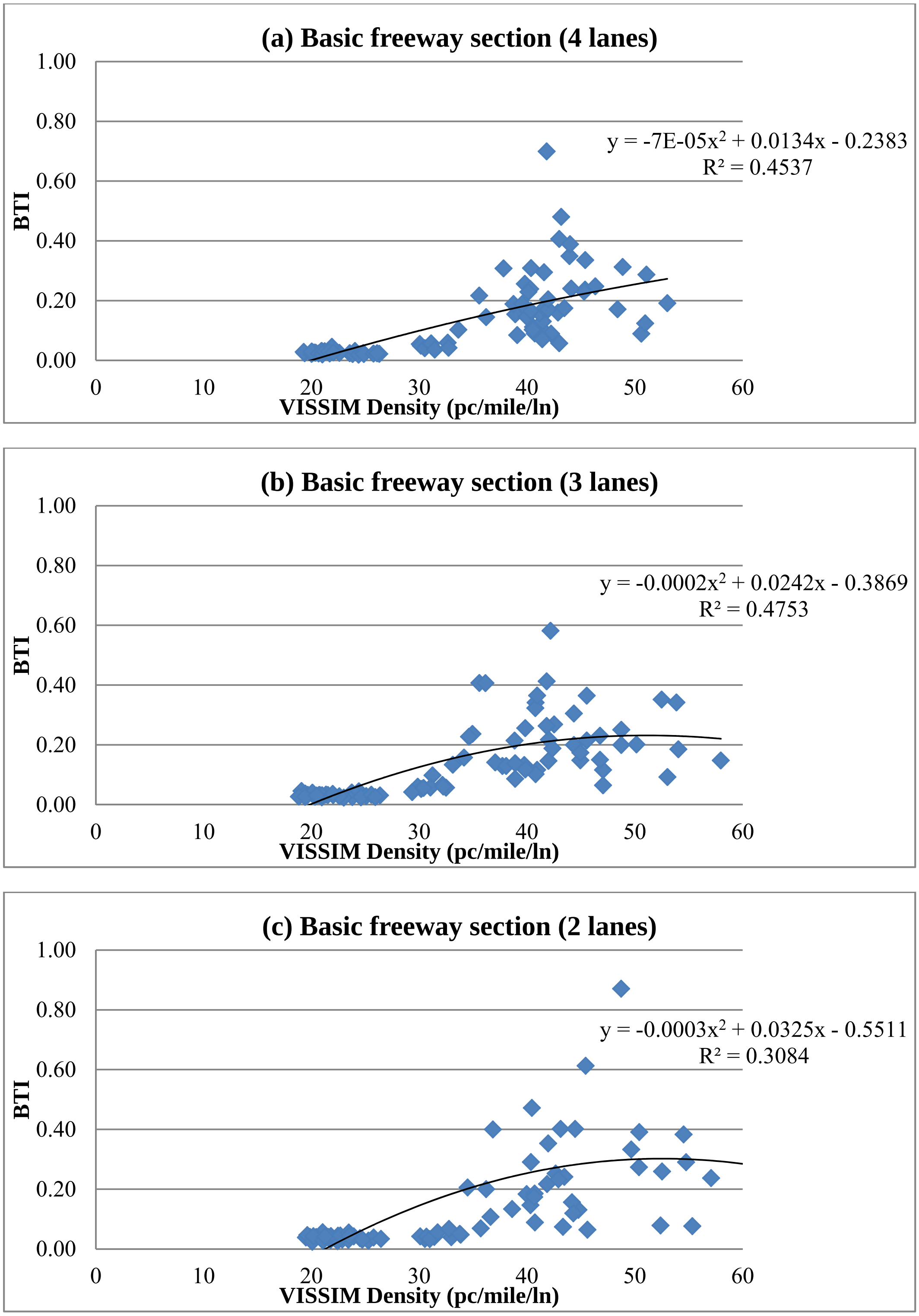


FIGURE 5.46: Density – BTI relationship for 55 mph speed limit on basic freeway section

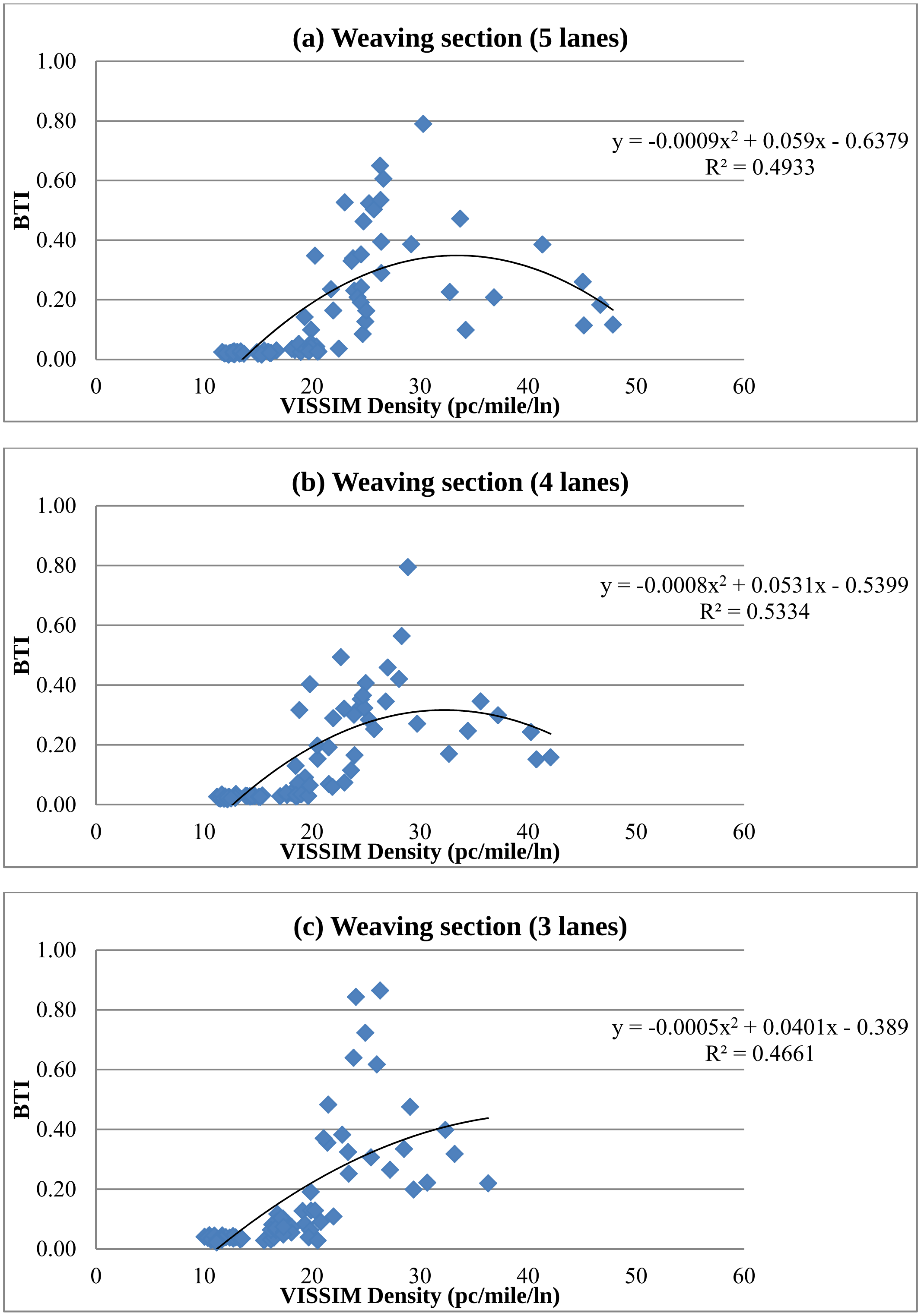


FIGURE 5.47: Density – BTI relationship for 70 mph speed limit on weaving section

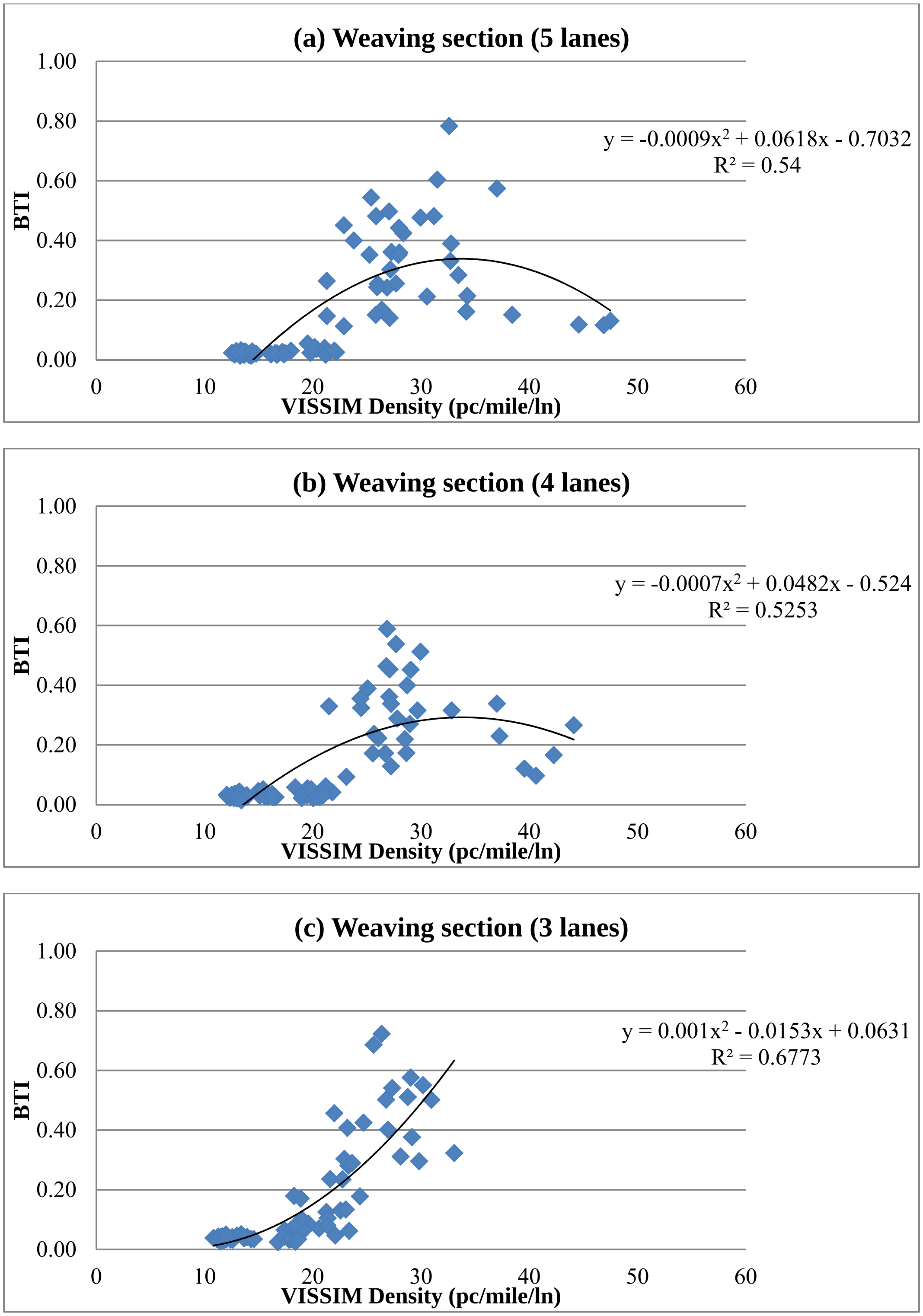


FIGURE 5.48: Density – BTI relationship for 65 mph speed limit on weaving section

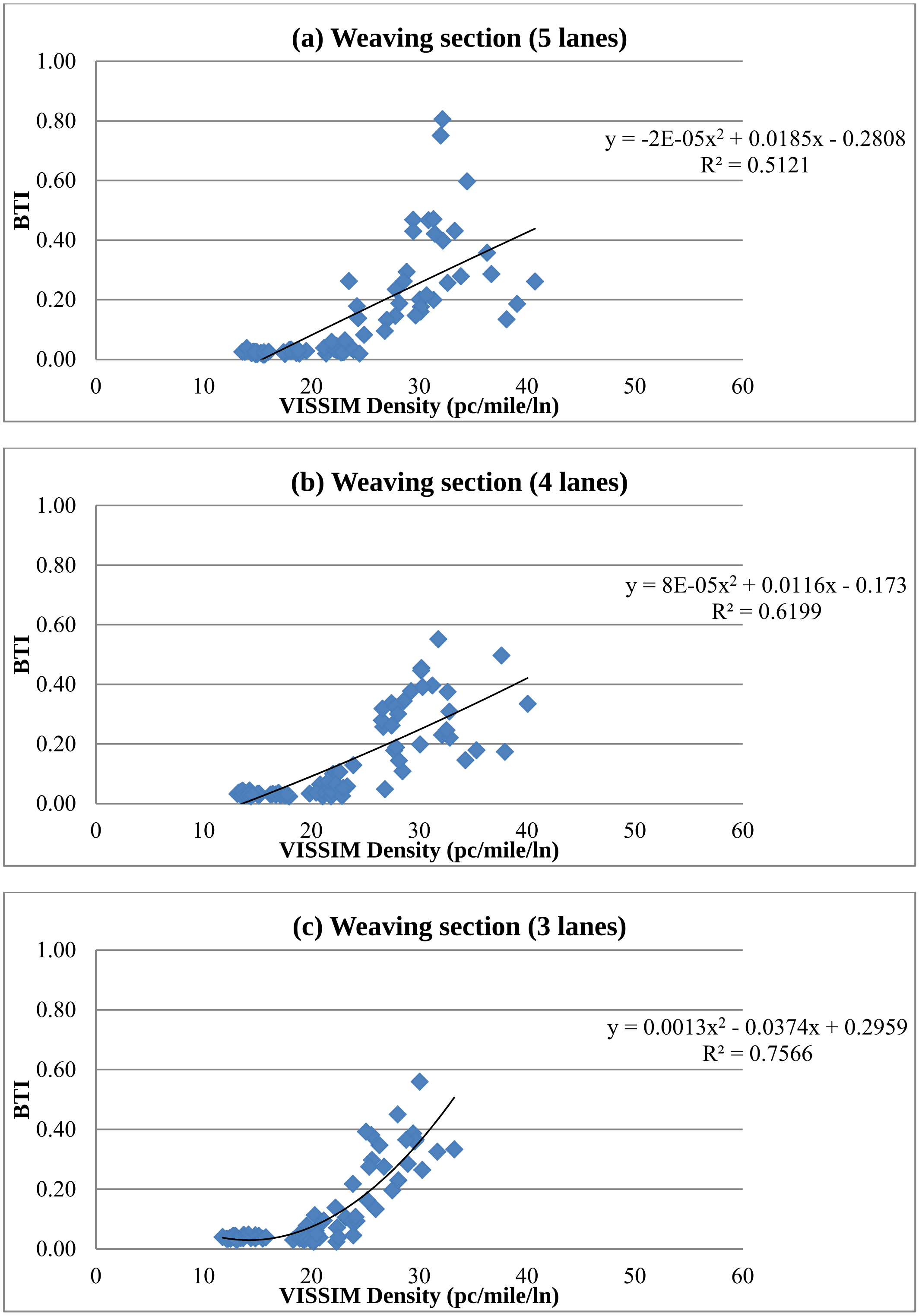


FIGURE 5.49: Density – BTI relationship for 60 mph speed limit on weaving section

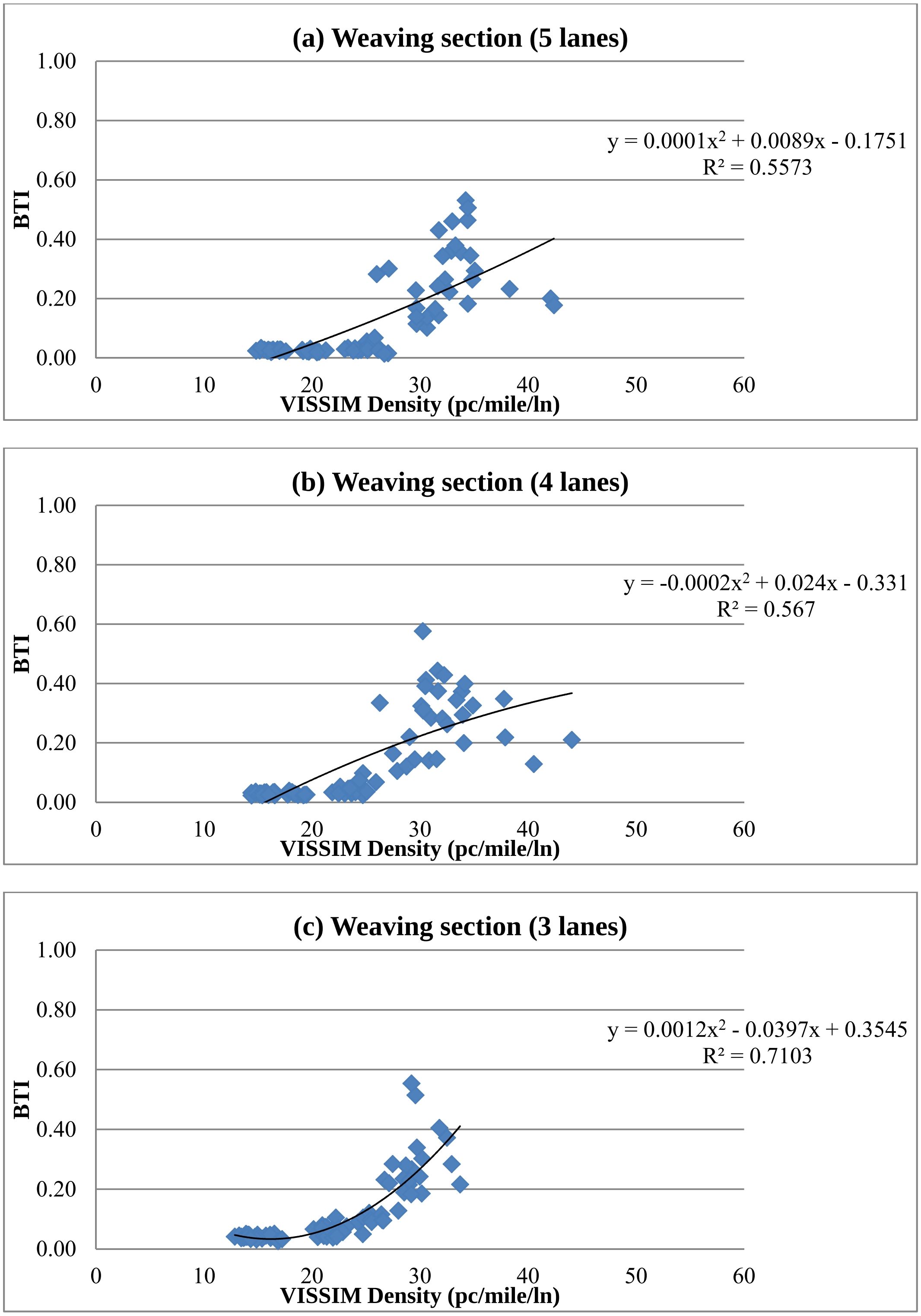


FIGURE 5.50: Density – BTI relationship for 55 mph speed limit on weaving section

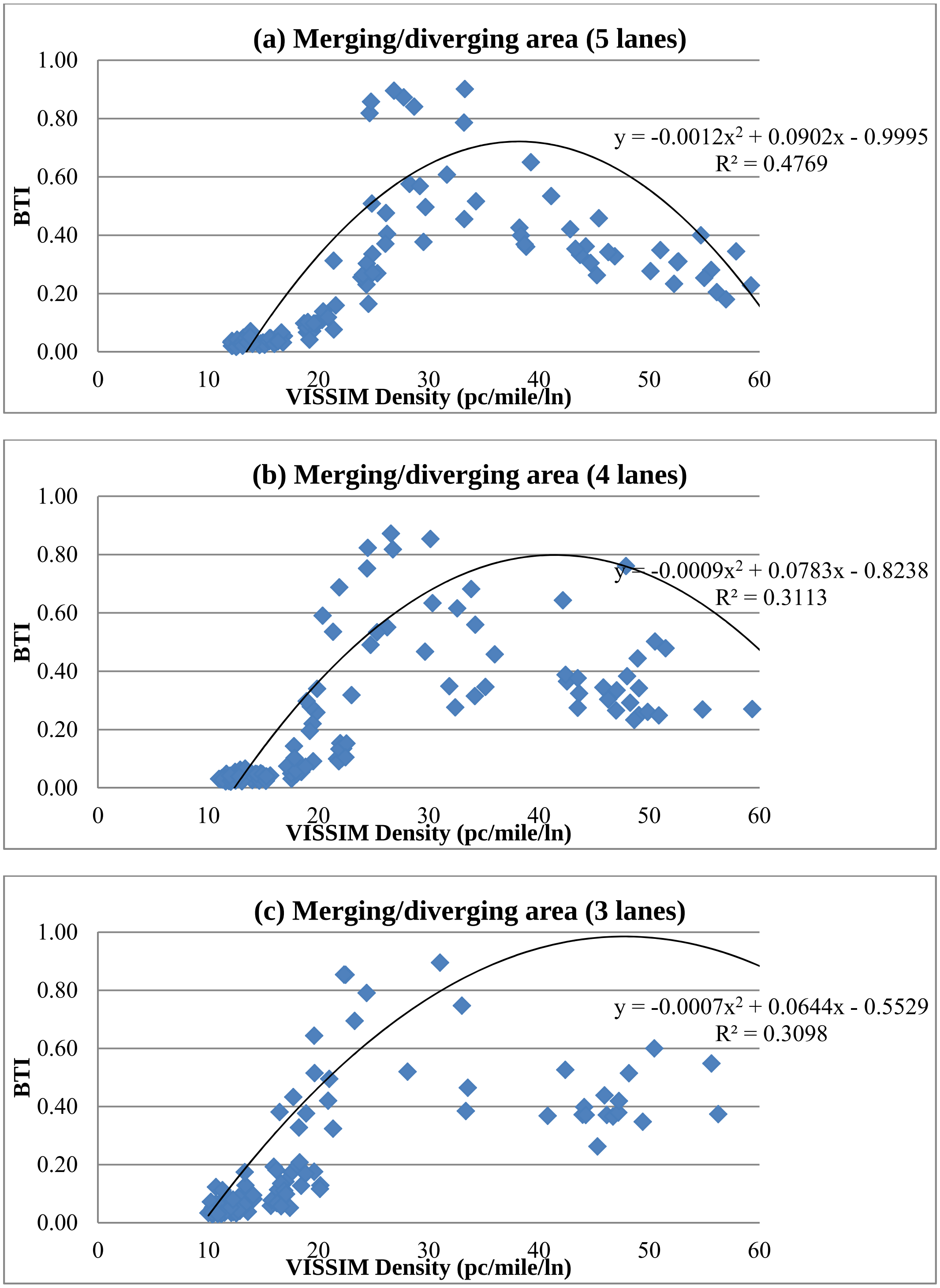


FIGURE 5.51: Density – BTI relationship for 70 mph speed limit on merging/diverging area

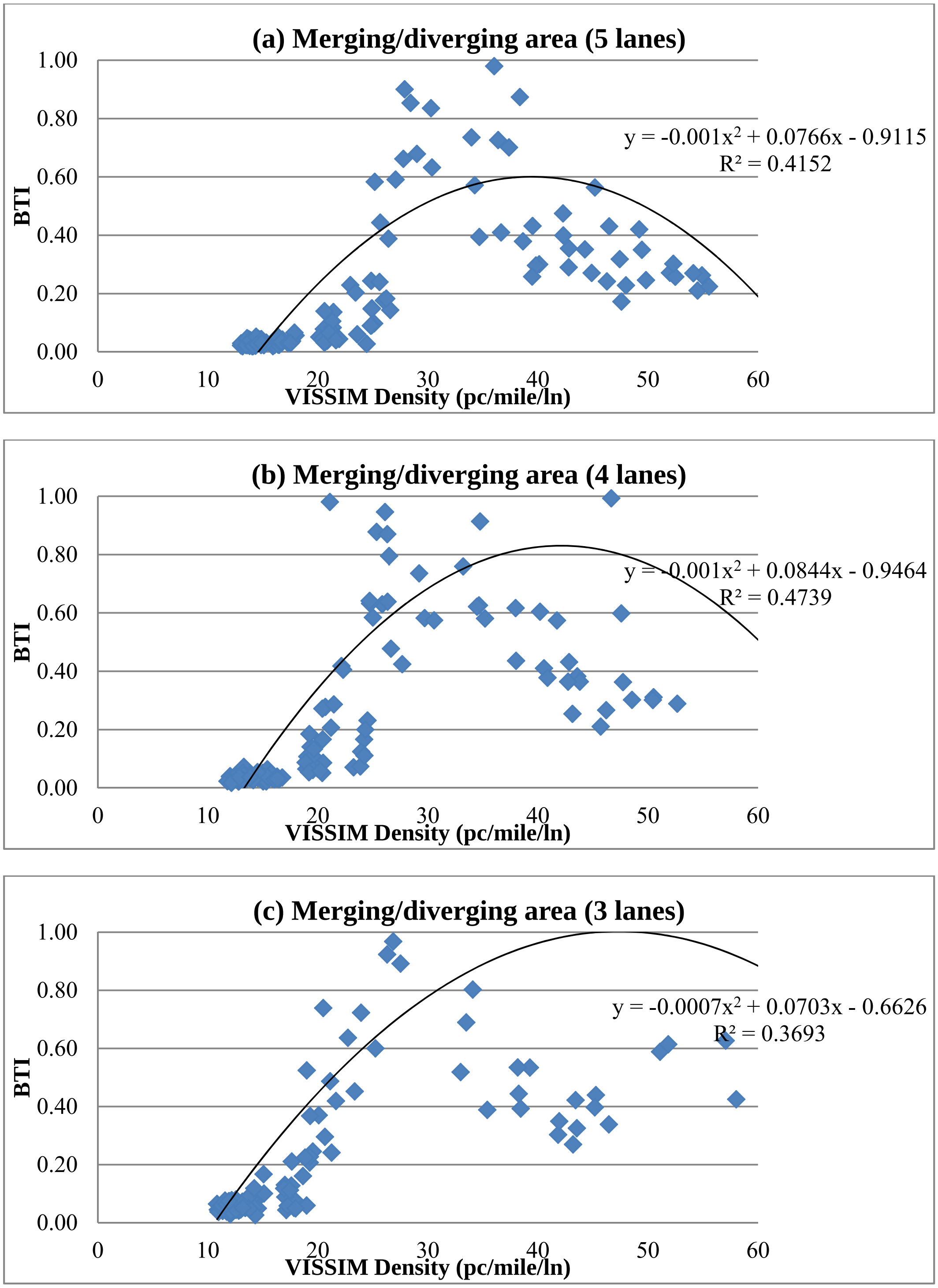


FIGURE 5.52: Density – BTI relationship for 65 mph speed limit on merging/diverging area

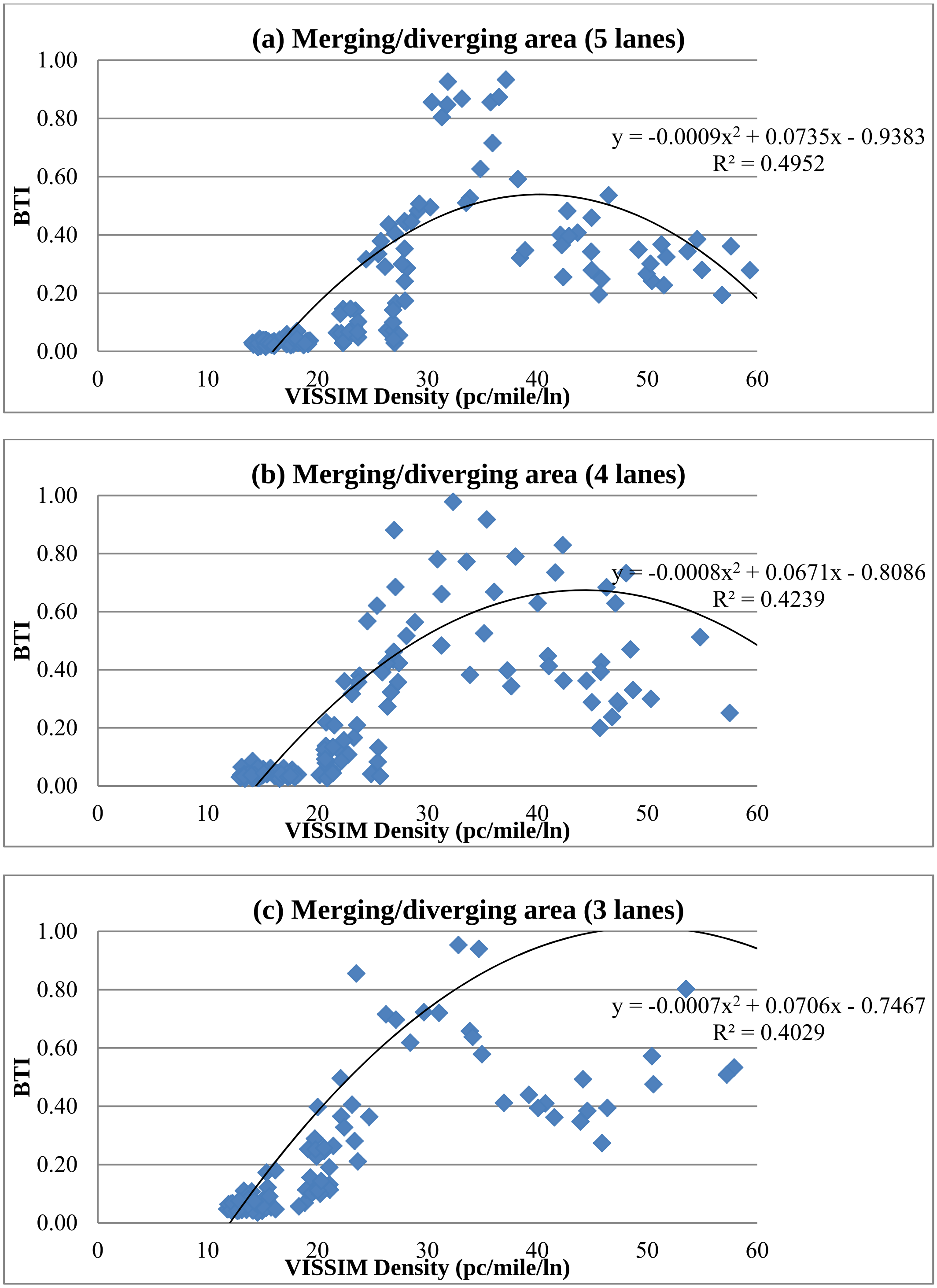


FIGURE 5.53: Density – BTI relationship for 60 mph speed limit on merging/diverging area

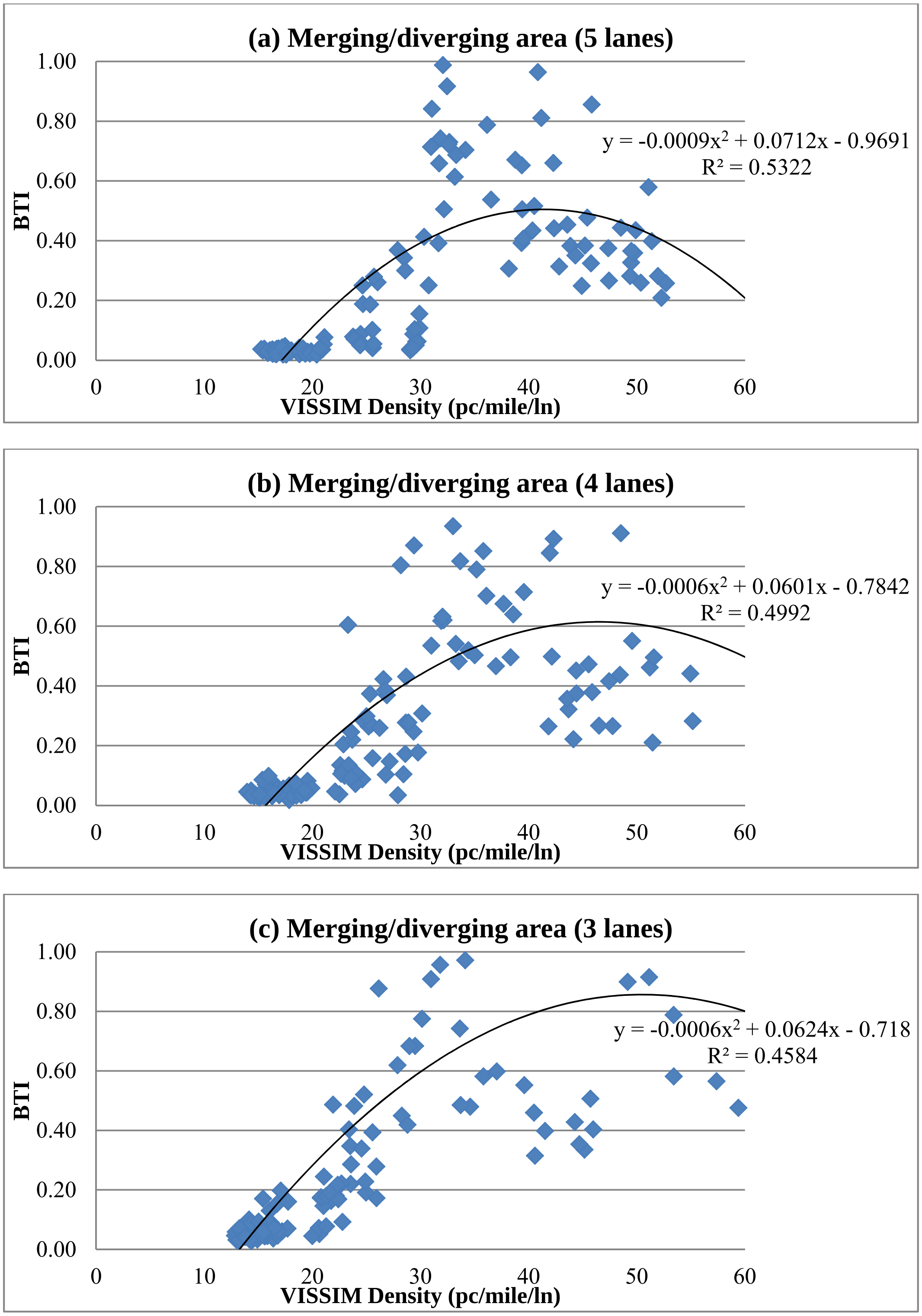


FIGURE 5.54: Density – BTI relationship for 55 mph speed limit on merging/diverging area

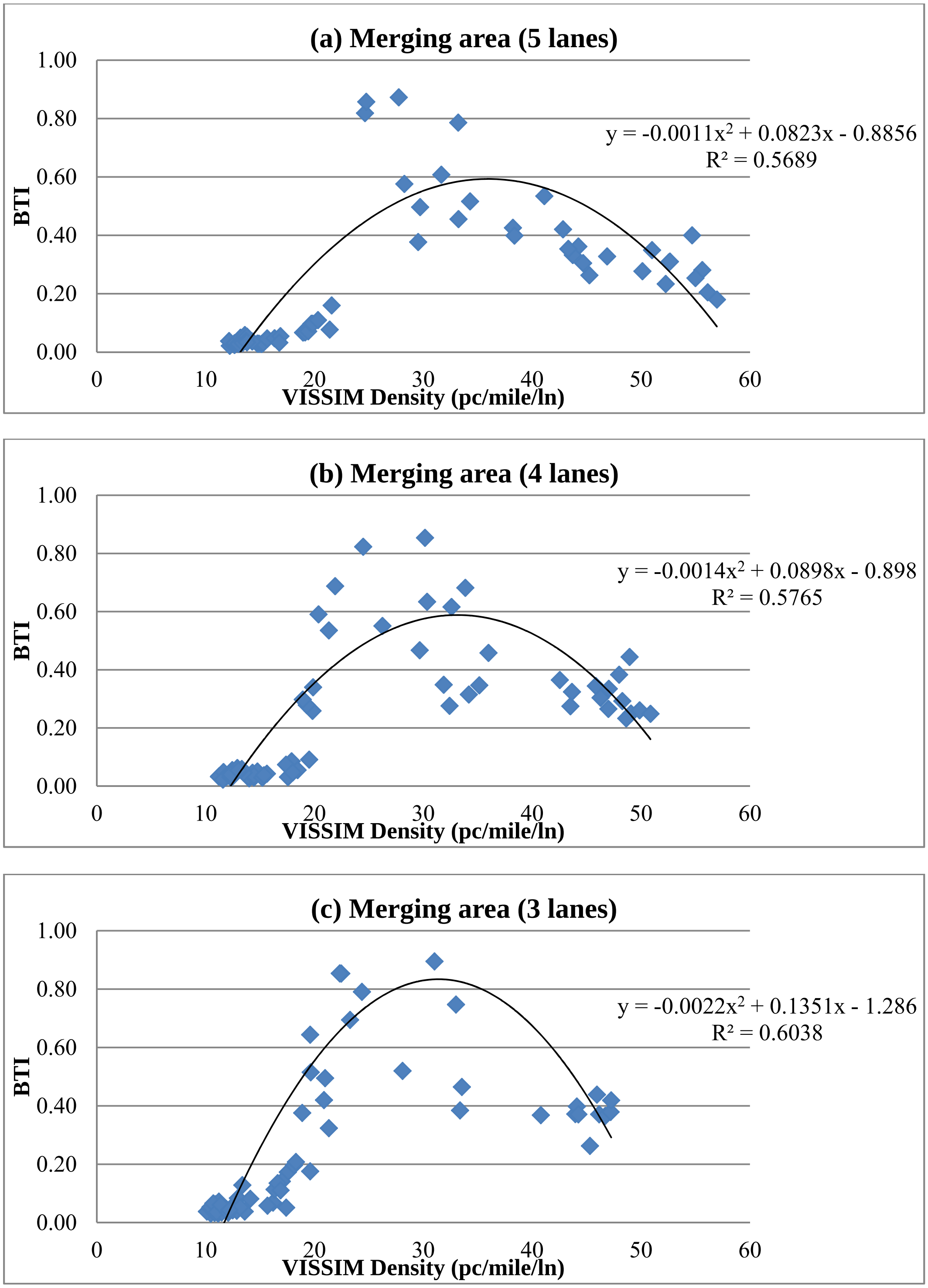


FIGURE 5.55: Density – BTI relationship for 70 mph speed limit on merging area

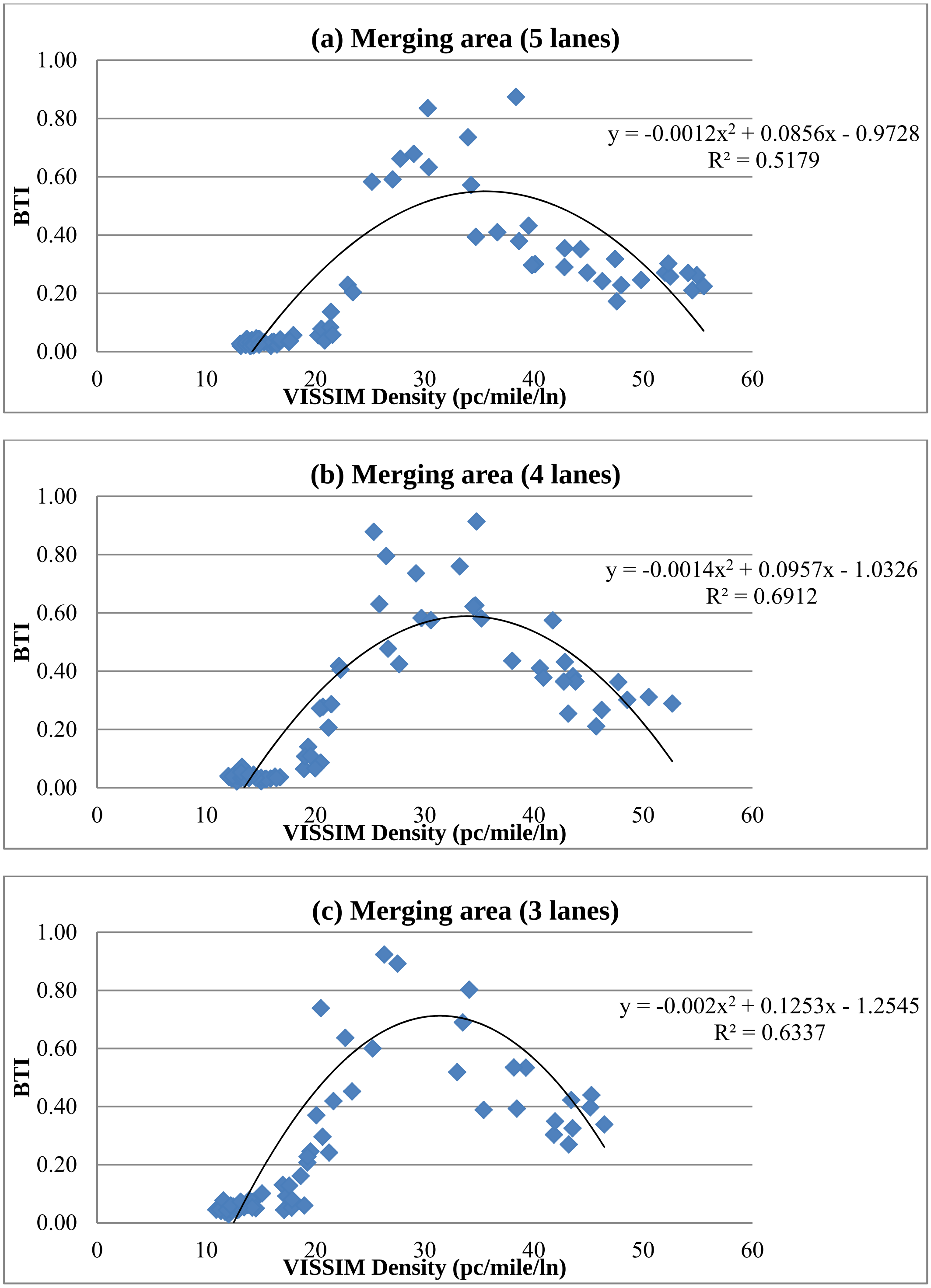


FIGURE 5.56: Density – BTI relationship for 65 mph speed limit on merging area

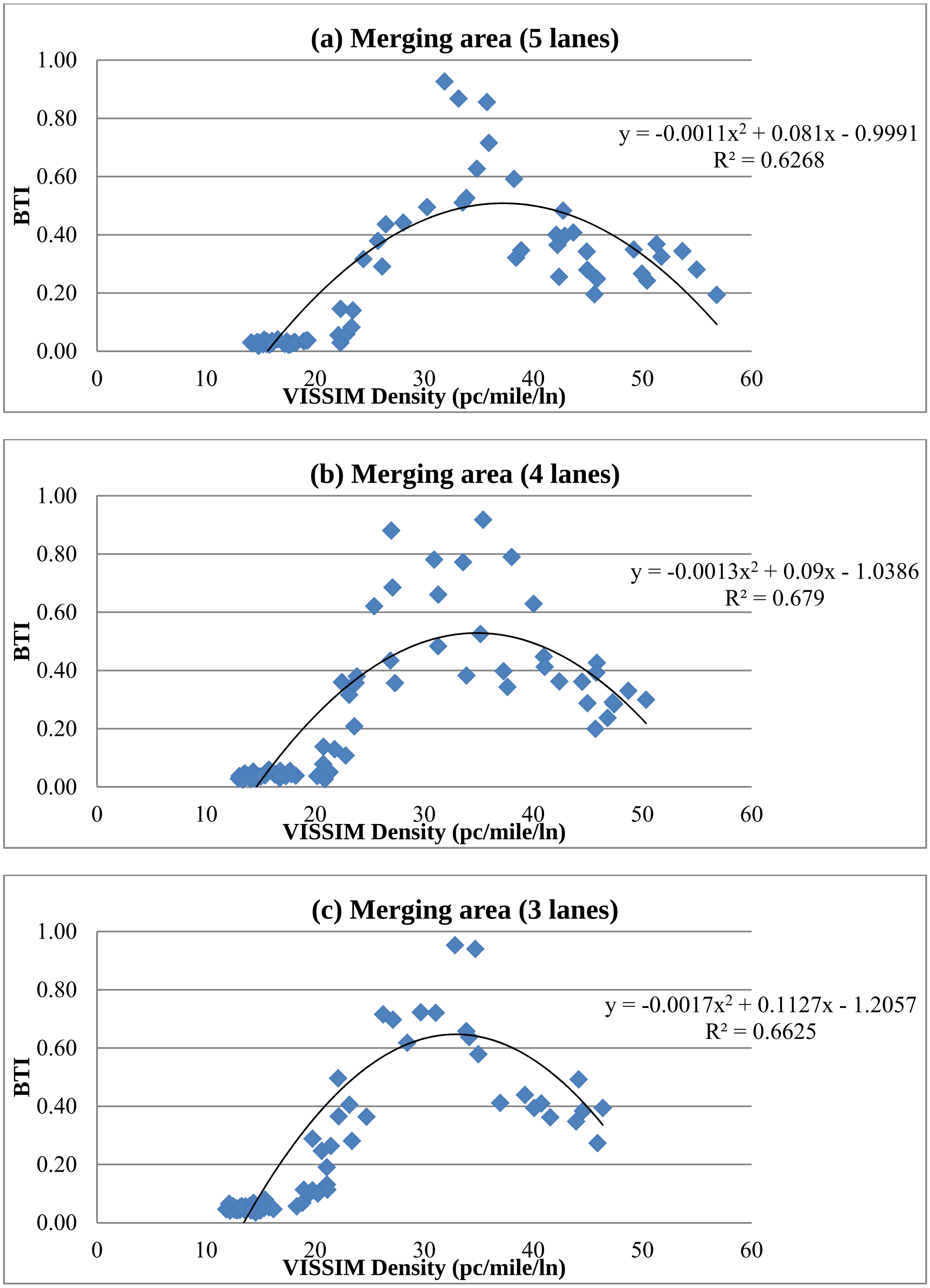


FIGURE 5.57: Density – BTI relationship for 60 mph speed limit on merging area

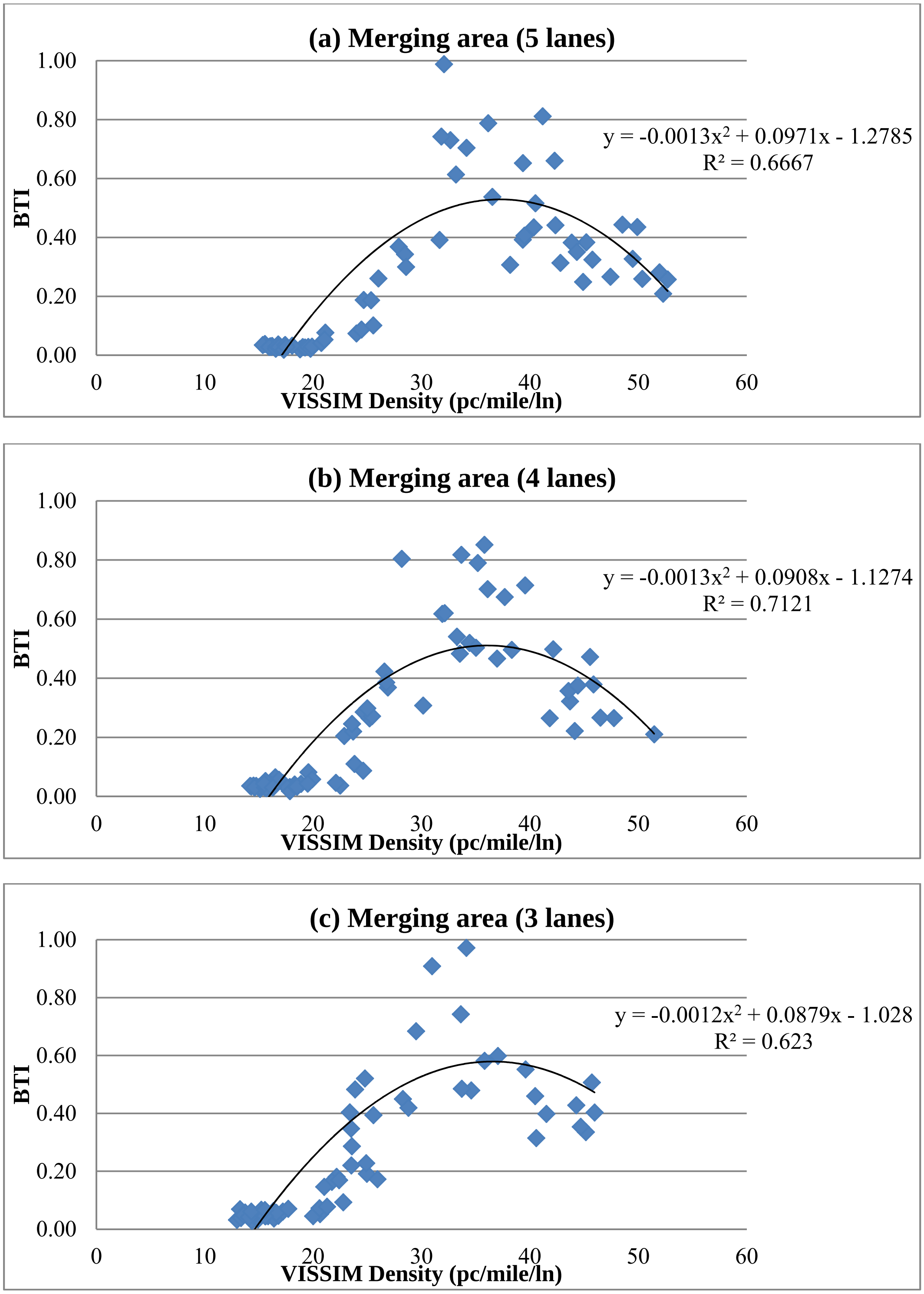


FIGURE 5.58: Density – BTI relationship for 55 mph speed limit on merging area

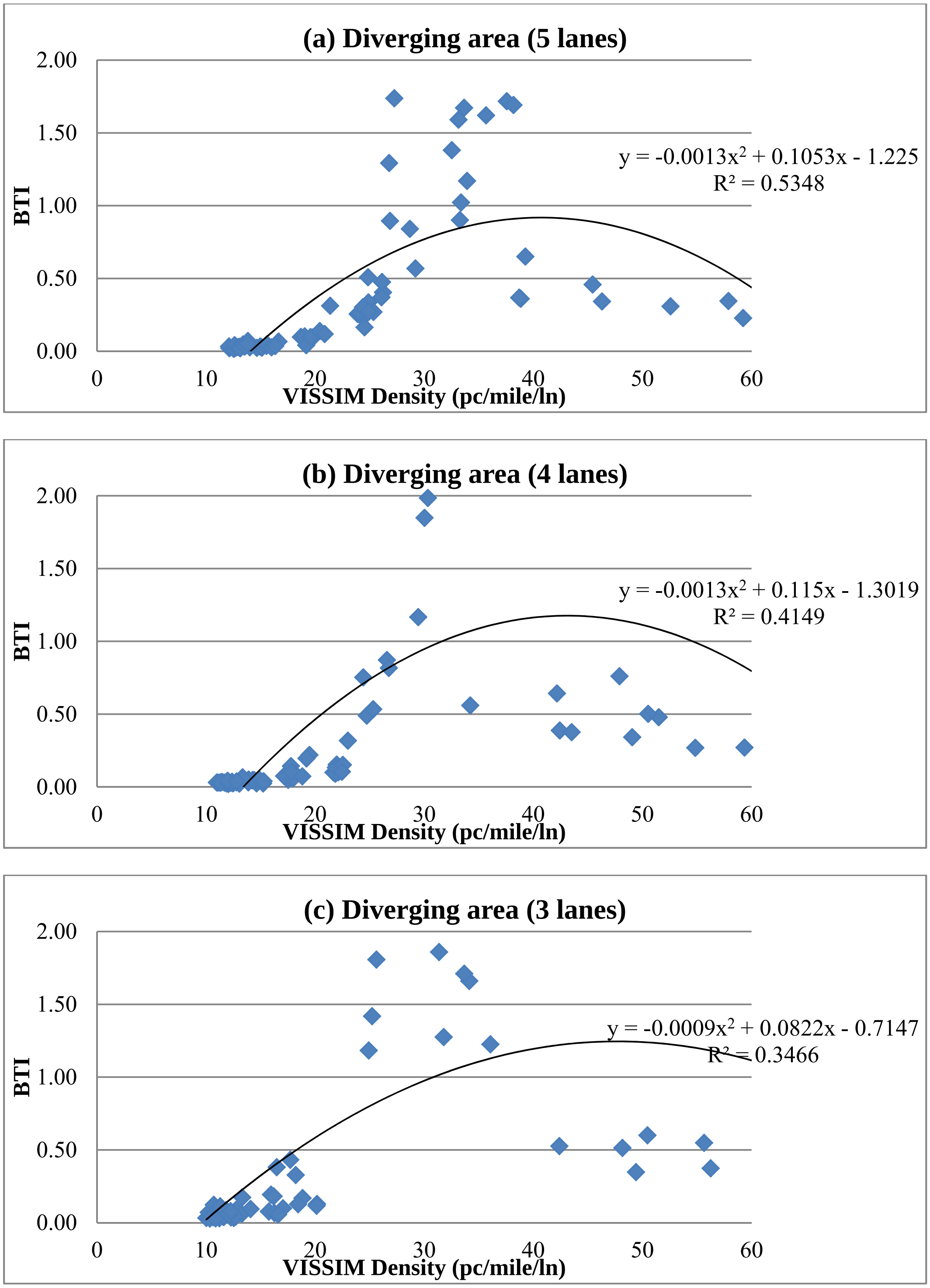


FIGURE 5.59: Density – BTI relationship for 70 mph speed limit on diverging area

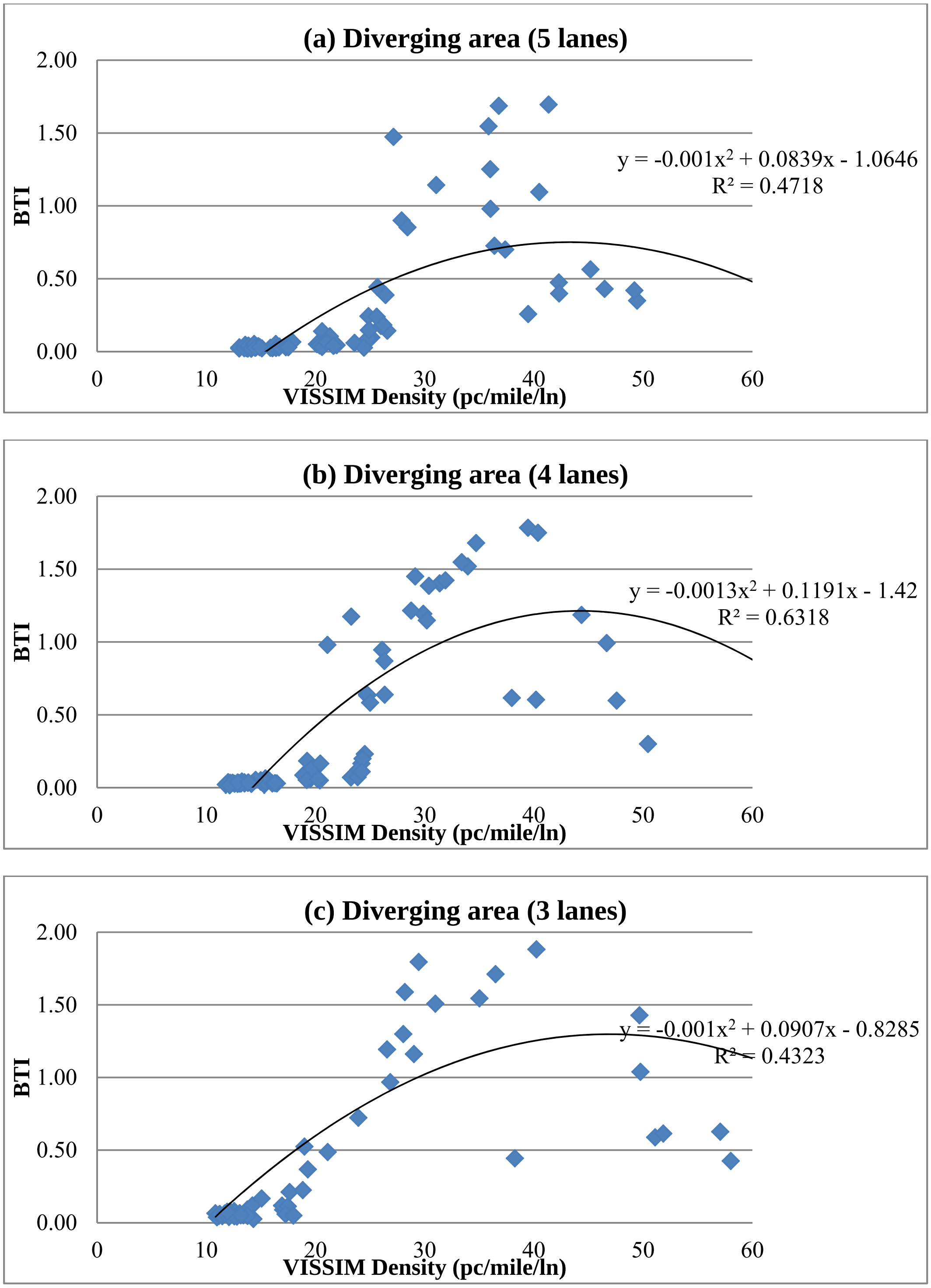


FIGURE 5.60: Density – BTI relationship for 65 mph speed limit on diverging area

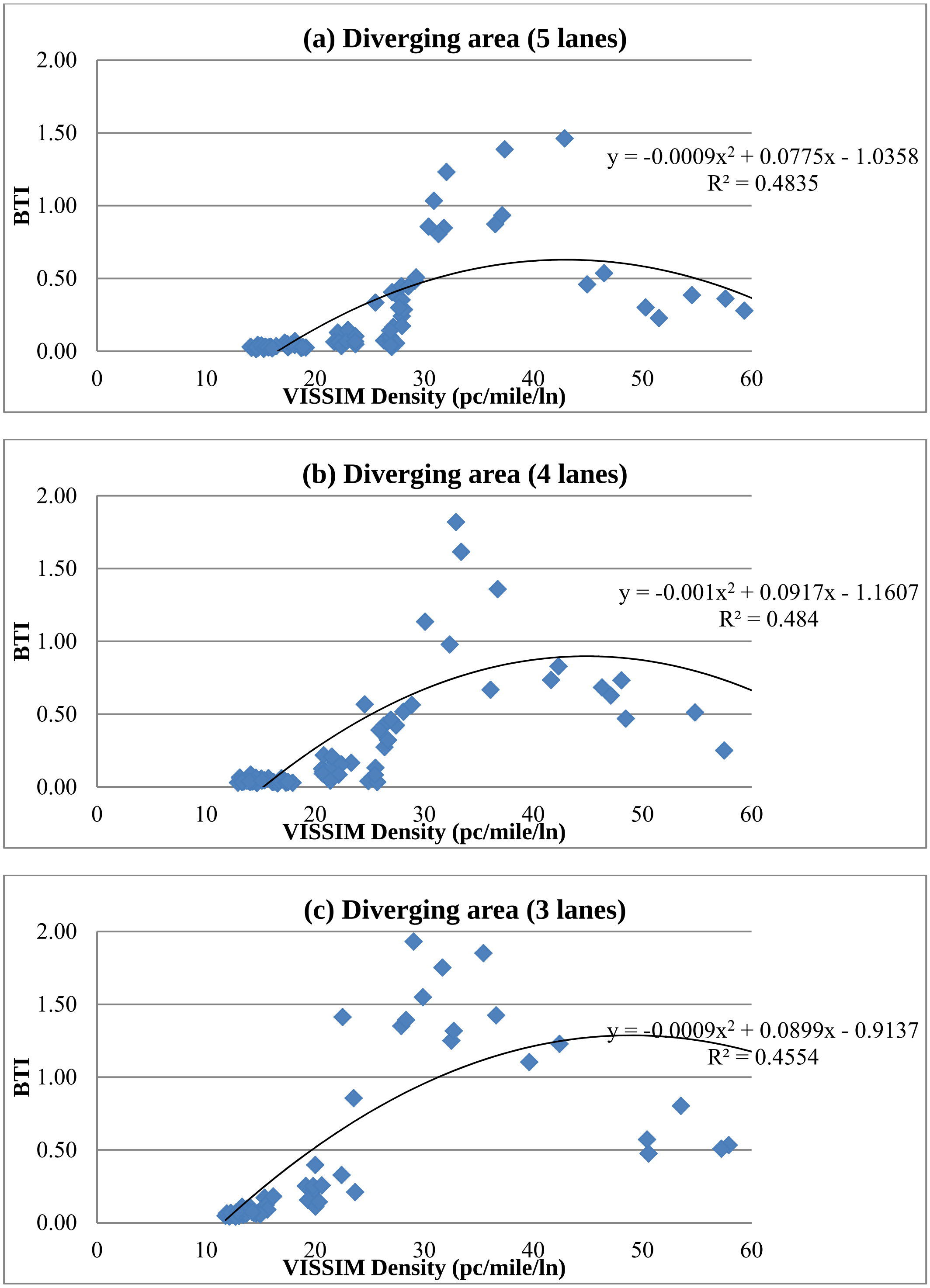


FIGURE 5.61: Density – BTI relationship for 60 mph speed limit on diverging area

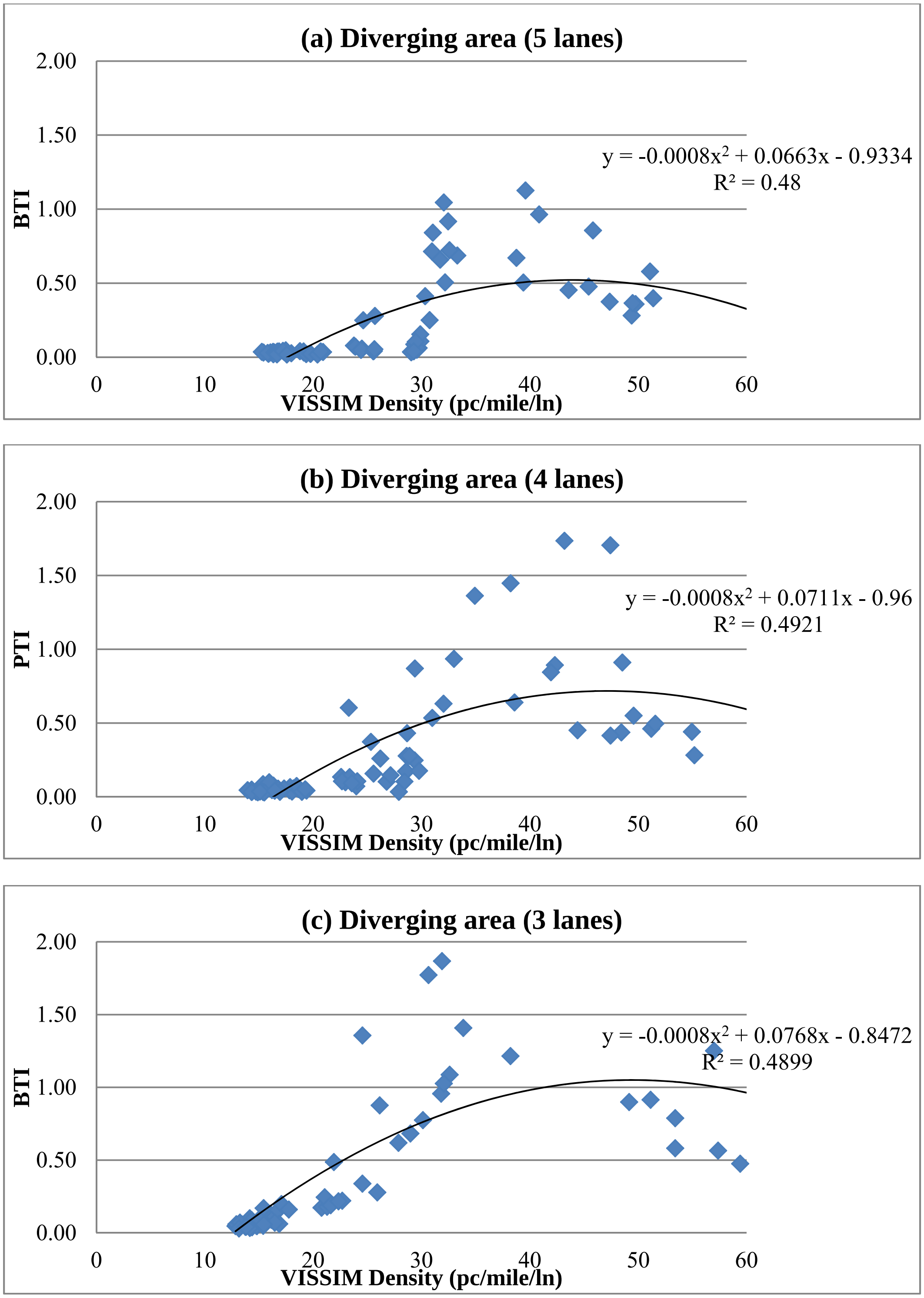


FIGURE 5.62: Density – BTI relationship for 55 mph speed limit on diverging area

5. 6. Travel Time and Travel Time Reliability LOS Thresholds

5.6.1. Travel Time per Mile and Speed based LOS Thresholds

The average travel time per mile and speed threshold values were computed against HCM density thresholds based on the non-linear relationships presented in Sub-section 5.4.1 for a basic freeway section. For example, for the 4-lane basic freeway section, the exponential relationship observed was: $y = 39.2e^{0.0172x}$, where y denotes travel time per mile and x denotes density (pc/mi/ln). Based on the relationship, travel time per mile thresholds were computed based on density threshold values for different LOS criteria. Speed thresholds were computed based on their respective travel time per mile thresholds values. Table 5.4 illustrates the summarized values for this example. Note that the speed threshold for LOS A can exceed the posted speed limit due to extreme low densities and the upper tolerance limit used in the analysis. For those instances, the posted limit can be used as a cut-off point for LOS A.

TABLE 5.4: Travel time per mile and speed thresholds computation illustration

LOS	Density (pc/mi/ln) Thresholds	TT/mile (sec) Thresholds	Speed (mph) Thresholds
A	11	47	76
B	18	53	67
C	25	60	60
D	36	73	49
E	45	85	42
F	> 45	> 85	< 42

Table 5.5 provides average travel time per mile and speed LOS threshold values corresponding to HCM density thresholds. It can be seen that average travel time per mile threshold values for respective LOS letters increase as the speed limit decreases until the condition comes close to a saturation point where the speed limit on the freeway does not

have any influence on the operation. It can also be noted that as the posted limit decreases, the percent difference between the two respective adjacent travel time per mile threshold values also decreases.

TABLE 5.5: LOS thresholds for travel time per mile and speed on basic freeway section

LOS	Density (pc/mi/ln) Thresholds	Travel Time/mile (sec) Thresholds					Speed (mph) Thresholds			
		No. of Lanes								
		4	3	2	Avg.	% Diff	4	3	2	Avg.
		Speed Limit - 70 mph								
A	11	47	47	47	47		76	76	77	76
B	18	53	53	53	53	13%	67	68	68	68
C	26	60	60	60	60	13%	60	60	60	60
D	35	73	72	73	72	21%	49	50	49	50
E	45	85	83	86	85	17%	42	43	42	43
F	> 45 or v/c>1	> 85	> 83	> 86	> 85		< 42	< 43	< 42	< 43
		Speed Limit - 65 mph								
A	11	50	50	50	50		71	71	72	72
B	18	56	56	56	56	11%	64	65	65	65
C	26	62	62	62	62	11%	58	58	58	58
D	35	74	72	74	73	18%	49	50	49	49
E	45	85	82	85	84	15%	43	44	42	43
F	> 45 or v/c>1	> 85	> 82	> 85	> 84		< 43	< 44	< 42	< 43
		Speed Limit - 60 mph								
A	11	55	55	53	54		66	66	67	66
B	18	60	59	59	59	10%	60	61	61	61
C	26	65	65	65	65	10%	55	56	55	55
D	35	75	74	76	75	15%	48	49	47	48
E	45	84	83	87	84	12%	43	44	41	43
F	> 45 or v/c>1	> 84	> 83	> 87	> 84		< 43	< 44	< 41	< 43
		Speed Limit - 55 mph								
A	11	60	61	58	59		60	59	62	61
B	18	64	65	63	64	8%	56	56	57	56
C	26	69	69	69	69	8%	52	52	52	52
D	35	77	77	79	78	12%	46	47	46	46
E	45	85	83	88	86	10%	42	43	41	42
F	> 45 or v/c>1	> 85	> 83	> 88	> 86		< 42	< 43	< 41	< 42

Table 5.6 provides the average travel time per mile and speed threshold values observed from the non-linear relationships as discussed in Sub-section 5.4.2 for the respective HCM density thresholds of a weaving section. As noted for the basic freeway sections, the percent difference between the two respective adjacent travel time per mile threshold values decreases as the posted speed limit decreases.

TABLE 5.6: LOS thresholds for travel time per mile and speed on weaving section

LOS	Density (pc/mi/ln) Thresholds	Travel Time/mile (sec) Thresholds					Speed (mph) Thresholds			
		No. of Lanes								
		5	4	3	Avg.	% Diff	5	4	3	Avg.
		Speed Limit - 70 mph								
A	10	47	49	51	49		76	74	71	74
B	20	57	59	61	59	20%	63	62	59	61
C	28	67	68	70	68	16%	54	53	51	53
D	35	76	77	80	78	14%	47	47	45	46
E	43	89	90	92	90	16%	40	40	39	40
F	> 43 or v/c>1	> 89	> 90	> 92	> 90		< 40	< 40	< 39	< 40
		Speed Limit - 65 mph								
A	10	51	52	55	53		70	69	65	68
B	20	60	60	63	61	16%	60	60	57	59
C	28	69	67	71	69	12%	52	53	51	52
D	35	77	75	78	76	11%	47	48	46	47
E	43	88	83	87	86	13%	41	43	42	42
F	> 43 or v/c>1	> 88	> 83	> 87	> 86		< 41	< 43	< 42	< 42
		Speed Limit - 60 mph								
A	10	57	57	60	58		63	63	60	62
B	20	64	65	67	65	12%	56	55	54	55
C	28	70	72	72	71	10%	51	50	50	51
D	35	76	78	78	77	8%	47	46	46	47
E	43	83	86	84	85	10%	43	42	43	43
F	> 43 or v/c>1	> 83	> 86	> 84	> 85		< 43	< 42	< 43	< 43
		Speed Limit - 55 mph								
A	10	63	63	66	64		57	57	54	56
B	20	68	69	71	70	9%	53	52	50	52
C	28	73	75	76	75	7%	49	48	47	48
D	35	78	81	80	79	6%	46	45	45	45
E	43	83	87	85	85	7%	43	41	42	42
F	> 43 or v/c>1	> 83	> 87	> 85	> 85		< 43	< 41	< 42	< 42

Table 5.7 illustrates the average travel time per mile and speed threshold values in relation to their respective HCM density thresholds observed from the non-linear relationships as discussed in Sub-section 5.4.3 for merging/diverging area on a freeway.

TABLE 5.7: LOS thresholds for travel time per mile and speed on merging/diverging area

LOS	Density (pc/mi/ln) Thresholds	Travel Time/mile (sec) Thresholds					Speed (mph) Thresholds			
		No. of Lanes								
		5	4	3	Avg.	% Diff	5	4	3	Avg.
		Speed Limit - 70 mph								
A	11	49	50	53	51		74	72	68	71
B	18	57	59	62	59	17%	63	61	58	61
C	26	68	71	75	71	20%	53	51	48	51
D	35	84	87	92	88	23%	43	41	39	41
E	43	100	105	110	105	20%	36	34	33	34
F	> 43 or v/c>1	> 100	> 105	> 110	> 105		< 36	< 34	< 33	< 34
		Speed Limit - 65 mph								
A	11	51	52	56	53		70	69	64	68
B	18	59	60	65	62	16%	61	60	55	58
C	26	70	72	78	73	19%	51	50	46	49
D	35	85	86	95	89	21%	42	42	38	41
E	43	101	102	113	105	19%	36	35	32	34
F	> 43 or v/c>1	> 101	> 102	> 113	> 105		< 36	< 35	< 32	< 34
		Speed Limit - 60 mph								
A	11	54	56	59	56		66	65	61	64
B	18	62	64	68	65	15%	58	56	53	56
C	26	72	75	81	76	17%	50	48	45	47
D	35	86	90	97	91	20%	42	40	37	40
E	43	101	106	115	107	17%	36	34	31	34
F	> 43 or v/c>1	> 101	> 106	> 115	> 107		< 36	< 34	< 31	< 34
		Speed Limit - 55 mph								
A	11	59	60	63	60		61	60	57	60
B	18	66	68	72	69	14%	54	53	50	53
C	26	76	78	84	79	16%	47	46	43	45
D	35	89	92	100	94	18%	40	39	36	39
E	43	102	106	117	108	16%	35	34	31	33
F	> 43 or v/c>1	> 102	> 106	> 117	> 108		< 35	< 34	< 31	< 33

Similar to basic freeway and weaving sections, as the posted limit decreases, the percent difference between the two respective adjacent travel time per mile threshold values also decreases. Tables 5.8 and 5.9 show the LOS thresholds for travel time per mile and speed for merging and diverging areas, respectively.

TABLE 5.8: LOS thresholds for travel time per mile and speed on merging area

LOS	Density (pc/mi/ln) Thresholds	Travel Time/mile (sec) Thresholds					Speed (mph) Thresholds			
		No. of Lanes								
		5	4	3	Avg.	% Diff	5	4	3	Avg.
		Speed Limit - 70 mph								
A	11	49	50	52	51		73	72	69	71
B	18	58	59	62	60	16%	62	61	58	60
C	26	69	72	77	73	18%	52	50	47	50
D	35	85	89	97	90	20%	43	41	37	40
E	43	101	107	120	109	18%	36	34	30	33
F	> 43 or v/c>1	>101	>107	>120	>109		<36	<34	<30	<33
		Speed Limit - 65 mph								
A	11	52	53	55	53		69	68	66	68
B	18	60	62	65	62	14%	60	58	56	58
C	26	71	73	78	74	16%	50	49	46	48
D	35	86	89	97	91	18%	42	40	37	40
E	43	102	106	118	109	16%	35	34	31	33
F	> 43 or v/c>1	>102	>106	>118	>109		<35	<34	<31	<33
		Speed Limit - 60 mph								
A	11	55	56	58	57		65	64	62	63
B	18	63	65	68	65	13%	57	56	53	55
C	26	74	76	81	77	15%	49	47	45	47
D	35	88	91	98	92	17%	41	40	37	39
E	43	102	106	117	108	15%	35	34	31	33
F	> 43 or v/c>1	>102	>106	>117	>108		<35	<34	<31	<33
		Speed Limit - 55 mph								
A	11	60	61	62	61		60	59	58	59
B	18	68	69	71	69	12%	53	52	50	52
C	26	77	79	83	80	13%	47	46	43	45
D	35	90	92	99	94	15%	40	39	36	39
E	43	103	105	115	108	13%	35	34	31	34
F	> 43 or v/c>1	>103	>105	>115	>108		<35	<34	<31	<34

TABLE 5.9: LOS thresholds for travel time per mile and speed on diverging area

LOS	Density (pc/mi/ln) Thresholds	Travel Time/mile (sec) Thresholds					Speed (mph) Thresholds			
		No. of Lanes								
		5	4	3	Avg.	% Diff	5	4	3	Avg.
		Speed Limit - 70 mph								
A	11	48	50	53	50		74	73	67	71
B	18	57	58	62	59	15%	64	62	58	61
C	26	68	70	75	71	17%	53	51	48	51
D	35	83	86	91	87	18%	44	42	40	42
E	43	99	104	109	104	17%	36	35	33	35
F	> 43 or v/c>1	>99	>104	>109	>104		<36	<35	<33	<35
		Speed Limit - 65 mph								
A	11	50	52	56	53		72	69	64	68
B	18	58	61	66	62	14%	62	59	55	59
C	26	69	72	78	73	16%	52	50	46	49
D	35	84	87	95	89	18%	43	41	38	41
E	43	100	104	113	105	16%	36	35	32	34
F	> 43 or v/c>1	>100	>104	>113	>105		<36	<35	<32	<34
		Speed Limit - 60 mph								
A	11	53	55	60	56		68	65	60	64
B	18	61	64	69	65	13%	59	57	52	56
C	26	71	75	81	76	15%	50	48	44	48
D	35	85	90	97	91	17%	42	40	37	40
E	43	99	106	115	106	15%	36	34	31	34
F	> 43 or v/c>1	>99	>106	>115	>106		<36	<34	<31	<34
		Speed Limit - 55 mph								
A	11	57	59	63	60		63	61	57	60
B	18	65	67	72	68	12%	56	54	50	53
C	26	75	78	84	79	14%	48	46	43	46
D	35	88	92	100	93	15%	41	39	36	39
E	43	101	107	117	108	14%	35	34	31	33
F	> 43 or v/c>1	>101	>107	>117	>108		<35	<34	<31	<33

5.6.2. Travel Time Reliability LOS Thresholds

Table 5.10 shows travel time reliability LOS thresholds corresponding to HCM density thresholds observed from the polynomial relationships discussed in Sub-sub-sections 5.5.1.1 and 5.5.2.1 for PTI and BTI on a basic freeway section, respectively. It can be noted that the average reliability threshold values for their respective LOS letters decrease as the posted speed limit decreases. For PTI based thresholds, the 95th percentile travel time decreases as the speed limit drops but the 5th percentile travel times remain relatively similar. In case of BTI based thresholds, the 95th and 50th percentile travel time values become closer as the speed limit decreases. Furthermore, the percent difference between two respective adjacent PTI threshold values remains relatively similar to slight increase or decrease as the speed limit decreases. However, as the speed limit decreases, the percent difference between two respective adjacent BTI threshold values increases.

Table 5.11 provides the PTI and BTI LOS thresholds observed from the non-linear relationships discussed in Sub-sub-sections 5.5.1.2 and 5.5.2.2 for the respective HCM density thresholds for a weaving section, respectively. As the speed limit decreases, the average thresholds values for their respective LOS letters also decrease. The percent difference between the two respective adjacent PTI threshold values remains similar to slight increase or decrease as the speed limit decreases, while for BTI, the percent difference tends to increase.

Table 5.12 illustrates PTI and BTI LOS threshold values in relation to their respective HCM density thresholds observed from the polynomial relationships discussed in Sub-sub-sections 5.5.1.3 and 5.5.2.3 for merging/diverging areas on a freeway, respectively. As the speed limit goes down at the merging/diverging area, PTI LOS thresholds for their respective LOS letters also tend to go down. However, for BTI, the values do not follow any particular pattern. The percent difference between the two respective adjacent PTI threshold values remain similar as the speed limit decreases, while for BTI, the trend follows what was observed for that of basic freeway and weaving sections. Tables 5.13 and 5.14 further illustrate the reliability LOS thresholds for merging and diverging areas, respectively.

TABLE 5.10: LOS thresholds for PTI and BTI on basic freeway section

LOS	Density (pc/mi/ln) Thresholds	PTI Thresholds					BTI Thresholds				
		No. of Lanes									
		4	3	2	Avg.	% Diff	4	3	2	Avg.	% Diff
		Speed Limit - 70 mph									
A	11	0.80	0.79	0.73	0.78		-0.08	-0.11	-0.14	-0.11	
B	18	1.07	1.08	1.05	1.07	27%	0.05	0.06	0.05	0.05	313 %
C	26	1.27	1.29	1.31	1.29	17%	0.13	0.18	0.19	0.17	70%
D	35	1.44	1.49	1.60	1.51	15%	0.19	0.27	0.34	0.27	37%
E	45	1.45	1.53	1.74	1.57	4%	0.16	0.25	0.39	0.27	1%
F	> 45 or v/c<1	< 1.45	< 1.53	< 1.74	< 1.57		< 0.16	< 0.25	< 0.39	< 0.27	
		Speed Limit - 65 mph									
A	11	0.77	0.79	0.74	0.77		-0.17	-0.15	-0.21	-0.18	
B	18	1.03	1.02	1.02	1.03	25%	0.00	0.02	0.01	0.01	1676 %
C	26	1.23	1.19	1.25	1.23	16%	0.11	0.15	0.18	0.15	92%
D	35	1.43	1.34	1.51	1.43	14%	0.17	0.24	0.35	0.25	42%
E	45	1.49	1.36	1.63	1.49	4%	0.11	0.22	0.39	0.24	-4%
F	> 45 or v/c<1	< 1.49	< 1.36	< 1.63	< 1.49		< 0.11	< 0.22	< 0.39	< 0.24	
		Speed Limit - 60 mph									
A	11	0.96	0.82	0.75	0.84		-0.09	-0.16	-0.22	-0.16	
B	18	1.03	0.99	0.98	1.00	16%	0.01	-0.02	-0.03	-0.01	1383 %
C	26	1.13	1.14	1.19	1.15	13%	0.10	0.09	0.13	0.11	110 %
D	35	1.35	1.31	1.45	1.37	16%	0.22	0.18	0.31	0.23	54%
E	45	1.58	1.40	1.61	1.53	10%	0.30	0.18	0.37	0.28	17%
F	> 45 or v/c<1	< 1.58	< 1.40	< 1.61	< 1.53		< 0.30	< 0.18	< 0.37	< 0.28	
		Speed Limit - 55 mph									
A	11	1.04	0.95	0.77	0.92		-0.10	-0.14	-0.23	-0.16	
B	18	1.03	1.01	0.95	0.99	8%	-0.02	-0.02	-0.06	-0.03	378 %
C	26	1.07	1.07	1.11	1.08	8%	0.05	0.09	0.07	0.07	145 %
D	35	1.24	1.19	1.32	1.25	13%	0.15	0.23	0.23	0.20	64%
E	45	1.46	1.31	1.46	1.41	11%	0.22	0.30	0.30	0.27	26%
F	> 45 or v/c<1	< 1.46	< 1.31	< 1.46	< 1.41		< 0.22	< 0.30	< 0.30	< 0.27	

TABLE 5.11: LOS thresholds for PTI and BTI on weaving section

LOS	Density (pc/mi/ln) Thresholds	PTI Thresholds					BTI Thresholds				
		No. of Lanes									
		5	4	3	Avg.	% Diff	5	4	3	Avg.	% Diff
		Speed Limit - 70 mph									
A	10	0.85	0.92	1.01	0.93		-0.14	-0.09	-0.04	-0.09	
B	20	1.21	1.26	1.29	1.25	26%	0.18	0.20	0.21	0.20	144 %
C	28	1.41	1.49	1.67	1.53	18%	0.31	0.32	0.34	0.32	38%
D	35	1.53	1.68	2.10	1.77	14%	0.32	0.34	0.40	0.36	9%
E	43	1.59	1.87	2.71	2.06	14%	0.24	0.26	0.41	0.30	-17%
F	> 43 or v/c>1	< 1.59	< 1.87	< 2.71	< 2.06		< 0.24	< 0.26	< 2.41	< 0.30	
		Speed Limit - 65 mph									
A	10	0.79	0.90	1.09	0.93		-0.18	-0.11	0.01	-0.09	
B	20	1.22	1.20	1.18	1.20	23%	0.17	0.16	0.16	0.16	157 %
C	28	1.45	1.41	1.65	1.50	20%	0.32	0.28	0.42	0.34	52%
D	35	1.57	1.57	2.34	1.83	18%	0.36	0.31	0.75	0.47	28%
E	43	1.61	1.73	3.45	2.26	19%	0.29	0.25	1.25	0.60	21%
F	> 43 or v/c>1	< 1.61	< 1.73	< 3.45	< 2.26		< 0.29	< 0.25	< 1.25	< 0.60	
		Speed Limit - 60 mph									
A	10	0.99	1.04	1.15	1.06		-0.10	-0.05	0.05	-0.03	
B	20	1.10	1.08	1.10	1.09	3%	0.08	0.09	0.07	0.08	140 %
C	28	1.32	1.27	1.45	1.34	19%	0.22	0.21	0.27	0.23	66%
D	35	1.60	1.56	2.03	1.73	22%	0.34	0.33	0.58	0.42	44%
E	43	2.03	2.01	3.03	2.36	27%	0.48	0.47	1.09	0.68	39%
F	> 43 or v/c>1	< 2.03	< 2.01	< 3.03	< 2.36		< 0.48	< 0.47	< 1.09	< 0.68	
		Speed Limit - 55 mph									
A	10	1.05	1.00	1.17	1.07		-0.08	-0.11	0.08	-0.04	
B	20	1.04	1.08	1.09	1.07	0%	0.04	0.07	0.04	0.05	172 %
C	28	1.15	1.24	1.32	1.24	14%	0.15	0.18	0.18	0.17	71%
D	35	1.35	1.46	1.72	1.51	18%	0.26	0.26	0.44	0.32	46%
E	43	1.69	1.78	2.43	1.97	23%	0.39	0.33	0.87	0.53	40%
F	> 43 or v/c>1	< 1.69	< 1.78	< 2.43	< 1.97		< 0.39	< 0.33	< 0.87	< 0.53	

TABLE 5.12: LOS thresholds for PTI and BTI on merging/diverging area

LOS	Density (pc/mi/ln) Thresholds	PTI Thresholds					BTI Thresholds				
		No. of Lanes									
		5	4	3	Avg.	% Diff	5	4	3	Avg.	% Diff
		Speed Limit - 70 mph									
A	11	0.71	0.77	0.86	0.78		-0.15	-0.07	0.07	-0.05	
B	18	1.34	1.41	1.66	1.47	47%	0.24	0.29	0.38	0.30	117 %
C	26	1.89	1.97	2.41	2.09	30%	0.53	0.60	0.65	0.60	49%
D	35	2.28	2.38	3.05	2.57	19%	0.69	0.81	0.84	0.78	24%
E	43	2.41	2.56	3.45	2.81	8%	0.66	0.88	0.92	0.82	5%
F	> 43 or v/c>1	< 2.41	< 2.56	< 3.45	< 2.81		< 0.66	< 0.88	< 0.92	< 0.82	
		Speed Limit - 65 mph									
A	10	0.62	0.69	0.77	0.69		-0.19	-0.14	0.03	-0.10	
B	20	1.19	1.33	1.56	1.36	49%	0.14	0.25	0.38	0.26	139 %
C	28	1.69	1.88	2.32	1.96	31%	0.40	0.57	0.69	0.56	54%
D	35	2.05	2.29	2.96	2.44	19%	0.54	0.78	0.94	0.76	26%
E	43	2.20	2.47	3.36	2.67	9%	0.53	0.83	1.07	0.81	7%
F	> 43 or v/c>1	< 2.20	< 2.47	< 3.36	< 2.67		< 0.53	< 0.83	< 1.07	< 0.81	
		Speed Limit - 60 mph									
A	10	0.68	0.63	0.64	0.65		-0.24	-0.17	-0.05	-0.15	
B	20	1.10	1.19	1.41	1.23	47%	0.09	0.14	0.30	0.18	187 %
C	28	1.51	1.71	2.13	1.79	31%	0.36	0.40	0.62	0.46	61%
D	35	1.87	2.15	2.77	2.26	21%	0.53	0.56	0.87	0.65	30%
E	43	2.12	2.40	3.17	2.56	12%	0.56	0.60	0.99	0.72	9%
F	> 43 or v/c>1	< 2.12	< 2.40	< 3.17	< 2.56		< 0.56	< 0.60	< 0.99	< 0.72	
		Speed Limit - 55 mph									
A	10	0.61	0.60	0.56	0.59		-0.29	-0.20	-0.10	-0.20	
B	20	1.02	1.08	1.22	1.10	46%	0.02	0.10	0.21	0.11	278 %
C	28	1.40	1.53	1.83	1.59	30%	0.27	0.37	0.50	0.38	71%
D	35	1.74	1.92	2.35	2.01	21%	0.42	0.58	0.73	0.58	34%
E	43	1.97	2.16	2.67	2.26	11%	0.43	0.69	0.86	0.66	12%
F	> 43 or v/c>1	< 1.97	< 2.16	< 2.67	< 2.26		< 0.43	< 0.69	< 0.86	< 0.66	

TABLE 5.13: LOS thresholds for PTI and BTI on merging area

LOS	Density (pc/mi/ln) Thresholds	PTI Thresholds					BTI Thresholds				
		No. of Lanes									
		5	4	3	Avg.	% Diff	5	4	3	Avg.	% Diff
		Speed Limit - 70 mph									
A	11	0.74	0.83	0.90	0.82		-0.11	-0.08	-0.07	-0.09	
B	18	1.37	1.44	1.62	1.48	44%	0.24	0.26	0.43	0.31	128 %
C	26	1.88	1.92	2.16	1.99	26%	0.51	0.49	0.74	0.58	46%
D	35	2.19	2.18	2.40	2.26	12%	0.65	0.53	0.75	0.64	10%
E	43	2.24	2.18	2.29	2.23	-1%	0.62	0.37	0.46	0.48	-33%
F	> 43 or v/c>1	< 2.24	< 2.18	< 2.29	< 2.23		< 0.62	< 0.37	< 0.46	< 0.48	
		Speed Limit - 65 mph									
A	10	0.67	0.75	0.84	0.75		-0.18	-0.15	-0.12	-0.15	
B	20	1.25	1.31	1.51	1.36	45%	0.18	0.24	0.35	0.26	158 %
C	28	1.75	1.77	2.03	1.85	27%	0.44	0.51	0.65	0.53	52%
D	35	2.07	2.06	2.31	2.15	14%	0.55	0.60	0.68	0.61	13%
E	43	2.16	2.11	2.28	2.18	2%	0.49	0.49	0.44	0.47	-29%
F	> 43 or v/c>1	< 2.16	< 2.11	< 2.28			< 0.49	< 0.49	< 0.44	< 0.47	
		Speed Limit - 60 mph									
A	10	0.65	0.75	0.85	0.75		-0.24	-0.21	-0.17	-0.21	
B	20	1.13	1.21	1.34	1.22	39%	0.10	0.16	0.27	0.18	216 %
C	28	1.55	1.62	1.80	1.66	26%	0.36	0.42	0.58	0.45	61%
D	35	1.85	1.95	2.19	2.00	17%	0.49	0.52	0.66	0.55	18%
E	43	1.97	2.12	2.44	2.17	8%	0.45	0.43	0.50	0.46	-21%
F	> 43 or v/c>1	< 1.97	< 2.12	< 2.44	< 2.17		< 0.45	< 0.43	< 0.50	< 0.46	
		Speed Limit - 55 mph									
A	10	0.60	0.76	0.82	0.73		-0.37	-0.29	-0.21	-0.29	
B	20	1.06	1.13	1.21	1.13	36%	0.05	0.09	0.17	0.10	387 %
C	28	1.48	1.51	1.59	1.52	26%	0.37	0.35	0.45	0.39	74%
D	35	1.81	1.85	1.94	1.87	18%	0.53	0.46	0.58	0.52	25%
E	43	1.98	2.08	2.19	2.08	10%	0.49	0.37	0.53	0.47	-12%
F	> 43 or v/c>1	< 1.98	< 2.08	< 2.19	< 2.08		< 0.49	< 0.37	< 0.53	< 0.47	

TABLE 5.13: LOS thresholds for PTI and BTI on diverging area

LOS	Density (pc/mi/ln) Thresholds	PTI Thresholds					BTI Thresholds				
		No. of Lanes									
		5	4	3	Avg.	% Diff	5	4	3	Avg.	% Diff
		Speed Limit - 70 mph									
A	11	0.57	0.58	0.74	0.63		-0.22	-0.19	0.08	-0.11	
B	18	1.32	1.48	1.90	1.57	60%	0.25	0.35	0.47	0.36	132 %
C	26	1.99	2.27	2.98	2.41	35%	0.63	0.81	0.81	0.75	53%
D	35	2.49	2.87	3.87	3.08	22%	0.87	1.13	1.06	1.02	26%
E	43	2.72	3.15	4.37	3.41	10%	0.90	1.24	1.16	1.10	7%
F	> 43 or v/c>1	< 2.72	< 3.15	< 4.37	< 3.41		< 0.90	< 1.24	< 1.16	< 1.10	
		Speed Limit - 65 mph									
A	10	0.51	0.55	0.66	0.57		-0.26	-0.27	0.05	-0.16	
B	20	1.16	1.36	1.76	1.43	60%	0.12	0.30	0.48	0.30	153 %
C	28	1.74	2.09	2.79	2.21	35%	0.44	0.80	0.85	0.70	57%
D	35	2.20	2.65	3.63	2.83	22%	0.65	1.16	1.12	0.97	28%
E	43	2.44	2.91	4.11	3.15	10%	0.69	1.30	1.22	1.07	9%
F	> 43 or v/c>1	< 2.44	< 2.91	< 4.11	< 3.15		< 0.69	< 1.30	< 1.22	< 1.07	
		Speed Limit - 60 mph									
A	10	0.63	0.42	0.53	0.53		-0.29	-0.27	-0.03	-0.20	
B	20	1.06	1.16	1.55	1.26	58%	0.07	0.17	0.41	0.22	193 %
C	28	1.49	1.82	2.50	1.94	35%	0.37	0.55	0.82	0.58	63%
D	35	1.89	2.34	3.28	2.51	23%	0.57	0.82	1.13	0.84	31%
E	43	2.19	2.60	3.74	2.84	12%	0.63	0.93	1.29	0.95	11%
F	> 43 or v/c>1	< 2.19	< 2.60	< 3.74	< 2.84		< 0.63	< 0.93	< 1.29	< 0.95	
		Speed Limit - 55 mph									
A	10	0.55	0.43	0.40	0.46		-0.30	-0.27	-0.10	-0.22	
B	20	0.97	1.05	1.34	1.12	59%	0.00	0.06	0.28	0.11	300 %
C	28	1.36	1.62	2.22	1.73	36%	0.25	0.35	0.61	0.40	72%
D	35	1.72	2.10	2.96	2.26	23%	0.41	0.55	0.86	0.61	34%
E	43	1.96	2.37	3.40	2.58	12%	0.44	0.62	0.98	0.68	11%
F	> 43 or v/c>1	< 1.96	< 2.37	< 3.40	< 2.58		< 0.44	< 0.62	< 0.98	< 0.68	

5.7. Implementation of Travel Time Reliability LOS

5.7.1. Case Study Description

I-10 is a heavily traveled freeway that traverses East-West of the city of Jacksonville, FL. Eastbound direction is the peak hour direction during the AM peak hour and westbound is the peak hour direction during the PM peak hour. For the case study purpose, one segment on I-10 Eastbound near the I-295 and I-10 interchange has been selected (Figure 5.63). TMC 102+04871 was used to illustrate the segment LOSs obtained using the travel time reliability and a composite LOS for a time window. Table 5.15 illustrates details of the section.

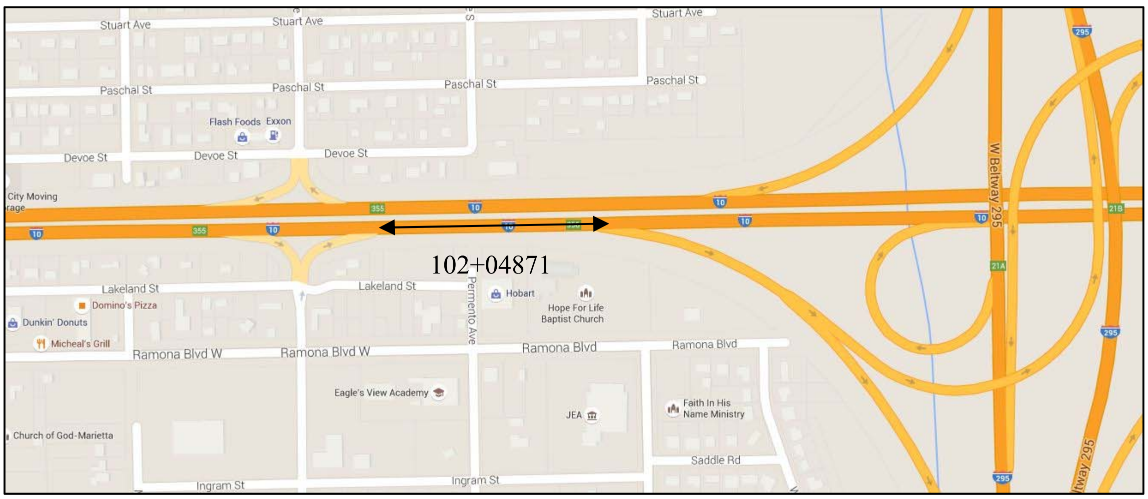


FIGURE 5.63: Case study location for travel time reliability LOS analysis

TABLE 5.15: Section for travel time reliability LOS analysis

TMC Code	Segment Type	No. of Lanes	Length (mile)
102+04871	Weave	4	0.11

5.7.2. Section Travel Time Reliability LOS

INRIX provided 5-min average travel time for TMC 102+04871 for the year of 2012 and 2013. July 1, 2013 (Monday) was selected as the analysis date and 7:00 AM to 8:00 AM as the analysis time window. For each five minutes of this hour, 95th, 50th, and 5th percentile of the average travel times were computed for four different observation periods (18, 12, 6, and 3 months). Further, analysis was performed by segregating the data into three different observation criteria groups (all days, weekdays only, and Mondays only). Both PTI and BTI were computed using Equations 3.1 and 3.2, respectively. By comparing the values with PTI and BTI based LOS thresholds, LOS letters were denoted to each of these 5-min time periods.

5.7.2.1. Planning Time Index based LOS

Table 5.16 shows the LOS predictions for July 1, 2013 based on 18, 12, 6, and 3 months of data. Table 5.17 shows the analysis of variance (ANOVA) results on the observation periods. Null hypothesis is all means are equal, while the alternative hypothesis is at least one mean is different (significance level, $\alpha = 0.01$). The F-value shows that the null hypothesis is rejected and at least one combination has different mean. Tukey HSD test (Table 5.18) shows that the mean is similar between 18 and 12 months, and 6 and 3 months observation periods, and a difference in the mean exists for all other combinations for this exercise. Therefore, the observation period is a significant factor for the computation of the PTI.

TABLE 5.16: PTI values and their respective LOSs

	18 months						12 months					
	All Days		Weekdays		Mondays		All Days		Weekdays		Mondays	
	PTI	LOS	PTI	LOS	PTI	LOS	PTI	LOS	PTI	LOS	PTI	LOS
7:00:00	1.30	C	1.18	B	1.18	B	1.30	C	1.32	C	1.24	C
7:05:00	1.30	C	1.18	B	1.38	C	1.30	C	1.18	B	1.23	C
7:10:00	1.30	C	1.18	B	1.18	B	1.30	C	1.30	C	1.18	B
7:15:00	1.30	C	1.27	C	1.49	C	1.30	C	1.32	C	1.32	C
7:20:00	1.40	C	1.52	D	1.82	D	1.40	C	1.62	D	1.50	C
7:25:00	1.45	C	1.88	E	1.91	E	1.50	C	1.92	E	1.86	E
7:30:00	1.80	D	2.09	E	2.36	F	1.79	D	2.20	E	2.64	F
7:35:00	1.70	D	2.00	E	2.98	F	1.70	D	2.12	E	2.68	F
7:40:00	1.79	D	1.82	D	2.24	E	1.80	D	1.92	E	1.86	E
7:45:00	1.79	D	1.80	D	1.95	E	1.79	D	2.02	E	2.10	E
7:50:00	1.60	D	1.64	D	1.84	E	1.70	D	2.00	E	2.15	E
7:55:00	1.61	D	1.77	D	1.59	D	1.60	D	1.84	E	2.60	F
	6 months						3 months					
	All Days		Weekdays		All Days		Weekdays		All Days		Weekdays	
	PTI	LOS	PTI	LOS	PTI	LOS	PTI	LOS	PTI	LOS	PTI	LOS
7:00:00	1.20	B	1.20	B	1.18	B	1.20	B	1.20	B	1.18	B
7:05:00	1.30	C	1.18	B	1.18	B	1.30	C	1.18	B	1.18	B
7:10:00	1.30	C	1.30	C	1.17	B	1.30	C	1.30	C	1.17	B
7:15:00	1.30	C	1.30	C	1.28	C	1.30	C	1.30	C	1.28	C
7:20:00	1.30	C	1.30	C	1.28	C	1.30	C	1.30	C	1.28	C
7:25:00	1.30	C	1.18	B	1.26	C	1.30	C	1.18	B	1.26	C
7:30:00	1.30	C	1.26	C	1.27	C	1.30	C	1.26	C	1.27	C
7:35:00	1.30	C	1.30	C	1.53	D	1.30	C	1.30	C	1.53	D
7:40:00	1.30	C	1.37	C	1.26	C	1.30	C	1.37	C	1.26	C
7:45:00	1.30	C	1.30	C	1.28	C	1.30	C	1.30	C	1.28	C
7:50:00	1.40	C	1.40	C	1.66	D	1.40	C	1.40	C	1.66	D
7:55:00	1.30	C	1.30	C	1.55	D	1.30	C	1.30	C	1.55	D

TABLE 5.17: ANOVA results on observation periods – PTI

Factor Information					
Factor	Levels	Values			
Factor	4	18 months, 12 months, 6 months, 3 months			
Analysis of Variance					
Source	DF	Adj. SS	Adj. MS	F-Value	P-Value
Factor	3	5.278	1.75936	20.1	0
Error	140	12.251	0.08751		
Total	143	17.529			

TABLE 5.18: Tukey HSD test on observation periods – PTI

	12 months	6 months	3 months
18 months	n/s	P<0.01	P<0.01
12 months	-	P<0.01	P<0.01
6 months	-	-	n/s

n/s= non-significant

Figure 5.64 shows the variation of PTI for different observation periods. The significance of observation period can also be identified. Figure 5.65 illustrates the PTI variations for different observation criteria. It can be noted that the weekdays and Mondays only PTI values were higher for the 18 and 12 months of observation periods. However, for 6 and 3 months of observation periods, the values are lower, which also shows the importance of observation period in the PTI analysis.

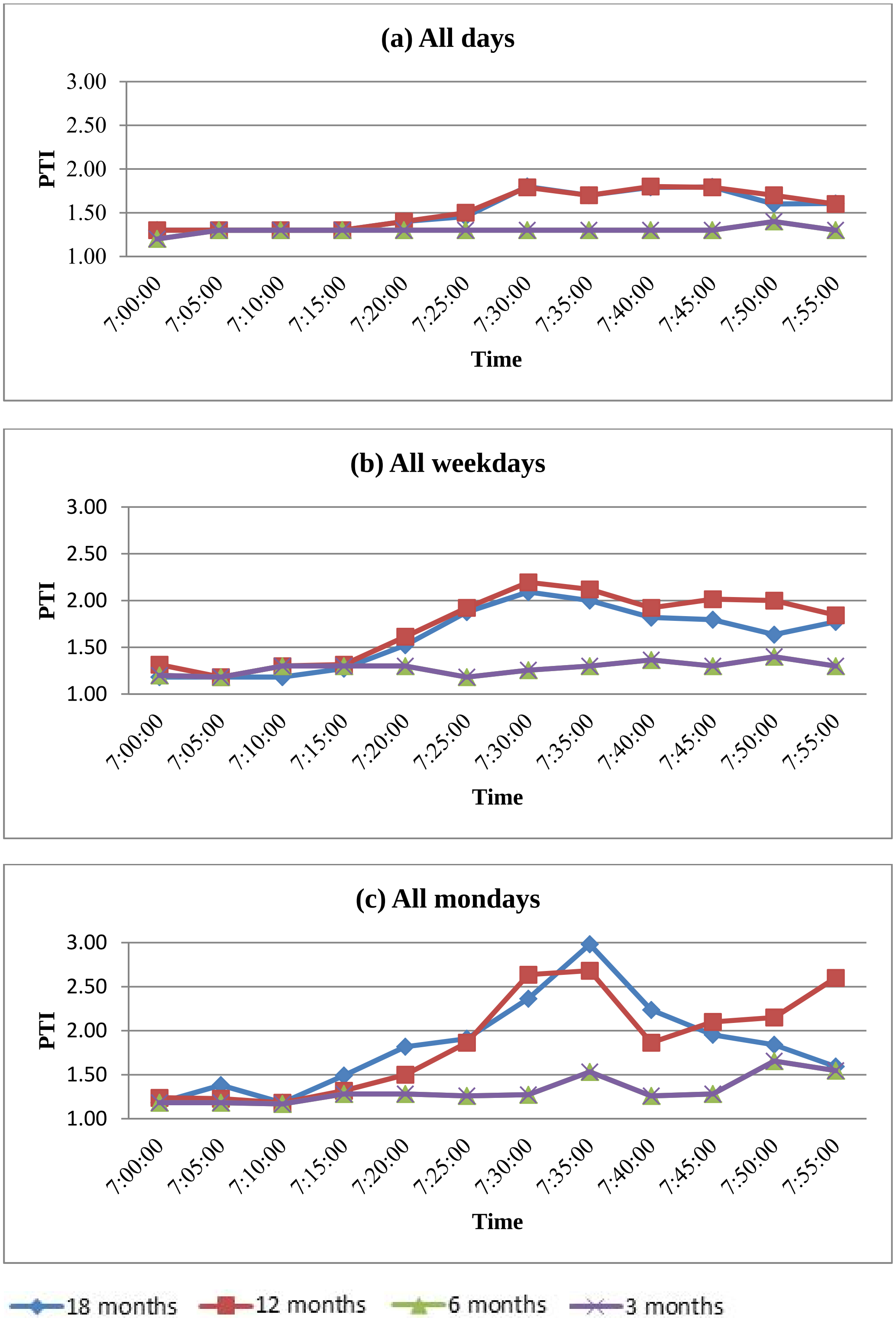
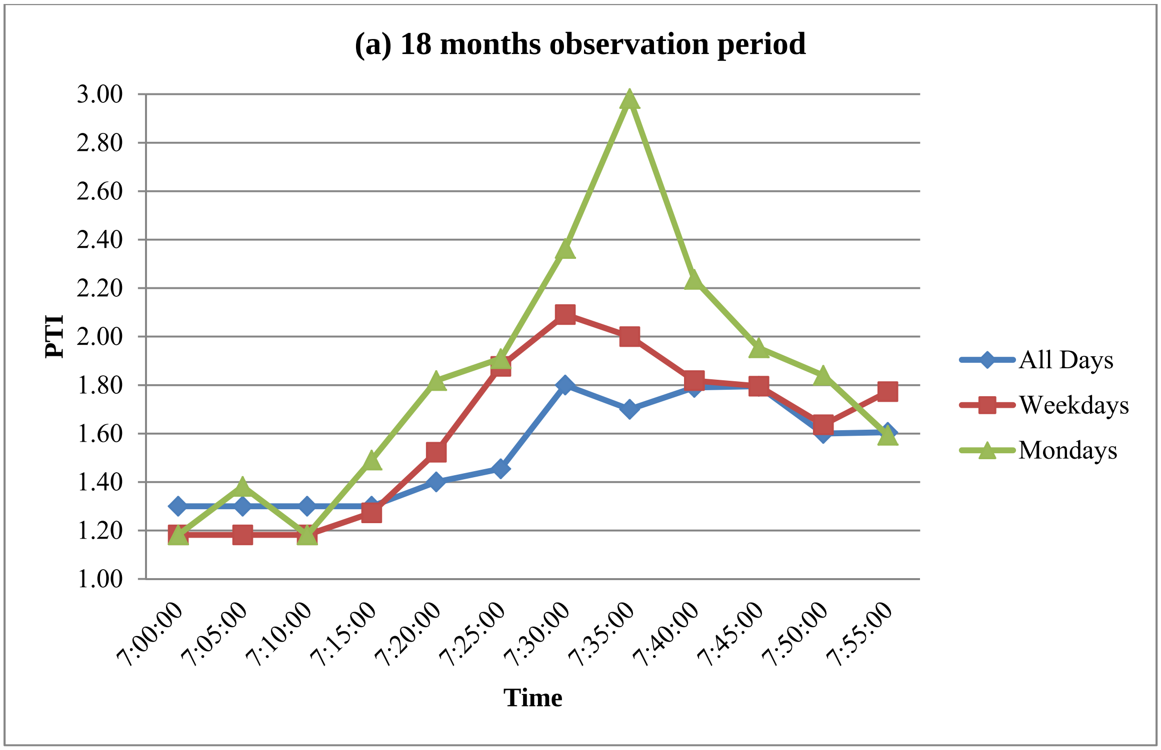


FIGURE 5.64: PTI variations for different observation periods



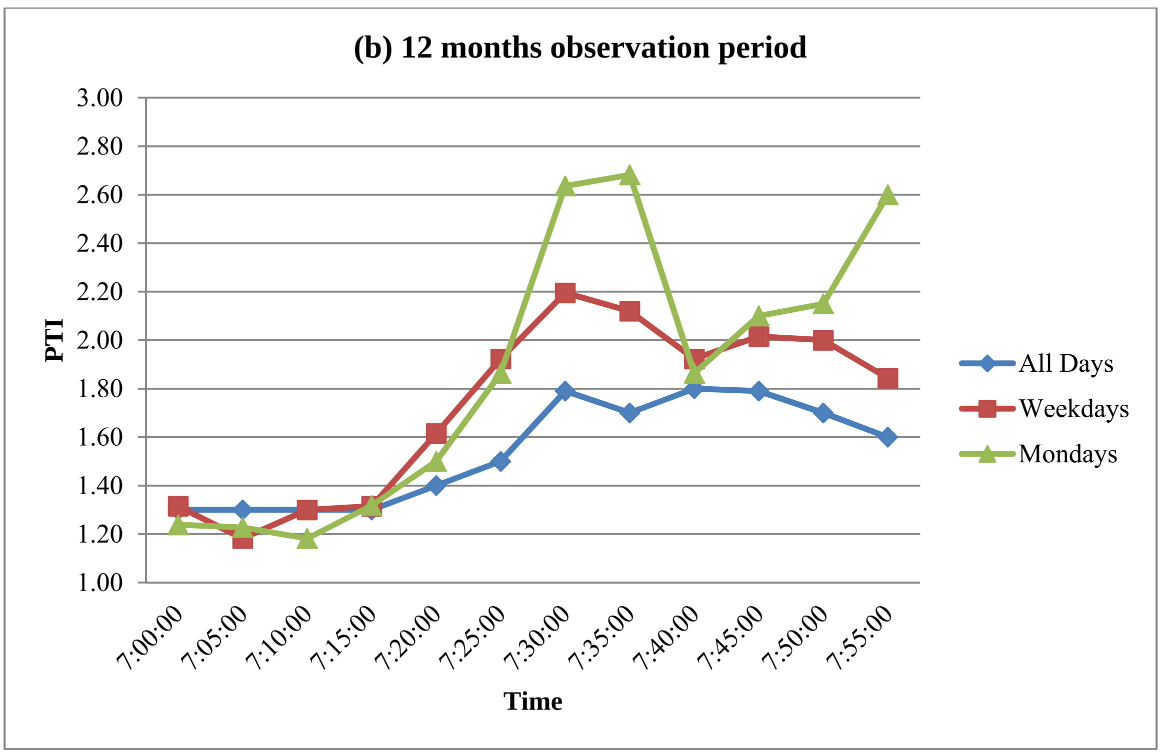
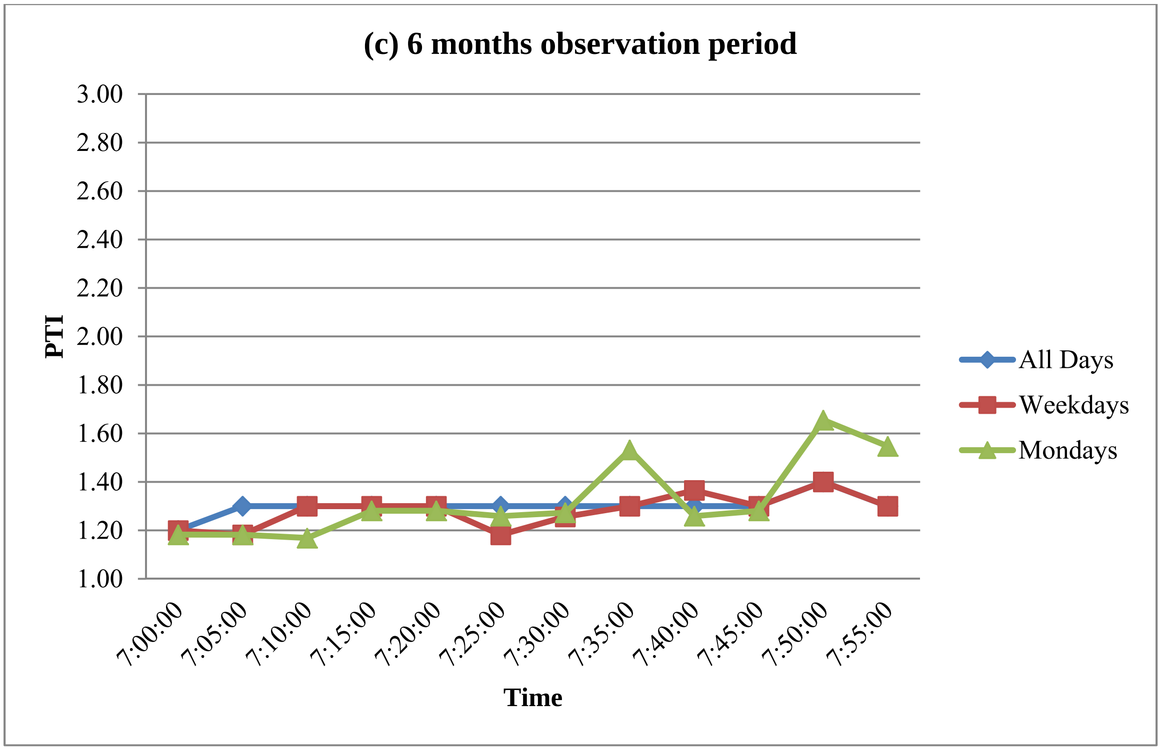


FIGURE 5.65: PTI variations for different observation criteria



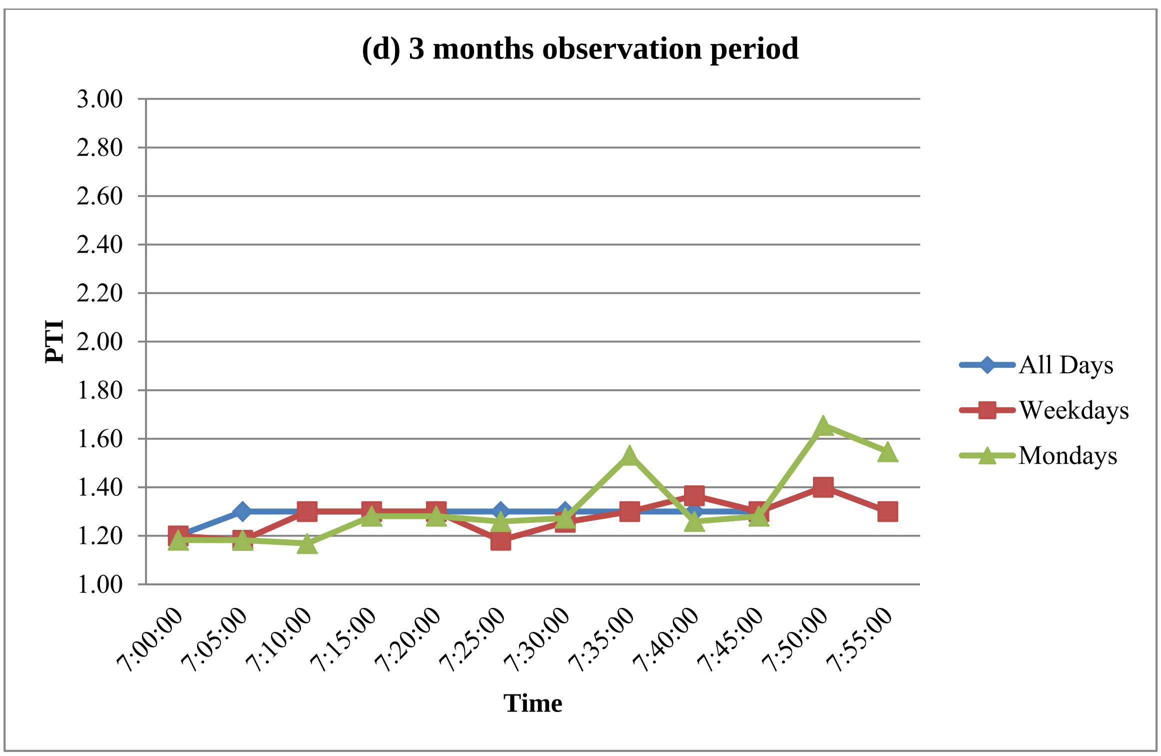


FIGURE 5.65 CONTD.: PTI variations for different observation criteria

5.7.2.2. Buffer Time Index based LOS

Table 5.19 shows the LOS predictions for July 1, 2013 based on 18, 12, 6, and 3 months of data based on the BTI. Table 5.20 shows the ANOVA results on the observation periods for BTI. Null hypothesis is all means are equal, while the alternative hypothesis is at least one mean is different (significance level, $\alpha = 0.01$). The F-value shows that null hypothesis is rejected and at least one combination has different mean. Similar to Sub-sub-section 5.5.2.1, Tukey HSD test (Table 5.21) also shows that the mean is similar between 18 and 12 months, and 6 and 3 months observation periods, and a difference in the mean exists for all other combinations for this exercise. Therefore, the observation period is a significant factor for the computation of the BTI.

Figure 5.66 shows the variation of BTI for different observation periods. The significance of observation period can also be identified. Figure 5.67 illustrates the BTI variations for different observation criteria. Similar to that of PTI, it can be noted that the weekdays and Mondays only BTI values were higher for the 18 and 12 months of observation periods. However, for 6 and 3 months of observation periods, the values are lower, which also shows the importance of observation period in the BTI analysis.

TABLE 5.19: BTI values and their respective LOSs

	18 months						12 months					
	All Days		Weekdays		Mondays		All Days		Weekdays		Mondays	
	BTI	LOS	BTI	LOS	BTI	LOS	BTI	LOS	BTI	LOS	BTI	LOS
7:00:00	0.08	B	0.08	B	0.08	B	0.08	B	0.10	B	0.08	B
7:05:00	0.08	B	0.08	B	0.27	C	0.08	B	0.08	B	0.13	B
7:10:00	0.08	B	0.08	B	0.08	B	0.08	B	0.08	B	0.08	B
7:15:00	0.08	B	0.17	C	0.37	D	0.08	B	0.10	B	0.21	C
7:20:00	0.17	C	0.40	D	0.67	F	0.17	C	0.35	D	0.38	D
7:25:00	0.33	C	0.72	F	0.75	F	0.25	C	0.76	F	0.71	F
7:30:00	0.50	E	0.92	F	1.17	F	0.49	E	1.01	F	1.42	F
7:35:00	0.42	D	0.83	F	1.73	F	0.42	D	0.92	F	1.46	F
7:40:00	0.49	E	0.67	F	1.05	F	0.50	E	0.76	F	0.71	F
7:45:00	0.50	E	0.65	F	0.75	F	0.49	E	0.68	F	0.75	F
7:50:00	0.33	C	0.50	E	0.53	E	0.42	D	0.67	F	0.79	F
7:55:00	0.34	D	0.63	F	0.43	D	0.33	C	0.67	F	1.17	F
	6 months						3 months					
	All Days		Weekdays		Mondays		All Days		Weekdays		Mondays	
	BTI	LOS	BTI	LOS	BTI	LOS	BTI	LOS	BTI	LOS	BTI	LOS
7:00:00	0.00	B	0.00	B	0.00	B	0.00	B	0.00	B	0.00	B
7:05:00	0.08	B	0.08	B	0.08	B	0.08	B	0.08	B	0.08	B
7:10:00	0.08	B	0.08	B	0.07	B	0.08	B	0.08	B	0.07	B
7:15:00	0.08	B	0.08	B	0.08	B	0.08	B	0.08	B	0.08	B
7:20:00	0.08	B	0.08	B	0.08	B	0.08	B	0.08	B	0.08	B
7:25:00	0.08	B	0.08	B	0.15	B	0.08	B	0.08	B	0.15	B
7:30:00	0.08	B	0.08	B	0.17	C	0.08	B	0.08	B	0.17	C
7:35:00	0.08	B	0.08	B	0.30	C	0.08	B	0.08	B	0.30	C
7:40:00	0.08	B	0.14	B	0.15	B	0.08	B	0.14	B	0.15	B
7:45:00	0.08	B	0.08	B	0.08	B	0.08	B	0.08	B	0.08	B
7:50:00	0.17	C	0.17	C	0.5	E	0.17	C	0.17	C	0.50	E
7:55:00	0.08	B	0.08	B	0.31	C	0.08	B	0.08	B	0.31	C

TABLE 5.20: ANOVA results on observation periods – BTI

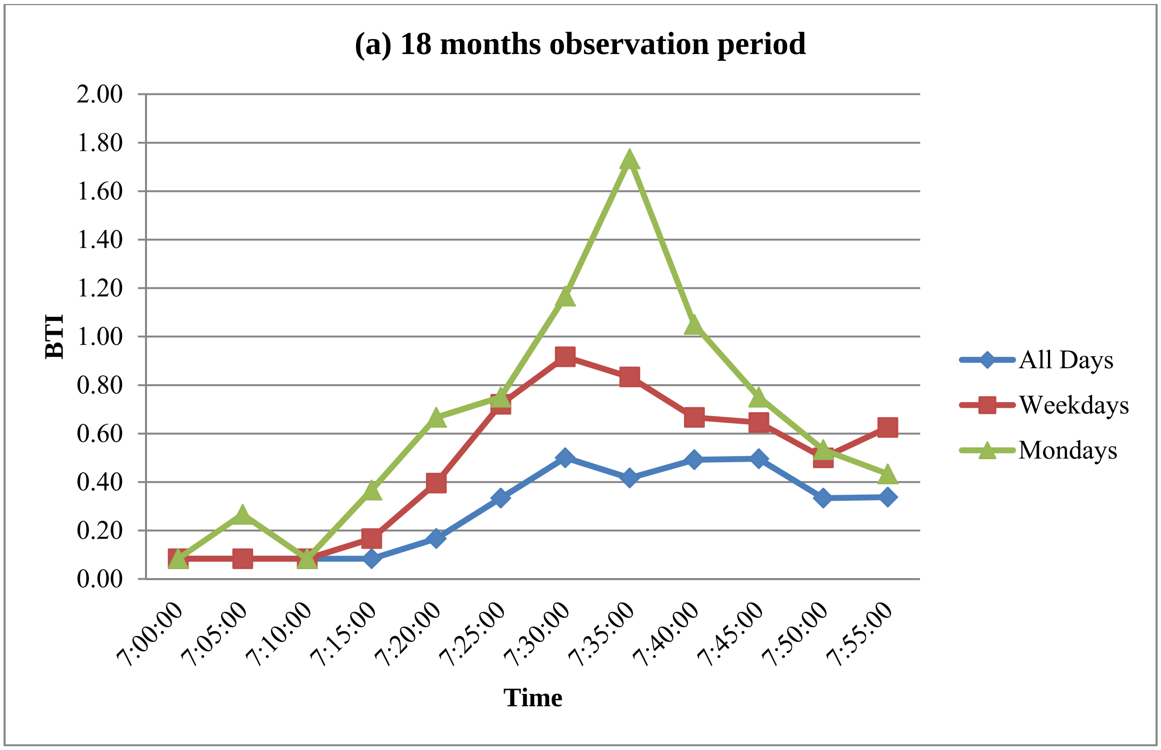
Factor Information					
Factor	Levels	Values			
Factor	4	18 months, 12 months, 6 months, 3 months			
Analysis of Variance					
Source	DF	Adj. SS	Adj. MS	F-Value	P-Value
Factor	3	4.833	1.61106	21.2	0
Error	140	10.637	0.07598		
Total	143	15.47			

TABLE 5.21: Tukey HSD test on observation periods – BTI

	12 months	6 months	3 months
18 months	n/s	P<0.01	P<0.01
12 months	-	P<0.01	P<0.01
6 months	-	-	n/s

n/s= non-significant

FIGURE 5.66: BTI variations for different observation periods



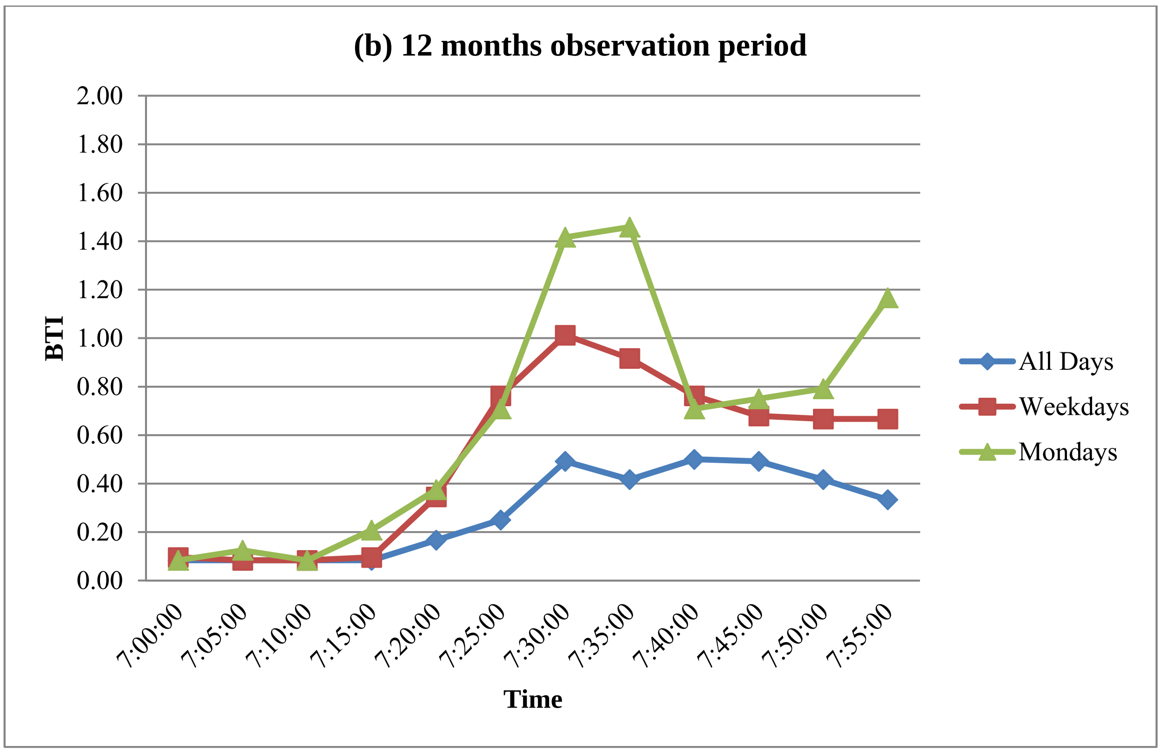
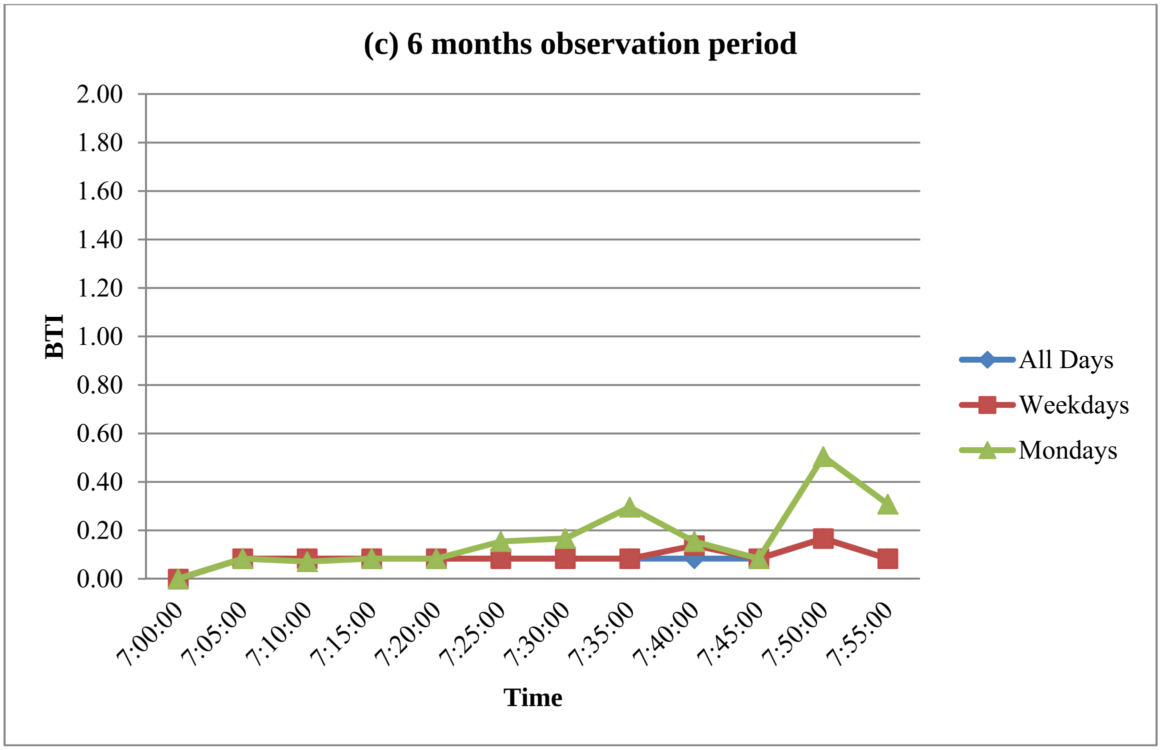


FIGURE 5.67: BTI variations for different observation criteria



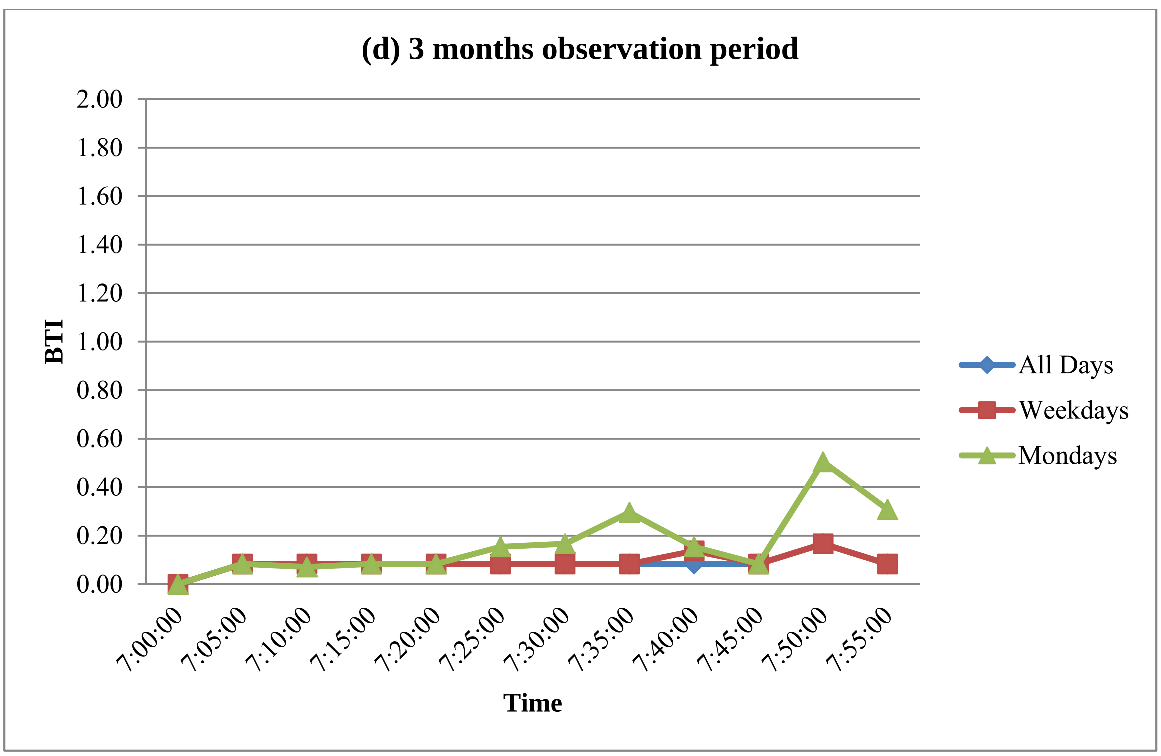


FIGURE 5.67 CONTD.: BTI variations for different observation criteria

5.7.3. Section Travel Time Reliability Composite LOS

A composite section level travel time reliability based LOS can be obtained by using a fuzzy logic set for a specific time window. For the TMC 102+04871, each 5-minute LOS can be evaluated to obtain a composite LOS for the entire hour. For that, the average of the membership function for each LOS is computed. This, defined as compatibility index, shows which LOS is predominant in the entire hour and in turn, has the highest degree of membership.

5.7.3.1. Planning Time Index based Composite LOS

The LOS with the highest degree of membership denotes the composite LOS for the entire hour. The PTI LOS ranges are simply related with LOS thresholds for that specific segment of roadways. Both the LOS ranges and 5th percentile travel time values are observation period dependent. Tables 5.22 (a) to (d) show the composite LOS based on PTI for 18, 12, 6, and 3 months of observation periods, respectively. It can be noted that both Tables 5.22 (a) and (b) predict higher travel time and LOS for the entire hour based on the 18 and 12 months data. However, Tables 5.22 (c) and (d) predict lower travel time for 6 and 3 months of data, as the 95th percentile average travel times are lower for the latter two observation periods.

TABLE 5.22 (a): Composite PTI based LOS for a section based on CI – 18 months of observation period

For the weaving section: 7:00 AM – 8:00 AM	All Days		Weekdays		Mondays	
If the service level is A:	CI =	0.00	CI =	0.00	CI =	0.00
If the service level is B:	CI =	0.00	CI =	0.25	CI =	0.17
If the service level is C:	CI =	0.50	CI =	0.08	CI =	0.17
If the service level is D:	**CI =**	**0.50**	**CI =**	**0.42**	CI =	0.17
If the service level is E:	CI =	0.00	CI =	0.25	**CI =**	**0.33**
If the service level is F:	CI =	0.00	CI =	0.00	CI =	0.17

PTI Range based on LOS	From	1.50	1.50	1.83
	To	1.83	1.83	2.26
5^{th} percentile TT (sec)		6.05	6.60	6.53
TT predicted (sec)	From	9.09	9.92	11.92
	To	11.05	12.05	14.77

TABLE 5.22 (b): Composite PTI based LOS for a section based on CI – 12 months of observation period

For the weaving section: 7:00 AM – 8:00 AM	All Days		Weekdays		Mondays	
If the service level is A:	CI =	0.00	CI =	0.00	CI =	0.00
If the service level is B:	CI =	0.00	CI =	0.08	CI =	0.08
If the service level is C:	CI =	0.50	CI =	0.25	CI =	0.33
If the service level is D:	**CI =**	**0.50**	CI =	0.08	CI =	0.00
If the service level is E:	CI =	0.00	**CI =**	**0.58**	**CI =**	**0.33**
If the service level is F:	CI =	0.00	CI =	0.00	CI =	0.25

PTI Range based on LOS	From	1.50	1.83	1.83
	To	1.83	2.26	2.26
5^{th} percentile TT (sec)		6.00	6.29	6.43
TT predicted (sec)	From	9.01	11.47	11.73
	To	10.95	14.23	14.54

TABLE 5.22 (c): Composite PTI based LOS for a section based on CI – 6 months of observation period

For the weaving section: 7:00 AM – 8:00 AM	All Days		Weekdays		Mondays	
If the service level is A:	CI =	0.00	CI =	0.00	CI =	0.00
If the service level is B:	CI =	0.08	CI =	0.25	CI =	0.25
If the service level is C:	**CI =**	**0.92**	**CI =**	**0.75**	**CI =**	**0.50**
If the service level is D:	CI =	0.00	CI =	0.00	CI =	0.25
If the service level is E:	CI =	0.00	CI =	0.00	CI =	0.00
If the service level is F:	CI =	0.00	CI =	0.00	CI =	0.00

PTI Range based on LOS	From	1.20	1.20	1.20
	To	1.50	1.50	1.50
5th percentile TT (sec)		6.00	6.12	6.30
TT predicted (sec)	From	7.20	7.35	7.56
	To	9.01	9.19	9.46

TABLE 5.22 (d): Composite PTI based LOS for a section based on CI – 3 months of observation period

For the weaving section: 7:00 AM – 8:00 AM	All Days		Weekdays		Mondays	
If the service level is A:	CI =	0.00	CI =	0.00	CI =	0.00
If the service level is B:	CI =	0.08	CI =	0.25	CI =	0.25
If the service level is C:	**CI =**	**0.92**	**CI =**	**0.75**	**CI =**	**0.50**
If the service level is D:	CI =	0.00	CI =	0.00	CI =	0.25
If the service level is E:	CI =	0.00	CI =	0.00	CI =	0.00
If the service level is F:	CI =	0.00	CI =	0.00	CI =	0.00

PTI Range based on LOS	From	1.20	1.20	1.20
	To	1.50	1.50	1.50
5th percentile TT (sec)		5.97	6.02	6.34
TT predicted (sec)	From	7.17	7.22	7.62
	To	8.97	9.04	9.53

5.7.3.2. Buffer Time Index based Composite LOS

Similar to 5.7.3.1, the LOS with the highest degree of membership denotes the composite LOS for the entire hour. The BTI LOS ranges and 50th percentile travel time values are observation period dependent. The BTI LOS ranges are also related with LOS thresholds for that specific segment of roadways. Tables 5.23 (a) to (d) show the composite LOS based on BTI for 18, 12, 6, and 3 months of observation periods, respectively. It can be noted that both Tables 5.23 (a) and (b) predict higher travel time and LOS for the entire hour based on the 18 and 12 months data, while Table 5.23 (c) and (d) predict lower travel times and LOS values.

TABLE 5.23 (a): Composite BTI based LOS for a section based on CI – 18 months of observation period

For the weaving section: 7:00 AM - 8:00 AM	All Days		Weekdays		Mondays	
If the service level is A:	CI =	0.00	CI =	0.00	CI =	0.00
If the service level is B:	**CI =**	**0.33**	CI =	0.25	CI =	0.17
If the service level is C:	CI =	0.25	CI =	0.08	CI =	0.08
If the service level is D:	CI =	0.17	CI =	0.08	CI =	0.17
If the service level is E:	CI =	0.25	CI =	0.08	CI =	0.08
If the service level is F:	CI =	0.00	**CI =**	**0.50**	**CI =**	**0.50**

		All Days	Weekdays	Mondays
BTI Range based on LOS	From	-0.09	0.60	0.60
	To	0.16	-	-
50th percentile TT (sec)		7.20	7.20	7.20
TT predicted (sec)	From	6.53	11.52	11.52
	To	7.71	-	-

TABLE 5.23 (b): Composite BTI based LOS for a section based on CI – 12 months of observation period

For the weaving section: 7:00 AM - 8:00 AM	All Days		Weekdays		Mondays	
If the service level is A:	CI =	0.00	CI =	0.00	CI =	0.00
If the service level is B:	CI =	**0.33**	CI =	0.33	CI =	0.25
If the service level is C:	CI =	0.25	CI =	0.00	CI =	0.08
If the service level is D:	CI =	0.17	CI =	0.08	CI =	0.08
If the service level is E:	CI =	0.25	CI =	0.00	CI =	0.00
If the service level is F:	CI =	0.00	**CI =**	**0.58**	**CI =**	**0.58**

BTI Range based on LOS	From	-0.09	0.60	0.60
	To	0.16	-	-
50^{th} percentile TT (sec)		7.20	7.20	7.20
TT predicted (sec)	From	6.53	11.52	11.52
	To	7.71	-	-

TABLE 5.23 (c): Composite BTI based LOS for a section based on CI – 6 months of observation period

For the weaving section: 7:00 AM - 8:00 AM	All Days		Weekdays		Mondays	
If the service level is A:	CI =	0.00	CI =	0.00	CI =	0.00
If the service level is B:	**CI =**	**0.92**	**CI =**	**0.92**	**CI =**	**0.67**
If the service level is C:	CI =	0.08	CI =	0.08	CI =	0.25
If the service level is D:	CI =	0.00	CI =	0.00	CI =	0.00
If the service level is E:	CI =	0.00	CI =	0.00	CI =	0.08
If the service level is F:	CI =	0.00	CI =	0.00	CI =	0.00

BTI Range based on LOS	From	-0.09	-0.09	-0.09
	To	0.16	0.16	0.16
50^{th} percentile TT (sec)		7.20	7.20	7.15
TT predicted (sec)	From	6.53	6.53	6.49
	To	8.38	8.38	8.32

Table 5.23 (d): Composite BTI based LOS for a section based on CI – 3 months of observation period

For the weaving section: 7:00 AM - 8:00 AM	All Days		Weekdays		Mondays	
If the service level is A:	CI =	0.00	CI =	0.00	CI =	0.00
If the service level is B:	**CI =**	**0.92**	**CI =**	**0.92**	**CI =**	**0.67**
If the service level is C:	CI =	0.08	CI =	0.08	CI =	0.25
If the service level is D:	CI =	0.00	CI =	0.00	CI =	0.00
If the service level is E:	CI =	0.00	CI =	0.00	CI =	0.08
If the service level is F:	CI =	0.00	CI =	0.00	CI =	0.00

BTI Range based on LOS	From	-0.09	-0.09	-0.09
	To	0.16	0.16	0.16
50^{th} percentile TT (sec)		7.00	7.10	6.98
TT predicted (sec)	From	6.35	6.44	6.33
	To	8.14	8.26	8.11

CHAPTER 6: CONCLUSIONS

6.1. Introduction

Travel time is one of the most important metrics for the practitioners and used in various transportation planning related decision making processes. It is also easily understood by road users and helps them choose their routes to reach their destinations quickly. Real time continuous data collection is possible through the use of Bluetooth detectors, on-road sensors, traffic cameras, and other technologies. This dissertation provides a new method to estimate the freeway LOS based on travel time and travel time reliability indices. Though the dissertation had some limitations, the overall contributions can open a new horizon for research.

6.2. Summary of Limitations

Some travel time/mile relationships with respective densities have signs that are contrary to expectations and the dissertation did not provide a full explanation of these "counter-intuitive signs", which could be attributed to the unmet demand that the VISSIM software captures.

The calibration procedure was performed based on Jacksonville, FL data. While the considered freeway corridor is representative of typical urban freeway corridors, a nationwide data source may be used to better understand the density – travel time per mile and density – travel time reliability relationships based on different lane capacities and study areas.

This dissertation did not consider type B and type C weaving sections, which could show some variations on the relationships developed between density and travel time based metrics. The analysis only considered single lane on- and off-ramps. Multiple on- and off-ramp configurations can affect the maneuverability, and hence, can affect the travel time.

Some factors were not considered in the simulation and analysis. They include median type / width and shoulder width, which can be correlated to speed and in return, may become a significant factor influencing travel time. Other factors, such as network familiarity, access control and spacing, and percent of heavy vehicles can also play a role on operational performance of freeways.

This dissertation focused on freeway LOS criteria based on travel time based metrics that is comparable to HCM methods. Hence, it did not account for the effect of incidents, inclement weather, special events, or construction activity directly during the simulation process. However, the method can be adopted if a calibrated and validated microscopic simulation model can be built to replicate such scenarios and develop LOS thresholds.

6.3. Summary of Contributions

Despite these limitations, the dissertation offered several contributions to the freeway LOS determination method. They are listed as follows.

- It is not often feasible and at times impossible to gather information on travelers' O–D patterns and density on urban freeways. However, travel time on a section, can provide an easy and readily assessable LOS criteria establishment. This dissertation looked into several key components on the freeway geometries, such as length and the number of lanes, and demand fluctuations and different posted speed limits, to establish a meaningful relationship between density and travel time (per mile), and density and travel time reliability indices for different freeway sections, such as basic freeway section, weaving section, and merging/diverging area.

- The dissertation finds a strong correlation between HCM based densities and densities from VISSIM, which further validates the notion that a calibrated microscopic simulation model can be effectively used to represent general traffic behavior. It also finds that the correlation coefficients for diverging areas are somewhat lower than that of merging areas. As the maneuverability of these two areas are fundamentally different (merging vs. diverging), question can be raised on combining them together for LOS analysis purpose.
- The density – travel time/mile relationship shows a non-linear (exponential) relationship for all the freeway section types, which further questions the generic speed assumptions made by HCM for different LOS profiles. A polynomial relationship was observed between density – travel time reliability indices.
- The travel time reliability thresholds based on planning time index and buffer time index (travel time reliability computation methods) were evaluated on freeway sections. Both of them can easily be expressed to technical audiences (e.g., traffic engineers, transportation planners, and MPO staff) and non-technical audiences (e.g., daily travelers, business owners, and truck drivers), in particular, policy makers, should funding be provided, to improve travel time reliability on a corridor.

- It was found that average travel time per mile threshold values for respective LOS letters increase as the speed limit decreases until the condition comes close to saturation where the speed limit on the freeway does not have any influence on the operation. It can also be noted that as the posted limit decreases, the percent difference between the two respective adjacent travel time per mile threshold values also decreases.
- The dissertation also finds that the average travel time reliability LOS threshold values for their respective LOS letters decrease for all freeway section types as the speed limit decreases. For PTI based thresholds, the 95th percentile travel time decreases as the speed limit decreases but the 5th percentile travel times remain relatively similar. In case of BTI based thresholds, the 95th and 50th percentile travel time values become closer as the speed limit decreases.
- For all freeway section types, the percent difference between the two respective adjacent PTI LOS threshold values remains relatively similar, or increase or decrease slightly as the speed limit decreases. However, as the speed limit decreases, the percent difference between the two adjacent BTI LOS thresholds tends to increase.
- The dissertation also showed that based on the observation period (number of data points), the LOS estimation can differ significantly. Therefore, while estimating the LOS based on travel time per mile or reliability metrics or merely calculating reliability indices for a particular segment, care should be taken on the observation period considered for the assessment.

- Both travel time per mile LOS and travel time reliability LOS criteria can be used to evaluate separate operational conditions. The travel time LOS criteria can be effectively used for evaluating before – after study, while the travel time reliability LOS criteria can better describe studies as congestion ranking.

6.4. Implications and Recommendations for Future Research

This dissertation has established baselines for researchers to examine the role of other influencing geometric and operational factors on travel time and travel time reliability thresholds. These factors could be attributed to lane capacity, median width, ramp characteristics, etc.

Most importantly, further research needs to be conducted on identifying generic driver and lane change parameters obtained through calibration of different location based microscopic simulation models. Since these researches require considerable amount of funding, a federal or state backed investigation is ideal.

The dissertation method can be considered as a theoretical process. For practical applications, field observation is recommended to further validate and apply the method. A similar method can be developed and applied for the arterial analysis with the consideration of signal timing fluctuations and travel time collection approaches.

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APPENDIX A: HOURLY FLOW COMPARISONS

TABLE A1: Hourly flow comparisons – AM peak period - hour 1

Roadway	VISSIM Link Number	Location	Peak Hour Count Volume (vph)	VISSIM Model Volume (vph)	Difference	Within 15%	Volume Criteria Met	Individual Link Flow < 700 vph		Individual Link Flow 700 vph to 2,700 vph		Individual Link Flow > 2,700 vph		GEH Statistic	GEH Criteria Met (<= 5)
								Within 100vph	Criteria Met	Within 15%	Criteria Met	Within 400vph	Criteria Met		
I-295 Northbound	1001	Upstream of I-95 on-ramps	1,515	1,513	-2	-0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	1002	Upstream of S.R. 5 (US 1) off-ramp	2,516	2,512	-4	-0.2%	YES	-	-	0%	YES	-	-	0.1	YES
	1003	Between S.R. 5 (US 1) off- and on-ramps	2,028	2,028	0	0.0%	YES	-	-	0%	YES	-	-	0.0	YES
	1004	S.R. 5 (US 1) on-ramp merge	2,362	2,351	-11	-0.4%	YES	-	-	0%	YES	-	-	0.2	YES
	1005	Downstream of S.R. 5 (US 1) on-ramp merge	2,362	2,360	-2	-0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	1006	3-lane to 2-lane merge - North of S.R. 5 (US 1)	2,362	2,362	0	0.0%	YES	-	-	0%	YES	-	-	0.0	YES
	1007	Upstream of S.R. 9B on-ramp	2,362	2,378	16	0.7%	YES	-	-	1%	YES	-	-	0.3	YES
	1008	S.R. 9B on-ramp merge	2,991	3,048	57	1.9%	YES	-	-	-	-	57	YES	1.0	YES
	1009	Downstream of S.R. 9B on-ramp merge - 3-lane section	2,991	3,229	238	8.0%	YES	-	-	-	-	238	YES	4.3	YES
	1010	Downstream of S.R. 9B on-ramp merge - 2-lane section	2,991	3,277	286	9.6%	YES	-	-	-	-	286	YES	5.1	NO
	1011	Upstream of S.R. 152 (Baymeadows Rd.) off-ramp	2,991	3,280	289	9.7%	YES	-	-	-	-	289	YES	5.2	NO
	1012	Between S.R. 152 (Baymeadows Rd.) off- and on-ramps	2,734	3,001	267	9.8%	YES	-	-	-	-	267	YES	5.0	YES
	1013	Between S.R. 152 on-ramp and Gate Pkwy. off-ramp	3,235	3,525	290	9.0%	YES	-	-	-	-	290	YES	5.0	YES
	1014	Between Gate Pkwy. off- and on-ramps	2,638	2,876	238	9.0%	YES	-	-	9%	YES	-	-	4.5	YES
	1015	Between Gate Pkwy. on-ramp and S.R. 202 (JTB) off-ramp	3,228	3,465	237	7.3%	YES	-	-	-	-	237	YES	4.1	YES
	1016	Downstream of of S.R. 202 (JTB) off-ramp - 3-lane section	1,688	1,810	122	7.2%	YES	-	-	7%	YES	-	-	2.9	YES
	1017	Downstream of of S.R. 202 (JTB) off-ramp - 2-lane section	1,688	1,815	127	7.5%	YES	-	-	8%	YES	-	-	3.0	YES
	1018	Eastbound S.R. 202 (JTB) on-ramp merge	2,176	2,299	123	5.7%	YES	-	-	6%	YES	-	-	2.6	YES
	1019	Downstream of Eastbound S.R. 202 (JTB) on-ramp merge	2,176	2,301	125	5.7%	YES	-	-	6%	YES	-	-	2.6	YES
	1020	Between WB S.R. 202 (JTB) on-ramp and Town Center Pkwy. off-ramp	2,606	2,752	146	5.6%	YES	-	-	6%	YES	-	-	2.8	YES
	1021	Between Town Center Pkwy. off- and on-ramps	1,996	2,122	126	6.3%	YES	-	-	6%	YES	-	-	2.8	YES
	1022	Town Center Pkwy. on-ramp merge	2,310	2,449	139	6.0%	YES	-	-	6%	YES	-	-	2.9	YES
I-295 Northbound - Ramps	1501	I-95 Southbound on-ramp	135	131	-4	-2.7%	YES	-4	YES	-	-	-	-	0.3	YES
	1502	I-95 Northbound on-ramp	866	868	2	0.2%	YES	-	-	0%	YES	-	-	0.1	YES
	1503	I-95 on-ramp (combined) - 2-lane section	1,001	999	-2	-0.2%	YES	-	-	0%	YES	-	-	0.1	YES
	1504	I-95 on-ramp (combined) - 1-lane section	1,001	996	-5	-0.5%	YES	-	-	0%	YES	-	-	0.2	YES
	1505	US-1 Off-Ramp	488	493	5	1.0%	YES	5	YES	-	-	-	-	0.2	YES
	1513	US-1 On-Ramp	334	326	-8	-2.4%	YES	-8	YES	-	-	-	-	0.4	YES
	1514	Baymeadows Rd. Off-Ramp	257	282	25	9.6%	YES	25	YES	-	-	-	-	1.5	YES
	1520	Baymeadows Rd. On-Ramp	501	513	12	2.4%	YES	12	YES	-	-	-	-	0.5	YES
	1521	Gate Pkwy. Off-Ramp	597	655	58	9.8%	YES	58	YES	-	-	-	-	2.3	YES
	1527	Gate Pkwy. On-Ramp	590	592	2	0.3%	YES	2	YES	-	-	-	-	0.1	YES
	1528	SR 202 (JTB) Off-Ramp (combined)	1,540	1,655	115	7.5%	YES	-	-	7%	YES	-	-	2.9	YES
	1529	SR 202 (JTB) WB Off-Ramp	469	513	44	9.4%	YES	44	YES	-	-	-	-	2.0	YES
	1530	SR 202 (JTB) EB Off-Ramp	1,072	1,142	70	6.5%	YES	-	-	7%	YES	-	-	2.1	YES
	1531	SR 202 (JTB) On-Ramp - 2-lane section	1,637	1,646	9	0.5%	YES	-	-	1%	YES	-	-	0.2	YES
	1532	SR 202 (JTB) On-Ramp - 1-lane section	1,637	1,647	10	0.6%	YES	-	-	1%	YES	-	-	0.2	YES
	1533	On-Ramp from WB SR 202 (JTB)	430	436	6	1.4%	YES	6	YES	-	-	-	-	0.3	YES
	1534	On-Ramp from EB SR 202 (JTB)	488	488	0	0.1%	YES	0	YES	-	-	-	-	0.0	YES
	1535	Town Center Pkwy Off-Ramp	610	634	24	3.9%	YES	24	YES	-	-	-	-	0.9	YES
	1539	Town Center Pkwy On-Ramp	314	311	-3	-1.1%	YES	-3	YES	-	-	-	-	0.2	YES
I-295 Southbound	3001	Upstream of Town Center Pkwy. off-ramp	3,382	3,374	-8	-0.2%	YES	-	-	-	-	-8	YES	0.1	YES
	3002	Between Town Center Pkwy. off- and on-ramps	2,798	2,836	38	1.3%	YES	-	-	-	-	38	YES	0.7	YES
	3003	Between Town Center Pkwy. On-ramp and S.R. 202 (JTB) off-ramp (combined)	3,196	3,253	57	1.8%	YES	-	-	-	-	57	YES	1.0	YES
	3004	Between S.R. 202 (JTB) off-ramp (combined) and Westbound S.R. 202 (JTB) on-ramp	1,752	1,795	43	2.5%	YES	-	-	2%	YES	-	-	1.0	YES
	3005	Westbound S.R. 202 (JTB) on-ramp merge	2,959	3,005	46	1.6%	YES	-	-	-	-	46	YES	0.9	YES
	3006	Downstream of Westbound S.R. 202 (JTB) on-ramp merge	2,959	2,942	-17	-0.6%	YES	-	-	-	-	-17	YES	0.3	YES
	3007	Between Eastbound S.R. 202 (JTB) on-ramp and Gate Pkwy. off-ramp	3,132	3,207	75	2.4%	YES	-	-	-	-	75	YES	1.3	YES
	3008	Between Gate Pkwy off- and on- ramps - 4-lane section	2,734	2,804	70	2.5%	YES	-	-	-	-	70	YES	1.3	YES
	3009	Between Gate Pkwy off- and on- ramps - 3-lane section	2,734	2,821	87	3.2%	YES	-	-	-	-	87	YES	1.6	YES
	3010	Between Gate Pkwy off- and on- ramps - 2-lane section	2,734	2,820	86	3.1%	YES	-	-	-	-	86	YES	1.6	YES
	3011	Between Gate Pkwy. on-ramp and S.R. 152 (Baymeadows Rd.) off-ramp	2,875	2,971	96	3.3%	YES	-	-	-	-	96	YES	1.8	YES
	3012	Between S.R. 152 (Baymeadows Rd.) off- and on-ramps	2,323	2,408	85	3.7%	YES	-	-	4%	YES	-	-	1.7	YES
	3013	S.R. 152 (Baymeadows Rd.) on-ramp merge	2,548	2,597	49	1.9%	YES	-	-	2%	YES	-	-	1.0	YES
	3014	Downstream of S.R. 152 (Baymeadows Rd.) on-ramp merge	2,548	2,633	85	3.3%	YES	-	-	3%	YES	-	-	1.7	YES
	3015	Upstream of S.R. 9B off-ramp - 2-lane section	2,548	2,639	91	3.6%	YES	-	-	4%	YES	-	-	1.8	YES
	3016	Upstream of S.R. 9B off-ramp - 3-lane section	2,548	2,644	96	3.8%	YES	-	-	4%	YES	-	-	1.9	YES
	3017	Downstraem of S.R. 9B off-ramp - 2-lane section	2,073	2,158	85	4.1%	YES	-	-	4%	YES	-	-	1.9	YES
	3018	Downstraem of S.R. 9B off-ramp - 3-lane section	2,073	2,163	90	4.4%	YES	-	-	4%	YES	-	-	2.0	YES
	3019	Upstream of S.R. 5 (US 1) off-ramp - 3-lane section	2,073	2,168	95	4.6%	YES	-	-	5%	YES	-	-	2.1	YES
	3020	Upstream of S.R. 5 (US 1) off-ramp - 4-lane section	2,073	2,167	94	4.5%	YES	-	-	5%	YES	-	-	2.0	YES
	3021	Between S.R. 5 (US 1) off- and on-ramps	1,515	1,585	70	4.6%	YES	-	-	5%	YES	-	-	1.8	YES
	3022	Downstream of S.R. 5 (US 1) on-ramp	1,977	2,086	109	5.5%	YES	-	-	6%	YES	-	-	2.4	YES
	3023	Upstream of Northbound I-95 off-ramp	1,977	2,077	100	5.1%	YES	-	-	5%	YES	-	-	2.2	YES
	3024	Downstream of Northbound I-95 off-ramp	1,675	1,765	90	5.3%	YES	-	-	5%	YES	-	-	2.2	YES
	3025	Upstream of Southbound I-95 off-ramp - 3-lane section	1,675	1,765	90	5.4%	YES	-	-	5%	YES	-	-	2.2	YES
	3026	Upstream of Southbound I-95 off-ramp - 4-lane section	1,675	1,762	87	5.2%	YES	-	-	5%	YES	-	-	2.1	YES
	3027	Downstream of Southbound I-95 off-ramp - 3-lane section	1,078	1,118	40	3.7%	YES	-	-	4%	YES	-	-	1.2	YES
	3028	Downstream of Southbound I-95 off-ramp - 2-lane section	1,078	1,143	65	6.0%	YES	-	-	6%	YES	-	-	1.9	YES
I-295 Southbound - Ramps	3501	Town Center Pkwy Off-Ramp	584	580	-4	-0.7%	YES	-4	YES	-	-	-	-	0.2	YES
	3505	Town Center Pkwy On-Ramp	398	404	6	1.6%	YES	6	YES	-	-	-	-	0.3	YES
	3506	SR 202 (JTB) Off-Ramp (combined)	1,444	1,461	17	1.2%	YES	-	-	1%	YES	-	-	0.4	YES
	3507	SR 202 (JTB) WB On-Ramp	841	857	16	1.9%	YES	-	-	2%	YES	-	-	0.5	YES
	3508	SR 202 (JTB) EB On-Ramp	603	605	2	0.4%	YES	2	YES	-	-	-	-	0.1	YES
	3509	On-Ramp from WB SR 202 (JTB)	1,207	1,217	10	0.8%	YES	-	-	1%	YES	-	-	0.3	YES
	3510	On-Ramp from EB SR 202 (combined)	661	674	13	1.9%	YES	13	YES	-	-	-	-	0.5	YES
	3511	On-Ramp from EB SR 202 (JTB)	173	187	14	8.4%	YES	14	YES	-	-	-	-	1.1	YES
	3512	Gate Pkwy. Off-Ramp	398	395	-3	-0.8%	YES	-3	YES	-	-	-	-	0.2	YES
	3516	Gate Pkwy. On-Ramp - 2-lane segment	141	138	-3	-1.8%	YES	-3	YES	-	-	-	-	0.2	YES
	3517	Gate Pkwy. On-Ramp - 1-lane segment	141	140	-1	-0.7%	YES	-1	YES	-	-	-	-	0.1	YES
	3518	Baymeadows Rd. Off-Ramp	552	575	23	4.1%	YES	23	YES	-	-	-	-	1.0	YES
	3523	Baymeadows Rd. On-Ramp	225	221	-4	-1.9%	YES	-4	YES	-	-	-	-	0.3	YES
	3524	US-1 Off-Ramp	558	586	28	5.1%	YES	28	YES	-	-	-	-	1.2	YES
	3530	US-1 On-Ramp - 2-lane segment	462	490	28	6.0%	YES	28	YES	-	-	-	-	1.3	YES
	3531	US-1 On-Ramp - 1-lane segment	462	488	26	5.7%	YES	26	YES	-	-	-	-	1.2	YES
	3532	I-95 NB Off-Ramp	302	320	18	6.1%	YES	18	YES	-	-	-	-	1.0	YES
	3533	I-95 SB Off-Ramp	597	645	48	8.1%	YES	48	YES	-	-	-	-	1.9	YES
S.R. 9B	17001	Northbound S.R. 9B	629	628	-1	-0.1%	YES	-1	YES	-	-	-	-	0.0	YES
	19001	Southbound S.R. 9B	475	495	20	4.2%	YES	20	YES	-	-	-	-	0.9	YES
S.R. 202 (JTB) Eastbound	16001	Upstream of Gate Pkwy. off-ramp - Section 1	2,458	2,457	-1	-0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	16002	Upstream of Gate Pkwy. off-ramp - Section 2	2,458	2,458	0	0.0%	YES	-	-	0%	YES	-	-	0.0	YES
	16003	Between Gate Pkwy. off- and on-ramps	1,810	1,817	7	0.4%	YES	-	-	0%	YES	-	-	0.2	YES
	16004	Between Gate Pkwy. on-ramp and I-295 off-ramp	2,233	2,258	25	1.1%	YES	-	-	1%	YES	-	-	0.5	YES
	16005	Between I-295 off-ramp and Southbound I-295 on-ramp	1,572	1,593	21	1.3%	YES	-	-	1%	YES	-	-	0.5	YES
	16006	Southbound I-295 on-ramp merge	2,176	2,190	14	0.7%	YES	-	-	1%	YES	-	-	0.3	YES
	16007	Downstream of Southbound I-295 on-ramp merge	2,176	2,142	-34	-1.6%	YES	-	-	-2%	YES	-	-	0.7	YES
	16008	Between Northbound I-295 on-ramp and Kernan Blvd. off-ramp	3,248	3,348	100	3.1%	YES	-	-	-	-	100	YES	1.7	YES
	16009	Between Kernan Blvd. off- and on- ramp - 4-lane section	2,811	2,896	85	3.0%	YES	-	-	-	-	85	YES	1.6	YES
	16010	Between Kernan Blvd. off- and on- ramp - 3-lane section	2,811	2,899	88	3.1%	YES	-	-	-	-	88	YES	1.6	YES
	16011	Kernan Blvd. on-ramp merge - 4-lane section	3,036	3,132	96	3.2%	YES	-	-	-	-	96	YES	1.7	YES
	16012	Kernan Blvd. on-ramp merge - 3-lane section	3,036	3,083	47	1.6%	YES	-	-	-	-	47	YES	0.9	YES
	16013	Downstream of Kernan Blvd. on-ramp	3,036	3,158	122	4.0%	YES	-	-	-	-	122	YES	2.2	YES
S.R. 202 (JTB) Eastbound - Ramps	16501	Gate Pkwy. Off-Ramp	648	647	-1	-0.2%	YES	-1	YES	-	-	-	-	0.0	YES
	16505	Gate Pkwy. On-Ramp	424	439	15	3.6%	YES	15	YES	-	-	-	-	0.7	YES
	16506	Kernan Blvd. Off-Ramp	436	451	15	3.4%	YES	15	YES	-	-	-	-	0.7	YES
	16510	Kernan Blvd. On-Ramp	225	236	11	4.8%	YES	11	YES	-	-	-	-	0.7	YES
S.R. 202 (JTB) Westbound	18001	Upstream of Kernan Blvd. off-ramp - section 1	3,844	3,850	6	0.2%	YES	-	-	-	-	6	YES	0.1	YES
	18002	Upstream of Kernan Blvd. off-ramp - section 2	3,844	3,865	21	0.6%	YES	-	-	-	-	21	YES	0.3	YES
	18003	Between Kernan Blvd. off- and on-ramps	3,729	3,753	24	0.7%	YES	-	-	-	-	24	YES	0.4	YES
	18004	Between Kernan Blvd. on-ramp and I-295 off-ramp (combined)	4,897	4,947	50	1.0%	YES	-	-	-	-	50	YES	0.7	YES
	18005	Between I-295 off-ramp (combined) and Northbound I-295 on-ramp	3,260	3,310	50	1.5%	YES	-	-	-	-	50	YES	0.9	YES
	18006	Northbound I-295 on-ramp merge - 5-lane section	3,729	3,815	86	2.3%	YES	-	-	-	-	86	YES	1.4	YES
	18007	Northbound I-295 on-ramp merge - 4-lane section	3,729	3,819	90	2.4%	YES	-	-	-	-	90	YES	1.5	YES
	18008	Between Southbound I-295 on-ramp and Gate Pkwy. Off-ramp	4,570	4,706	136	3.0%	YES	-	-	-	-	136	YES	2.0	YES
	18009	Between Gate Pkwy. off- and on-ramps	3,466	3,555	89	2.6%	YES	-	-	-	-	89	YES	1.5	YES
	18010	Gate Pkwy. on-ramp merge - 5-lane section	4,011	4,104	93	2.3%	YES	-	-	-	-	93	YES	1.5	YES
	18011	Gate Pkwy. on-ramp merge - 4-lane section	4,011	4,155	144	3.6%	YES	-	-	-	-	144	YES	2.2	YES
S.R. 202 (JTB) Westbound - Ramps	18501	Kernan Blvd. Off-Ramp	116	112	-4	-3.0%	YES	-4	YES	-	-	-	-	0.3	YES
	18505	Kernan Blvd. On-Ramp	1,168	1,169	1	0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	18506	Gate Pkwy. Off-Ramp	1,104	1,156	52	4.7%	YES	-	-	5%	YES	-	-	1.5	YES
	18511	Gate Pkwy. On-Ramp	546	551	5	0.8%	YES	5	YES	-	-	-	-	0.2	YES

TABLE A1 CONTD.: Hourly flow comparisons – AM peak period - hour 1

Roadway	VISSIM Link Number	Location	Peak Hour Count Volume (vph)	VISSIM Model Volume (vph)	Difference	Within 15%	Volume Criteria Met	Individual Link Flow: < 700 vph Within 100vph	< 700 vph Criteria Met	700 vph to 2,700 vph Within 15%	700 vph to 2,700 vph Criteria Met	> 2,700 vph Within 400vph	> 2,700 vph Criteria Met	GEH Statistic	GEH Criteria Met (<= 5)
US 1 (Philips Hwy.) Eastbound	4001	Begin Project to SR 115 (Southside Blvd.)	809	783	-26	-3.2%	YES	-	-	-3%	YES	-	-	0.9	YES
	4002		469	460	-9	-1.8%	YES	-9	YES	-	-	-	-	0.4	YES
	4006	SR 115 (Southside Blvd.) to I-295 SB off-ramp	751	741	-10	-1.3%	YES	-	-	-1%	YES	-	-	0.4	YES
	4007		751	750	-1	-0.2%	YES	-	-	0%	YES	-	-	0.1	YES
	4008		751	748	-3	-0.5%	YES	-	-	0%	YES	-	-	0.1	YES
	4009		706	697	-9	-1.2%	YES	-	-	-1%	YES	-	-	0.3	YES
	4011	I-295 SB off-ramp to Greenland Rd./I-295 NB off-ramp	1,033	1,040	7	0.7%	YES	-	-	1%	YES	-	-	0.2	YES
	4012		757	769	12	1.6%	YES	-	-	2%	YES	-	-	0.4	YES
	4015	Greenland Rd./I-295 NB off-ramp to Business Park Blvd.	1,155	1,164	9	0.8%	YES	-	-	1%	YES	-	-	0.3	YES
	4018	Business Park Blvd. to End Project	995	1,035	40	4.1%	YES	-	-	4%	YES	-	-	1.3	YES
US 1 (Philips Hwy.) Westbound	2001	Begin Project to Business Park Blvd.	956	963	7	0.8%	YES	-	-	1%	YES	-	-	0.2	YES
	2002		905	893	-12	-1.4%	YES	-	-	-1%	YES	-	-	0.4	YES
	2005	Business Park Blvd.to Greenland Rd./I-295 NB off-ramp	982	970	-12	-1.2%	YES	-	-	-1%	YES	-	-	0.4	YES
	2006		982	978	-4	-0.4%	YES	-	-	0%	YES	-	-	0.1	YES
	2010	Greenland Rd./I-295 NB off-ramp to I-295 SB off-ramp	1,412	1,427	15	1.1%	YES	-	-	1%	YES	-	-	0.4	YES
	2011		995	1,019	24	2.4%	YES	-	-	2%	YES	-	-	0.7	YES
	2013	I-295 SB off-ramp to SR 115 (Southside Blvd.)	1,226	1,259	33	2.7%	YES	-	-	3%	YES	-	-	0.9	YES
	2014		1,226	1,278	52	4.3%	YES	-	-	4%	YES	-	-	1.5	YES
	2018	SR 115 (Southside Blvd.) to End Project	943	956	13	1.4%	YES	-	-	1%	YES	-	-	0.4	YES
SR 152 (Baymeadows Rd.) Eastbound	6001	Begin Project to Pointmeadows Rd.	892	853	-39	-4.4%	YES	-	-	-4%	YES	-	-	1.3	YES
	6003	Pointmeadows Rd. to I-295 SB off-ramp	802	786	-16	-2.0%	YES	-	-	-2%	YES	-	-	0.6	YES
	6005	I-295 SB off-ramp to I-295 NB off-ramp	732	703	-29	-4.0%	YES	-	-	-4%	YES	-	-	1.1	YES
	6007		314	309	-5	-1.4%	YES	-5	YES	-	-	-	-	0.3	YES
	6008	I-295 NB off-ramp to R.G. E. Skinner Blvd.	417	385	-32	-7.7%	YES	-32	YES	-	-	-	-	1.6	YES
	6011	R.G.E. Skinner Blvd. to End Project	353	361	8	2.2%	YES	8	YES	-	-	-	-	0.4	YES
SR 152 (Baymeadows Rd.) Westbound	8001	Begin Project to R.G.E. Skinner Blvd.	218	216	-2	-1.0%	YES	-2	YES	-	-	-	-	0.2	YES
	8002		205	202	-3	-1.4%	YES	-3	YES	-	-	-	-	0.2	YES
	8005	R.G.E. Skinner Blvd. to I-295 NB off-ramp	340	335	-5	-1.4%	YES	-5	YES	-	-	-	-	0.3	YES
	8006	I-295 NB off-ramp to I-295 SB off-ramp	411	413	2	0.5%	YES	2	YES	-	-	-	-	0.1	YES
	8007		314	327	13	4.2%	YES	13	YES	-	-	-	-	0.7	YES
	8009	I-295 SB off-ramp to Pointmeadows Rd.	809	794	-15	-1.8%	YES	-	-	-2%	YES	-	-	0.5	YES
	8011	Pointmeadows Rd. to End Project	879	921	42	4.8%	YES	-	-	5%	YES	-	-	1.4	YES
Gate Pkwy. Eastbound	12001	Begin Project to Pointmeadows Rd.	424	415	-9	-2.2%	YES	-9	YES	-	-	-	-	0.5	YES
	12002		340	331	-9	-2.7%	YES	-9	YES	-	-	-	-	0.5	YES
	12004	Pointmeadows Rd. to I-295 SB off-ramp	513	499	-14	-2.7%	YES	-14	YES	-	-	-	-	0.6	YES
	12006	I-295 SB off-ramp to I-295 NB off-ramp	558	547	-11	-2.0%	YES	-11	YES	-	-	-	-	0.5	YES
	12007		212	210	-2	-0.8%	YES	-2	YES	-	-	-	-	0.1	YES
	12009	I-295 NB off-ramp to End Project	237	229	-8	-3.4%	YES	-8	YES	-	-	-	-	0.5	YES
	12010		237	244	7	2.8%	YES	7	YES	-	-	-	-	0.4	YES
Gate Pkwy. Westbound	14001	Begin Project to I-295 NB off-ramp	379	359	-20	-5.3%	YES	-20	YES	-	-	-	-	1.0	YES
	14002	I-295 NB off-ramp to I-295 SB off-ramp	706	765	59	8.3%	YES	-	-	8%	YES	-	-	2.2	YES
	14003		693	763	70	10.1%	YES	70	YES	-	-	-	-	2.6	YES
	14005	I-295 SB off-ramp to Pointmeadows Rd.	918	970	52	5.7%	YES	-	-	6%	YES	-	-	1.7	YES
	14006		841	903	62	7.4%	YES	-	-	7%	YES	-	-	2.1	YES
	14009	Pointmeadows Rd. to End Project	1,033	1,109	76	7.4%	YES	-	-	7%	YES	-	-	2.3	YES
Town Center Pkwy. Eastbound	20001	Begin Project to St. Johns Bluff Rd.	642	638	-4	-0.7%	YES	-4	YES	-	-	-	-	0.2	YES
	20002		456	455	-1	-0.2%	YES	-1	YES	-	-	-	-	0.0	YES
	20005	St. Johns Bluff Rd. to I-295	667	672	5	0.7%	YES	5	YES	-	-	-	-	0.2	YES
	20006		109	104	-5	-4.8%	YES	-5	YES	-	-	-	-	0.5	YES
	20010	I-295 to Eco Dr.	430	433	3	0.6%	YES	3	YES	-	-	-	-	0.1	YES
	20012		302	257	-45	-14.8%	YES	-45	YES	-	-	-	-	2.7	YES
	20014	Eco Dr. to End Project	205	204	-1	-0.6%	YES	-1	YES	-	-	-	-	0.1	YES
Town Center Pkwy. Westbound	22001	Begin Project to Eco Dr.	109	100	-9	-8.0%	YES	-9	YES	-	-	-	-	0.9	YES
	22003	Eco Dr. to I-295	263	249	-14	-5.4%	YES	-14	YES	-	-	-	-	0.9	YES
	22004		263	244	-19	-7.1%	YES	-19	YES	-	-	-	-	3.2	YES
	22006		109	103	-6	-5.4%	YES	-6	YES	-	-	-	-	0.6	YES
	22008	I-295 to St. Johns Bluff Rd.	770	745	-25	-3.2%	YES	-	-	-3%	YES	-	-	0.9	YES
	22011	St. Johns Bluff Rd. to End Project	1,110	1,133	23	2.1%	YES	-	-	2%	YES	-	-	0.7	YES
Gate Pkwy. Southbound	35001	Begin Project to Town Center Pkwy.	629	635	6	0.9%	YES	6	YES	-	-	-	-	0.2	YES
	35002		398	402	4	1.1%	YES	4	YES	-	-	-	-	0.2	YES
	35004	Town Center Pkwy. to Costco/Skinner Lake Dr.	1,014	1,007	-7	-0.7%	YES	-	-	-1%	YES	-	-	0.2	YES
	35005		976	982	6	0.6%	YES	-	-	1%	YES	-	-	0.2	YES
	35007	Costco/Skinner Lake Dr. to I-295	1,110	1,113	3	0.2%	YES	-	-	0%	YES	-	-	0.1	YES
	35008	I-295 to Deerwood Park Blvd.	757	766	9	1.2%	YES	-	-	1%	YES	-	-	0.3	YES
	35011		1,341	1,367	26	2.0%	YES	-	-	2%	YES	-	-	0.7	YES
	35012	Deerwood Park Blvd. to End Project	533	550	17	3.2%	YES	17	YES	-	-	-	-	0.7	YES
Gate Pkwy. Northbound	33001	Begin Project to Deerwood Park Blvd.	834	732	-102	-12.2%	YES	-	-	-12%	YES	-	-	3.6	YES
	33003	Deerwood Park Blvd. to I-295	764	771	7	0.9%	YES	-	-	1%	YES	-	-	0.3	YES
	33004	I-295 to Costco/Skinner Lake Dr.	411	411	0	-0.1%	YES	0	YES	-	-	-	-	0.0	YES
	33007		783	720	-63	-8.1%	YES	-	-	-8%	YES	-	-	2.3	YES
	33011	Costco/Skinner Lake Dr. to Town Center Pkwy.	1,239	1,259	20	1.6%	YES	-	-	2%	YES	-	-	0.6	YES
	33014	Town Center Pkwy. to End Project	1,431	1,489	58	4.1%	YES	-	-	4%	YES	-	-	1.5	YES
Kernan Blvd. Southbound	39001	Begin Project to 1st Tech Pkwy.	1,290	1,299	9	0.7%	YES	-	-	1%	YES	-	-	0.3	YES
	39003	1st Tech Pkwy. to SR 202 (JTB)	218	311	93	42.9%	NO	93	YES	-	-	-	-	5.7	NO
	39008	SR 202 (JTB) to End. Project	13	12	-1	-10.2%	YES	-1	YES	-	-	-	-	0.4	YES
Kernan Blvd. Northbound	37001	Begin Project to SR 202 (JTB)	13	11	-2	-15.8%	NO	-2	YES	-	-	-	-	0.6	YES
	37005	SR 202 (JTB) to 1st Tech Pkwy.	481	532	51	10.5%	YES	51	YES	-	-	-	-	2.2	YES
	37007	1st Tech Pkwy. To End Project	494	512	18	3.6%	YES	18	YES	-	-	-	-	0.8	YES
Total			275,841	283,058	7,217										
Sum of all Link Flows				2.62%											
Sum of all Link Flows (Flows within 5%)				YES											
Total Count						199	197	73	73	92	92	34	34	199	196
Individual Links						99%		100%		100%		100%		98%	
Individual Links (Flows met for >85% Cases and GEH Statistic < 5 for 85% of Cases)						YES		YES		YES		YES		YES	

TABLE A2: Hourly flow comparisons – AM peak period - hour 2

Roadway	VISSIM Link Number	Location	Peak Hour Count Volume (vph)	VISSIM Model Volume (vph)	Difference	Within 15%	Criteria Met	Individual Link Flow < 700 vph Within 100vph	< 700 vph Criteria Met	700 vph to 2,700 vph Within 15%	700 vph to 2,700 vph Criteria Met	> 2,700 vph Within 400vph	> 2,700 vph Criteria Met	GEH Statistic	GEH Criteria Met (<= 5)
I-295 Northbound	1001	Upstream of I-95 on-ramps	2,360	2,377	17	0.7%	YES	-	-	1%	YES	-	-	0.4	YES
	1002	Upstream of S.R. 5 (US 1) off-ramp	3,920	3,911	-9	-0.2%	YES	-	-	-	-	-9	YES	0.1	YES
	1003	Between S.R. 5 (US 1) off- and on-ramps	3,160	3,151	-9	-0.3%	YES	-	-	-	-	-9	YES	0.2	YES
	1004	S.R. 5 (US 1) on-ramp merge	3,680	3,661	-19	-0.5%	YES	-	-	-	-	-19	YES	0.3	YES
	1005	Downstream of S.R. 5 (US 1) on-ramp merge	3,680	3,673	-7	-0.2%	YES	-	-	-	-	-7	YES	0.1	YES
	1006	3-lane to 2-lane merge - North of S.R. 5 (US 1)	3,680	3,670	-10	-0.3%	YES	-	-	-	-	-10	YES	0.2	YES
	1007	Upstream of S.R. 9B on-ramp	3,680	3,620	-60	-1.6%	YES	-	-	-	-	-60	YES	1.0	YES
	1008	S.R. 9B on-ramp merge	4,660	4,327	-333	-7.1%	YES	-	-	-	-	-333	YES	5.0	YES
	1009	Downstream of S.R. 9B on-ramp merge - 3-lane section	4,660	4,073	-587	-12.6%	YES	-	-	-	-	-587	NO	8.9	NO
	1010	Downstream of S.R. 9B on-ramp merge - 2-lane section	4,660	4,024	-636	-13.6%	YES	-	-	-	-	-636	NO	9.6	NO
	1011	Upstream of S.R. 152 (Baymeadows Rd.) off-ramp	4,660	4,021	-639	-13.7%	YES	-	-	-	-	-639	NO	9.7	NO
	1012	Between S.R. 152 (Baymeadows Rd.) off- and on-ramps	4,260	3,675	-585	-13.7%	YES	-	-	-	-	-585	NO	9.3	NO
	1013	Between S.R. 152 on-ramp and Gate Pkwy. off-ramp	5,040	4,435	-605	-12.0%	YES	-	-	-	-	-605	NO	8.8	NO
	1014	Between Gate Pkwy. off- and on-ramps	4,110	3,593	-517	-12.6%	YES	-	-	-	-	-517	NO	8.3	NO
	1015	Between Gate Pkwy. on-ramp and S.R. 202 (JTB) off-ramp	5,030	4,484	-546	-10.9%	YES	-	-	-	-	-546	NO	7.9	NO
	1016	Downstream of of S.R. 202 (JTB) off-ramp - 3-lane section	2,630	2,339	-291	-11.1%	YES	-	-	-11%	YES	-	-	5.8	NO
	1017	Downstream of of S.R. 202 (JTB) off-ramp - 2-lane section	2,630	2,344	-286	-10.9%	YES	-	-	-11%	YES	-	-	5.7	NO
	1018	Eastbound S.R. 202 (JTB) on-ramp merge	3,390	3,086	-304	-9.0%	YES	-	-	-	-	-304	YES	5.3	NO
	1019	Downstream of Eastbound S.R. 202 (JTB) on-ramp merge	3,390	3,081	-309	-9.1%	YES	-	-	-	-	-309	YES	5.4	NO
	1020	Between WB S.R. 202 (JTB) on-ramp and Town Center Pkwy. off-ramp	4,060	3,749	-311	-7.7%	YES	-	-	-	-	-311	YES	5.0	YES
	1021	Between Town Center Pkwy. off- and on-ramps	3,110	2,870	-240	-7.7%	YES	-	-	-	-	-240	YES	4.4	YES
	1022	Town Center Pkwy. on-ramp merge	3,600	3,352	-248	-6.9%	YES	-	-	-	-	-248	YES	4.2	YES
I-295 Northbound - Ramps	1501	I-95 Southbound on-ramp	210	208	-2	-1.1%	YES	-2	YES	-	-	-	-	0.2	YES
	1502	I-95 Northbound on-ramp	1,350	1,339	-11	-0.8%	YES	-	-	-1%	YES	-	-	0.3	YES
	1503	I-95 on-ramp (combined) - 2-lane section	1,560	1,543	-17	-1.1%	YES	-	-	-1%	YES	-	-	0.4	YES
	1504	I-95 on-ramp (combined) - 1-lane section	1,560	1,536	-24	-1.5%	YES	-	-	-2%	YES	-	-	0.6	YES
	1505	US-1 Off-Ramp	760	759	-1	-0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	1513	US-1 On-Ramp	520	513	-7	-1.3%	YES	-7	YES	-	-	-	-	0.3	YES
	1514	Baymeadows Rd. Off-Ramp	400	343	-57	-14.2%	YES	-57	YES	-	-	-	-	2.9	YES
	1520	Baymeadows Rd. On-Ramp	780	770	-10	-1.2%	YES	-	-	-1%	YES	-	-	0.3	YES
	1521	Gate Pkwy. Off-Ramp	930	805	-125	-13.5%	YES	-	-	-13%	YES	-	-	4.3	YES
	1527	Gate Pkwy. On-Ramp	920	899	-21	-2.3%	YES	-	-	-2%	YES	-	-	0.7	YES
	1528	SR 202 (JTB) Off-Ramp (combined)	2,400	2,138	-262	-10.9%	YES	-	-	-11%	YES	-	-	5.5	NO
	1529	SR 202 (JTB) WB Off-Ramp	730	643	-87	-11.9%	YES	-	-	-12%	YES	-	-	3.3	YES
	1530	SR 202 (JTB) EB Off-Ramp	1,670	1,492	-178	-10.7%	YES	-	-	-11%	YES	-	-	4.5	YES
	1531	SR 202 (JTB) On-Ramp - 2-lane section	2,550	2,499	-51	-2.0%	YES	-	-	-2%	YES	-	-	1.0	YES
	1532	SR 202 (JTB) On-Ramp - 1-lane section	2,550	2,499	-51	-2.0%	YES	-	-	-2%	YES	-	-	1.0	YES
	1533	On-Ramp from WB SR 202 (JTB)	670	657	-13	-1.9%	YES	-13	YES	-	-	-	-	0.5	YES
	1534	On-Ramp from EB SR 202 (JTB)	760	751	-9	-1.2%	YES	-	-	-1%	YES	-	-	0.3	YES
	1535	Town Center Pkwy Off-Ramp	950	876	-74	-7.8%	YES	-	-	-8%	YES	-	-	2.4	YES
	1539	Town Center Pkwy On-Ramp	490	477	-13	-2.6%	YES	-13	YES	-	-	-	-	0.6	YES
I-295 Southbound	3001	Upstream of Town Center Pkwy. off-ramp	5,270	5,241	-29	-0.5%	YES	-	-	-	-	-29	YES	0.4	YES
	3002	Between Town Center Pkwy. off- and on-ramps	4,360	4,291	-69	-1.6%	YES	-	-	-	-	-69	YES	1.0	YES
	3003	Between Town Center Pkwy. On-ramp and S.R. 202 (JTB) off-ramp (combined)	4,980	4,880	-100	-2.0%	YES	-	-	-	-	-100	YES	1.4	YES
	3004	Between S.R. 202 (JTB) off-ramp (combined) and Westbound S.R. 202 (JTB) on-ramp	2,730	2,666	-64	-2.3%	YES	-	-	-	-	-64	YES	1.2	YES
	3005	Westbound S.R. 202 (JTB) on-ramp merge	4,610	4,475	-135	-2.9%	YES	-	-	-	-	-135	YES	2.0	YES
	3006	Downstream of Westbound S.R. 202 (JTB) on-ramp merge	4,610	4,375	-235	-5.1%	YES	-	-	-	-	-235	YES	3.5	YES
	3007	Between Eastbound S.R. 202 (JTB) on-ramp and Gate Pkwy. off-ramp	4,880	4,754	-126	-2.6%	YES	-	-	-	-	-126	YES	1.8	YES
	3008	Between Gate Pkwy off- and on- ramps - 4-lane section	4,260	4,136	-124	-2.9%	YES	-	-	-	-	-124	YES	1.9	YES
	3009	Between Gate Pkwy off- and on- ramps - 3-lane section	4,260	4,144	-116	-2.7%	YES	-	-	-	-	-116	YES	1.8	YES
	3010	Between Gate Pkwy off- and on- ramps - 2-lane section	4,260	4,123	-137	-3.2%	YES	-	-	-	-	-137	YES	2.1	YES
	3011	Between Gate Pkwy. on-ramp and S.R. 152 (Baymeadows Rd.) off-ramp	4,480	4,330	-150	-3.3%	YES	-	-	-	-	-150	YES	2.3	YES
	3012	Between S.R. 152 (Baymeadows Rd.) off- and on-ramps	3,620	3,484	-136	-3.8%	YES	-	-	-	-	-136	YES	2.3	YES
	3013	S.R. 152 (Baymeadows Rd.) on-ramp merge	3,970	3,764	-206	-5.2%	YES	-	-	-	-	-206	YES	3.3	YES
	3014	Downstream of S.R. 152 (Baymeadows Rd.) on-ramp merge	3,970	3,809	-161	-4.1%	YES	-	-	-	-	-161	YES	2.6	YES
	3015	Upstream of S.R. 9B off-ramp - 2-lane section	3,970	3,811	-159	-4.0%	YES	-	-	-	-	-159	YES	2.5	YES
	3016	Upstream of S.R. 9B off-ramp - 3-lane section	3,970	3,823	-147	-3.7%	YES	-	-	-	-	-147	YES	2.4	YES
	3017	Downstream of S.R. 9B off-ramp - 2-lane section	3,230	3,097	-133	-4.1%	YES	-	-	-	-	-133	YES	2.4	YES
	3018	Downstraem of S.R. 9B off-ramp - 3-lane section	3,230	3,094	-136	-4.2%	YES	-	-	-	-	-136	YES	2.4	YES
	3019	Upstream of S.R. 5 (US 1) off-ramp - 3-lane section	3,230	3,091	-139	-4.3%	YES	-	-	-	-	-139	YES	2.5	YES
	3020	Upstream of S.R. 5 (US 1) off-ramp - 4-lane section	3,230	3,083	-147	-4.6%	YES	-	-	-	-	-147	YES	2.6	YES
	3021	Between S.R. 5 (US 1) off- and on-ramps	2,360	2,260	-100	-4.2%	YES	-	-	-4%	YES	-	-	2.1	YES
	3022	Downstream of S.R. 5 (US 1) on-ramp	3,080	2,968	-112	-3.7%	YES	-	-	-	-	-112	YES	2.0	YES
	3023	Upstream of Northbound I-95 off-ramp	3,080	2,947	-133	-4.3%	YES	-	-	-	-	-133	YES	2.4	YES
	3024	Downstream of Northbound I-95 off-ramp	2,610	2,500	-110	-4.2%	YES	-	-	-4%	YES	-	-	2.2	YES
	3025	Upstream of Southbound I-95 off-ramp - 3-lane section	2,610	2,504	-106	-4.1%	YES	-	-	-4%	YES	-	-	2.1	YES
	3026	Upstream of Southbound I-95 off-ramp - 4-lane section	2,610	2,503	-107	-4.1%	YES	-	-	-4%	YES	-	-	2.1	YES
	3027	Downstream of Southbound I-95 off-ramp - 3-lane section	1,680	1,592	-88	-5.2%	YES	-	-	-5%	YES	-	-	2.2	YES
	3028	Downstream of Southbound I-95 off-ramp - 2-lane section	1,680	1,624	-56	-3.3%	YES	-	-	-3%	YES	-	-	1.4	YES
I-295 Southbound - Ramps	3501	Town Center Pkwy Off-Ramp	910	898	-12	-1.3%	YES	-	-	-1%	YES	-	-	0.4	YES
	3505	Town Center Pkwy On-Ramp	620	598	-22	-3.6%	YES	-22	YES	-	-	-	-	0.9	YES
	3506	SR 202 (JTB) Off-Ramp (combined)	2,250	2,204	-46	-2.0%	YES	-	-	-2%	YES	-	-	1.0	YES
	3507	SR 202 (JTB) WB On-Ramp	1,310	1,281	-29	-2.2%	YES	-	-	-2%	YES	-	-	0.8	YES
	3508	SR 202 (JTB) EB On-Ramp	940	923	-17	-1.8%	YES	-	-	-2%	YES	-	-	0.6	YES
	3509	On-Ramp from WB SR 202 (JTB)	1,880	1,838	-42	-2.2%	YES	-	-	-2%	YES	-	-	1.0	YES
	3510	On-Ramp from EB SR 202 (combined)	1,030	1,015	-15	-1.4%	YES	-	-	-1%	YES	-	-	0.5	YES
	3511	On-Ramp from EB SR 202 (JTB)	270	263	-7	-2.7%	YES	-7	YES	-	-	-	-	0.4	YES
	3512	Gate Pkwy. Off-Ramp	620	598	-22	-3.6%	YES	-22	YES	-	-	-	-	0.9	YES
	3516	Gate Pkwy. On-Ramp - 2-lane segment	220	214	-6	-2.6%	YES	-6	YES	-	-	-	-	0.4	YES
	3517	Gate Pkwy. On-Ramp - 1-lane segment	220	215	-5	-2.2%	YES	-5	YES	-	-	-	-	0.3	YES
	3518	Baymeadows Rd. Off-Ramp	860	833	-27	-3.1%	YES	-	-	-3%	YES	-	-	0.9	YES
	3523	Baymeadows Rd. On-Ramp	350	346	-4	-1.0%	YES	-4	YES	-	-	-	-	0.2	YES
	3524	US-1 Off-Ramp	870	828	-42	-4.9%	YES	-	-	-5%	YES	-	-	1.4	YES
	3530	US-1 On-Ramp - 2-lane segment	720	717	-3	-0.4%	YES	-	-	0%	YES	-	-	0.1	YES
	3531	US-1 On-Ramp - 1-lane segment	720	709	-11	-1.5%	YES	-	-	-1%	YES	-	-	0.4	YES
	3532	I-95 NB Off-Ramp	470	460	-10	-2.1%	YES	-10	YES	-	-	-	-	0.4	YES
	3533	I-95 SB Off-Ramp	930	910	-20	-2.1%	YES	-	-	-2%	YES	-	-	0.6	YES
S.R. 9B	17001	Northbound S.R. 9B	980	978	-2	-0.2%	YES	-	-	0%	YES	-	-	0.1	YES
	19001	Southbound S.R. 9B	740	716	-24	-3.2%	YES	-	-	-3%	YES	-	-	0.9	YES
S.R. 202 (JTB) Eastbound	16001	Upstream of Gate Pkwy. off-ramp - Section 1	3,830	3,809	-21	-0.5%	YES	-	-	-	-	-21	YES	0.3	YES
	16002	Upstream of Gate Pkwy. off-ramp - Section 2	3,830	3,797	-33	-0.9%	YES	-	-	-	-	-33	YES	0.5	YES
	16003	Between Gate Pkwy. off- and on-ramps	2,820	2,778	-42	-1.5%	YES	-	-	-	-	-42	YES	0.8	YES
	16004	Between Gate Pkwy. on-ramp and I-295 off-ramp	3,480	3,426	-54	-1.5%	YES	-	-	-	-	-54	YES	0.9	YES
	16005	Between I-295 off-ramp and Southbound I-295 on-ramp	2,450	2,410	-40	-1.6%	YES	-	-	-2%	YES	-	-	0.8	YES
	16006	Southbound I-295 on-ramp merge	3,390	3,312	-78	-2.3%	YES	-	-	-	-	-78	YES	1.4	YES
	16007	Downstream of Southbound I-295 on-ramp merge	3,390	3,239	-151	-4.4%	YES	-	-	-	-	-151	YES	2.6	YES
	16008	Between Northbound I-295 on-ramp and Kernan Blvd. off-ramp	5,060	4,823	-237	-4.7%	YES	-	-	-	-	-237	YES	3.4	YES
	16009	Between Kernan Blvd. off- and on- ramp - 4-lane section	4,380	4,169	-211	-4.8%	YES	-	-	-	-	-211	YES	3.2	YES
	16010	Between Kernan Blvd. off- and on- ramp - 3-lane section	4,380	4,159	-221	-5.0%	YES	-	-	-	-	-221	YES	3.4	YES
	16011	Kernan Blvd. on-ramp merge - 4-lane section	4,730	4,502	-228	-4.8%	YES	-	-	-	-	-228	YES	3.4	YES
	16012	Kernan Blvd. on-ramp merge - 3-lane section	4,730	4,430	-300	-6.3%	YES	-	-	-	-	-300	YES	4.4	YES
	16013	Downstream of Kernan Blvd. on-ramp	4,730	4,525	-205	-4.3%	YES	-	-	-	-	-205	YES	3.0	YES
S.R. 202 (JTB) Eastbound - Ramps	16501	Gate Pkwy. Off-Ramp	1,010	1,017	7	0.7%	YES	-	-	1%	YES	-	-	0.2	YES
	16505	Gate Pkwy. On-Ramp	660	656	-4	-0.7%	YES	-4	YES	-	-	-	-	0.2	YES
	16506	Kernan Blvd. Off-Ramp	680	644	-36	-5.3%	YES	-36	YES	-	-	-	-	1.4	YES
	16510	Kernan Blvd. On-Ramp	350	362	12	3.5%	YES	12	YES	-	-	-	-	0.7	YES
S.R. 202 (JTB) Westbound	18001	Upstream of Kernan Blvd. off-ramp - section 1	5,990	6,008	18	0.3%	YES	-	-	-	-	18	YES	0.2	YES
	18002	Upstream of Kernan Blvd. off-ramp - section 2	5,990	5,994	4	0.1%	YES	-	-	-	-	4	YES	0.1	YES
	18003	Between Kernan Blvd. off- and on-ramps	5,810	5,779	-31	-0.5%	YES	-	-	-	-	-31	YES	0.4	YES
	18004	Between Kernan Blvd. on-ramp and I-295 off-ramp (combined)	7,630	7,565	-65	-0.9%	YES	-	-	-	-	-65	YES	0.7	YES
	18005	Between I-295 off-ramp (combined) and Northbound I-295 on-ramp	5,080	5,057	-23	-0.4%	YES	-	-	-	-	-23	YES	0.3	YES
	18006	Northbound I-295 on-ramp merge - 5-lane section	5,810	5,672	-138	-2.4%	YES	-	-	-	-	-138	YES	1.8	YES
	18007	Northbound I-295 on-ramp merge - 4-lane section	5,810	5,668	-142	-2.4%	YES	-	-	-	-	-142	YES	1.9	YES
	18008	Between Southbound I-295 on-ramp and Gate Pkwy. Off-ramp	7,120	6,977	-143	-2.0%	YES	-	-	-	-	-143	YES	1.7	YES
	18009	Between Gate Pkwy. off- and on-ramps	5,400	5,288	-112	-2.1%	YES	-	-	-	-	-112	YES	1.5	YES
	18010	Gate Pkwy. on-ramp merge - 5-lane section	6,250	6,126	-124	-2.0%	YES	-	-	-	-	-124	YES	1.6	YES
	18011	Gate Pkwy. on-ramp merge - 4-lane section	6,250	6,192	-58	-0.9%	YES	-	-	-	-	-58	YES	0.7	YES
S.R. 202 (JTB) Westbound - Ramps	18501	Kernan Blvd. Off-Ramp	180	183	3	1.7%	YES	3	YES	-	-	-	-	0.2	YES
	18505	Kernan Blvd. On-Ramp	1,820	1,782	-38	-2.1%	YES	-	-	-2%	YES	-	-	0.9	YES
	18506	Gate Pkwy. Off-Ramp	1,720	1,661	-59	-3.5%	YES	-	-	-3%	YES	-	-	1.4	YES
	18511	Gate Pkwy. On-Ramp	850	847	-3	-0.4%	YES	-	-	0%	YES	-	-	0.1	YES

TABLE A2 CONTD.: Hourly flow comparisons – AM peak period - hour 2

Roadway	VISSIM Link Number	Location	Peak Hour Count Volume (vph)	VISSIM Model Volume (vph)	Difference	Within 15%	Criteria Met	Individual Link Flow: < 700 vph Within 100vph	< 700 vph Criteria Met	700 vph to 2,700 vph Within 15%	700 vph to 2,700 vph Criteria Met	> 2,700 vph Within 400vph	> 2,700 vph Criteria Met	GEH Statistic	GEH Criteria Met (<= 5)
US 1 (Philips Hwy.) Eastbound	4001	Begin Project to SR 115 (Southside Blvd.)	1,260	1,234	-26	-2.1%	YES	-	-	-2%	YES	-	-	0.7	YES
	4002		730	727	-3	-0.4%	YES	-	-	0%	YES	-	-	0.1	YES
	4006	SR 115 (Southside Blvd.) to I-295 SB off-ramp	1,170	1,166	-4	-0.3%	YES	-	-	0%	YES	-	-	0.1	YES
	4007		1,370	1,371	1	0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	4008		1,170	1,170	0	0.0%	YES	-	-	0%	YES	-	-	0.0	YES
	4009		1,100	1,089	-11	-1.0%	YES	-	-	-1%	YES	-	-	0.3	YES
	4011	I-295 SB off-ramp to Greenland Rd./I-295 NB off-ramp	1,610	1,586	-24	-1.5%	YES	-	-	-1%	YES	-	-	0.6	YES
	4012		1,180	1,147	-33	-2.8%	YES	-	-	-3%	YES	-	-	1.0	YES
	4015	Greenland Rd./I-295 NB off-ramp to Business Park Blvd.	1,800	1,744	-56	-3.1%	YES	-	-	-3%	YES	-	-	1.3	YES
	4018	Business Park Blvd. to End Project	1,550	1,528	-22	-1.4%	YES	-	-	-1%	YES	-	-	0.6	YES
US 1 (Philips Hwy.) Westbound	2001	Begin Project to Business Park Blvd.	1,490	1,481	-9	-0.6%	YES	-	-	-1%	YES	-	-	0.2	YES
	2002		1,410	1,379	-31	-2.2%	YES	-	-	-2%	YES	-	-	0.8	YES
	2005	Business Park Blvd.to Greenland Rd./I-295 NB off-ramp	1,530	1,507	-23	-1.5%	YES	-	-	-2%	YES	-	-	0.6	YES
	2006		1,530	1,524	-6	-0.4%	YES	-	-	0%	YES	-	-	0.1	YES
	2010	Greenland Rd./I-295 NB off-ramp to I-295 SB off-ramp	2,200	2,136	-64	-2.9%	YES	-	-	-3%	YES	-	-	1.4	YES
	2011		1,550	1,536	-14	-0.9%	YES	-	-	-1%	YES	-	-	0.4	YES
	2013	I-295 SB off-ramp to SR 115 (Southside Blvd.)	1,910	1,852	-58	-3.1%	YES	-	-	-3%	YES	-	-	1.3	YES
	2014		1,910	1,854	-56	-3.0%	YES	-	-	-3%	YES	-	-	1.3	YES
	2018	SR 115 (Southside Blvd.) to End Project	1,470	1,419	-51	-3.5%	YES	-	-	-3%	YES	-	-	1.3	YES
SR 152 (Baymeadows Rd.) Eastbound	6001	Begin Project to Pointmeadows Rd.	1,390	1,323	-67	-4.8%	YES	-	-	-5%	YES	-	-	1.8	YES
	6003	Pointmeadows Rd. to I-295 SB off-ramp	1,250	1,225	-25	-2.0%	YES	-	-	-2%	YES	-	-	0.7	YES
	6005	I-295 SB off-ramp to I-295 NB off-ramp	1,140	1,079	-61	-5.4%	YES	-	-	-5%	YES	-	-	1.8	YES
	6007		490	484	-6	-1.1%	YES	-6	YES	-	-	-	-	0.3	YES
	6008	I-295 NB off-ramp to R.G. E. Skinner Blvd.	650	564	-86	-13.2%	YES	-86	YES	-	-	-	-	3.5	YES
	6011	R.G.E. Skinner Blvd. to End Project	550	527	-23	-4.3%	YES	-23	YES	-	-	-	-	1.0	YES
SR 152 (Baymeadows Rd.) Westbound	8001	Begin Project to R.G.E. Skinner Blvd.	340	334	-6	-1.7%	YES	-6	YES	-	-	-	-	0.3	YES
	8002		320	313	-7	-2.1%	YES	-7	YES	-	-	-	-	0.4	YES
	8005	R.G.E. Skinner Blvd. to I-295 NB off-ramp	530	512	-18	-3.3%	YES	-18	YES	-	-	-	-	0.8	YES
	8006	I-295 NB off-ramp to I-295 SB off-ramp	640	579	-61	-9.5%	YES	-61	YES	-	-	-	-	2.5	YES
	8007		490	443	-47	-9.5%	YES	-47	YES	-	-	-	-	2.2	YES
	8009	I-295 SB off-ramp to Pointmeadows Rd.	1,260	1,111	-149	-11.8%	YES	-	-	-12%	YES	-	-	4.3	YES
	8011	Pointmeadows Rd. to End Project	1,370	1,307	-63	-4.6%	YES	-	-	-5%	YES	-	-	1.7	YES
Gate Pkwy. Eastbound	12001	Begin Project to Pointmeadows Rd.	660	646	-14	-2.1%	YES	-14	YES	-	-	-	-	0.5	YES
	12002		530	509	-21	-4.0%	YES	-21	YES	-	-	-	-	0.9	YES
	12004	Pointmeadows Rd. to I-295 SB off-ramp	800	761	-39	-4.9%	YES	-	-	-5%	YES	-	-	1.4	YES
	12006	I-295 SB off-ramp to I-295 NB off-ramp	870	824	-46	-5.2%	YES	-	-	-5%	YES	-	-	1.6	YES
	12007		330	315	-15	-4.5%	YES	-15	YES	-	-	-	-	0.8	YES
	12009	I-295 NB off-ramp to End Project	370	337	-33	-8.9%	YES	-33	YES	-	-	-	-	1.8	YES
	12010		370	355	-15	-4.0%	YES	-15	YES	-	-	-	-	0.8	YES
Gate Pkwy. Westbound	14001	Begin Project to I-295 NB off-ramp	590	556	-34	-5.8%	YES	-34	YES	-	-	-	-	1.4	YES
	14002	I-295 NB off-ramp to I-295 SB off-ramp	1,100	947	-153	-13.9%	YES	-	-	-14%	YES	-	-	4.8	YES
	14003		1,080	941	-139	-12.9%	YES	-	-	-13%	YES	-	-	4.4	YES
	14005	I-295 SB off-ramp to Pointmeadows Rd.	1,430	1,252	-178	-12.4%	YES	-	-	-12%	YES	-	-	4.9	YES
	14006		1,310	1,164	-146	-11.1%	YES	-	-	-11%	YES	-	-	4.1	YES
	14009	Pointmeadows Rd. to End Project	1,610	1,487	-123	-7.6%	YES	-	-	-8%	YES	-	-	3.1	YES
Town Center Pkwy. Eastbound	20001	Begin Project to St. Johns Bluff Rd.	1,000	994	-6	-0.6%	YES	-	-	-1%	YES	-	-	0.2	YES
	20002		710	699	-11	-1.5%	YES	-	-	-1%	YES	-	-	0.4	YES
	20005	St. Johns Bluff Rd. to I-295	1,040	1,023	-17	-1.6%	YES	-	-	-2%	YES	-	-	0.5	YES
	20006		170	172	2	1.3%	YES	2	YES	-	-	-	-	0.2	YES
	20010	I-295 to Eco Dr.	670	650	-20	-3.0%	YES	-20	YES	-	-	-	-	0.8	YES
	20012		470	388	-82	-17.4%	NO	-82	YES	-	-	-	-	4.0	YES
	20014	Eco Dr. to End Project	320	321	1	0.2%	YES	1	YES	-	-	-	-	0.0	YES
Town Center Pkwy. Westbound	22001	Begin Project to Eco Dr.	170	162	-8	-4.9%	YES	-8	YES	-	-	-	-	0.6	YES
	22003	Eco Dr. to I-295	410	399	-11	-2.8%	YES	-11	YES	-	-	-	-	0.6	YES
	22004		410	389	-21	-5.1%	YES	-21	YES	-	-	-	-	1.1	YES
	22006		170	166	-4	-2.6%	YES	-4	YES	-	-	-	-	0.3	YES
	22008	I-295 to St. Johns Bluff Rd.	1,200	1,084	-116	-9.6%	YES	-	-	-10%	YES	-	-	3.4	YES
	22011	St. Johns Bluff Rd. to End Project	1,730	1,683	-47	-2.7%	YES	-	-	-3%	YES	-	-	1.1	YES
Gate Pkwy. Southbound	35001	Begin Project to Town Center Pkwy.	980	988	8	0.8%	YES	-	-	1%	YES	-	-	0.3	YES
	35002		620	616	-4	-0.6%	YES	-4	YES	-	-	-	-	0.1	YES
	35004	Town Center Pkwy. to Costco/Skinner Lake Dr.	1,580	1,504	-76	-4.8%	YES	-	-	-5%	YES	-	-	1.9	YES
	35005		1,520	1,471	-49	-3.3%	YES	-	-	-3%	YES	-	-	1.3	YES
	35007	Costco/Skinner Lake Dr. to I-295	1,730	1,678	-52	-3.0%	YES	-	-	-3%	YES	-	-	1.3	YES
	35008	I-295 to Deerwood Park Blvd.	1,180	1,139	-41	-3.5%	YES	-	-	-4%	YES	-	-	1.2	YES
	35011		2,090	2,007	-83	-4.0%	YES	-	-	-4%	YES	-	-	1.8	YES
	35012	Deerwood Park Blvd. to End Project	830	811	-19	-2.3%	YES	-	-	-2%	YES	-	-	0.7	YES
Gate Pkwy. Northbound	33001	Begin Project to Deerwood Park Blvd.	1,300	1,140	-160	-12.3%	YES	-	-	-12%	YES	-	-	4.6	YES
	33003	Deerwood Park Blvd. to I-295	1,190	1,182	-8	-0.6%	YES	-	-	-1%	YES	-	-	0.2	YES
	33004	I-295 to Costco/Skinner Lake Dr.	640	611	-29	-4.6%	YES	-29	YES	-	-	-	-	1.2	YES
	33007		1,220	1,099	-121	-9.9%	YES	-	-	-10%	YES	-	-	3.6	YES
	33011	Costco/Skinner Lake Dr. to Town Center Pkwy.	1,930	1,871	-59	-3.0%	YES	-	-	-3%	YES	-	-	1.3	YES
	33014	Town Center Pkwy. to End Project	2,230	2,176	-55	-2.4%	YES	-	-	-2%	YES	-	-	1.2	YES
Kernan Blvd. Southbound	39001	Begin Project to 1st Tech Pkwy.	2,010	1,998	-12	-0.6%	YES	-	-	-1%	YES	-	-	0.3	YES
	39003	1st Tech Pkwy. to SR 202 (JTB)	340	483	143	42.0%	NO	143	NO	-	-	-	-	7.0	NO
	39008	SR 202 (JTB) to End. Project	20	17	-3	-13.5%	YES	-3	YES	-	-	-	-	0.6	YES
Kernan Blvd. Northbound	37001	Begin Project to SR 202 (JTB)	20	15	-5	-23.4%	NO	-5	YES	-	-	-	-	1.1	YES
	37005	SR 202 (JTB) to 1st Tech Pkwy.	750	778	28	3.7%	YES	-	-	4%	YES	-	-	1.0	YES
	37007	1st Tech Pkwy. To End Project	770	748	-22	-2.9%	YES	-	-	-3%	YES	-	-	0.8	YES
Total			429,790	412,648	-17,142										
Sum of all Link Flows				-3.99%											
Sum of all Link Flows (Flows within 5%)				YES											
Total Count						199	196	43	42	92	92	64	57	199	186
Individual Links						98%		98%		100%		89%		93%	
Individual Links (Flows met for >85% Cases and GEH Statistic < 5 for 85% of Cases)						YES		YES		YES		YES		YES	

TABLE A3: Hourly flow comparisons – AM peak period - hour 3

Roadway	VISSIM Link Number	Location	Peak Hour Count Volume (vph)	VISSIM Model Volume (vph)	Difference	Within 15%	Criteria Met	Individual Link Flow: < 700 vph Within 100vph	< 700 vph Criteria Met	700 vph to 2,700 vph Within 15%	700 vph to 2,700 vph Criteria Met	> 2,700 vph Within 400vph	> 2,700 vph Criteria Met	GEH Statistic	GEH Criteria Met (<= 5)
I-295 Northbound	1001	Upstream of I-95 on-ramps	2,052	2,066	14	0.7%	YES	-	-	1%	YES	-	-	0.3	YES
	1002	Upstream of S.R. 5 (US 1) off-ramp	3,408	3,425	17	0.5%	YES	-	-	-	-	17	YES	0.3	YES
	1003	Between S.R. 5 (US 1) off- and on-ramps	2,748	2,759	11	0.4%	YES	-	-	-	-	11	YES	0.2	YES
	1004	S.R. 5 (US 1) on-ramp merge	3,200	3,220	20	0.6%	YES	-	-	-	-	20	YES	0.4	YES
	1005	Downstream of S.R. 5 (US 1) on-ramp merge	3,200	3,224	24	0.7%	YES	-	-	-	-	24	YES	0.4	YES
	1006	3-lane to 2-lane merge - North of S.R. 5 (US 1)	3,200	3,220	20	0.6%	YES	-	-	-	-	20	YES	0.4	YES
	1007	Upstream of S.R. 9B on-ramp	3,200	3,209	9	0.3%	YES	-	-	-	-	9	YES	0.2	YES
	1008	S.R. 9B on-ramp merge	4,052	4,060	8	0.2%	YES	-	-	-	-	8	YES	0.1	YES
	1009	Downstream of S.R. 9B on-ramp merge - 3-lane section	4,052	4,063	11	0.3%	YES	-	-	-	-	11	YES	0.2	YES
	1010	Downstream of S.R. 9B on-ramp merge - 2-lane section	4,052	4,063	11	0.3%	YES	-	-	-	-	11	YES	0.2	YES
	1011	Upstream of S.R. 152 (Baymeadows Rd.) off-ramp	4,052	4,058	6	0.2%	YES	-	-	-	-	6	YES	0.1	YES
	1012	Between S.R. 152 (Baymeadows Rd.) off- and on-ramps	3,704	3,719	15	0.4%	YES	-	-	-	-	15	YES	0.2	YES
	1013	Between S.R. 152 on-ramp and Gate Pkwy. off-ramp	4,382	4,413	31	0.7%	YES	-	-	-	-	31	YES	0.5	YES
	1014	Between Gate Pkwy. off- and on-ramps	3,574	3,597	23	0.6%	YES	-	-	-	-	23	YES	0.4	YES
	1015	Between Gate Pkwy. on-ramp and S.R. 202 (JTB) off-ramp	4,374	4,401	27	0.6%	YES	-	-	-	-	27	YES	0.4	YES
	1016	Downstream of of S.R. 202 (JTB) off-ramp - 3-lane section	2,287	2,301	14	0.6%	YES	-	-	1%	YES	-	-	0.3	YES
	1017	Downstream of of S.R. 202 (JTB) off-ramp - 2-lane section	2,287	2,307	20	0.9%	YES	-	-	1%	YES	-	-	0.4	YES
	1018	Eastbound S.R. 202 (JTB) on-ramp merge	2,948	2,980	32	1.1%	YES	-	-	-	-	32	YES	0.6	YES
	1019	Downstream of Eastbound S.R. 202 (JTB) on-ramp merge	2,948	2,980	32	1.1%	YES	-	-	-	-	32	YES	0.6	YES
	1020	Between WB S.R. 202 (JTB) on-ramp and Town Center Pkwy. off-ramp	3,530	3,587	57	1.6%	YES	-	-	-	-	57	YES	1.0	YES
	1021	Between Town Center Pkwy. off- and on-ramps	2,704	2,759	55	2.0%	YES	-	-	-	-	55	YES	1.1	YES
	1022	Town Center Pkwy. on-ramp merge	3,130	3,198	68	2.2%	YES	-	-	-	-	68	YES	1.2	YES
I-295 Northbound - Ramps	1501	I-95 Southbound on-ramp	183	179	-4	-2.4%	YES	-4	YES	-	-	-	-	0.3	YES
	1502	I-95 Northbound on-ramp	1,174	1,175	1	0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	1503	I-95 on-ramp (combined) - 2-lane section	1,356	1,358	2	0.1%	YES	-	-	0%	YES	-	-	0.1	YES
	1504	I-95 on-ramp (combined) - 1-lane section	1,356	1,353	-3	-0.2%	YES	-	-	0%	YES	-	-	0.1	YES
	1505	US-1 Off-Ramp	661	674	13	2.0%	YES	13	YES	-	-	-	-	0.5	YES
	1513	US-1 On-Ramp	452	466	14	3.2%	YES	14	YES	-	-	-	-	0.7	YES
	1514	Baymeadows Rd. Off-Ramp	348	339	-9	-2.5%	YES	-9	YES	-	-	-	-	0.5	YES
	1520	Baymeadows Rd. On-Ramp	678	691	13	2.0%	YES	13	YES	-	-	-	-	0.5	YES
	1521	Gate Pkwy. Off-Ramp	809	801	-8	-1.0%	YES	-	-	-1%	YES	-	-	0.3	YES
	1527	Gate Pkwy. On-Ramp	800	807	7	0.8%	YES	-	-	1%	YES	-	-	0.2	YES
	1528	SR 202 (JTB) Off-Ramp (combined)	2,087	2,099	12	0.6%	YES	-	-	1%	YES	-	-	0.3	YES
	1529	SR 202 (JTB) WB Off-Ramp	635	641	6	0.9%	YES	6	YES	-	-	-	-	0.2	YES
	1530	SR 202 (JTB) EB Off-Ramp	1,452	1,457	5	0.4%	YES	-	-	0%	YES	-	-	0.1	YES
	1531	SR 202 (JTB) On-Ramp - 2-lane section	2,217	2,240	23	1.1%	YES	-	-	1%	YES	-	-	0.5	YES
	1532	SR 202 (JTB) On-Ramp - 1-lane section	2,217	2,243	26	1.2%	YES	-	-	1%	YES	-	-	0.6	YES
	1533	On-Ramp from WB SR 202 (JTB)	583	589	6	1.0%	YES	6	YES	-	-	-	-	0.2	YES
	1534	On-Ramp from EB SR 202 (JTB)	661	680	19	2.9%	YES	19	YES	-	-	-	-	0.7	YES
	1535	Town Center Pkwy Off-Ramp	826	827	1	0.2%	YES	-	-	0%	YES	-	-	0.0	YES
	1539	Town Center Pkwy On-Ramp	426	426	0	-0.1%	YES	0	YES	-	-	-	-	0.0	YES
I-295 Southbound	3001	Upstream of Town Center Pkwy. off-ramp	4,582	4,609	27	0.6%	YES	-	-	-	-	27	YES	0.4	YES
	3002	Between Town Center Pkwy. off- and on-ramps	3,791	3,849	58	1.5%	YES	-	-	-	-	58	YES	0.9	YES
	3003	Between Town Center Pkwy. On-ramp and S.R. 202 (JTB) off-ramp (combined)	4,330	4,400	70	1.6%	YES	-	-	-	-	70	YES	1.1	YES
	3004	Between S.R. 202 (JTB) off-ramp (combined) and Westbound S.R. 202 (JTB) on-ramp	2,374	2,415	41	1.7%	YES	-	-	2%	YES	-	-	0.8	YES
	3005	Westbound S.R. 202 (JTB) on-ramp merge	4,008	4,060	52	1.3%	YES	-	-	-	-	52	YES	0.8	YES
	3006	Downstream of Westbound S.R. 202 (JTB) on-ramp merge	4,008	3,975	-33	-0.8%	YES	-	-	-	-	-33	YES	0.5	YES
	3007	Between Eastbound S.R. 202 (JTB) on-ramp and Gate Pkwy. off-ramp	4,243	4,325	82	1.9%	YES	-	-	-	-	82	YES	1.3	YES
	3008	Between Gate Pkwy off- and on- ramps - 4-lane section	3,704	3,767	63	1.7%	YES	-	-	-	-	63	YES	1.0	YES
	3009	Between Gate Pkwy off- and on- ramps - 3-lane section	3,704	3,797	93	2.5%	YES	-	-	-	-	93	YES	1.5	YES
	3010	Between Gate Pkwy off- and on- ramps - 2-lane section	3,704	3,806	102	2.8%	YES	-	-	-	-	102	YES	1.7	YES
	3011	Between Gate Pkwy. on-ramp and S.R. 152 (Baymeadows Rd.) off-ramp	3,895	4,029	134	3.4%	YES	-	-	-	-	134	YES	2.1	YES
	3012	Between S.R. 152 (Baymeadows Rd.) off- and on-ramps	3,148	3,278	130	4.1%	YES	-	-	-	-	130	YES	2.3	YES
	3013	S.R. 152 (Baymeadows Rd.) on-ramp merge	3,452	3,535	83	2.4%	YES	-	-	-	-	83	YES	1.4	YES
	3014	Downstream of S.R. 152 (Baymeadows Rd.) on-ramp merge	3,452	3,585	133	3.9%	YES	-	-	-	-	133	YES	2.2	YES
	3015	Upstream of S.R. 9B off-ramp - 2-lane section	3,452	3,596	144	4.2%	YES	-	-	-	-	144	YES	2.4	YES
	3016	Upstream of S.R. 9B off-ramp - 3-lane section	3,452	3,613	161	4.7%	YES	-	-	-	-	161	YES	2.7	YES
	3017	Downstraem of S.R. 9B off-ramp - 2-lane section	2,808	2,946	138	4.9%	YES	-	-	-	-	138	YES	2.6	YES
	3018	Downstraem of S.R. 9B off-ramp - 3-lane section	2,808	2,958	150	5.3%	YES	-	-	-	-	150	YES	2.8	YES
	3019	Upstream of S.R. 5 (US 1) off-ramp - 3-lane section	2,808	2,962	154	5.5%	YES	-	-	-	-	154	YES	2.9	YES
	3020	Upstream of S.R. 5 (US 1) off-ramp - 4-lane section	2,808	2,955	147	5.2%	YES	-	-	-	-	147	YES	2.7	YES
	3021	Between S.R. 5 (US 1) off- and on-ramps	2,052	2,167	115	5.6%	YES	-	-	6%	YES	-	-	2.5	YES
	3022	Downstream of S.R. 5 (US 1) on-ramp	2,678	2,823	145	5.4%	YES	-	-	5%	YES	-	-	2.8	YES
	3023	Upstream of Northbound I-95 off-ramp	2,678	2,813	135	5.0%	YES	-	-	5%	YES	-	-	2.6	YES
	3024	Downstream of Northbound I-95 off-ramp	2,269	2,388	119	5.2%	YES	-	-	5%	YES	-	-	2.5	YES
	3025	Upstream of Southbound I-95 off-ramp - 3-lane section	2,269	2,395	126	5.5%	YES	-	-	6%	YES	-	-	2.6	YES
	3026	Upstream of Southbound I-95 off-ramp - 4-lane section	2,269	2,394	125	5.5%	YES	-	-	6%	YES	-	-	2.6	YES
	3027	Downstream of Southbound I-95 off-ramp - 3-lane section	1,461	1,543	82	5.6%	YES	-	-	6%	YES	-	-	2.1	YES
	3028	Downstream of Southbound I-95 off-ramp - 2-lane section	1,461	1,576	115	7.9%	YES	-	-	8%	YES	-	-	3.0	YES
I-295 Southbound - Ramps	3501	Town Center Pkwy Off-Ramp	791	804	13	1.7%	YES	-	-	2%	YES	-	-	0.5	YES
	3505	Town Center Pkwy On-Ramp	539	537	-2	-0.4%	YES	-2	YES	-	-	-	-	0.1	YES
	3506	SR 202 (JTB) Off-Ramp (combined)	1,956	1,989	33	1.7%	YES	-	-	2%	YES	-	-	0.7	YES
	3507	SR 202 (JTB) WB On-Ramp	1,139	1,169	30	2.7%	YES	-	-	3%	YES	-	-	0.9	YES
	3508	SR 202 (JTB) EB On-Ramp	817	826	9	1.1%	YES	-	-	1%	YES	-	-	0.3	YES
	3509	On-Ramp from WB SR 202 (JTB)	1,635	1,660	25	1.5%	YES	-	-	2%	YES	-	-	0.6	YES
	3510	On-Ramp from EB SR 202 (combined)	896	921	25	2.8%	YES	-	-	3%	YES	-	-	0.8	YES
	3511	On-Ramp from EB SR 202 (JTB)	235	242	7	2.8%	YES	7	YES	-	-	-	-	0.4	YES
	3512	Gate Pkwy. Off-Ramp	539	554	15	2.8%	YES	15	YES	-	-	-	-	0.6	YES
	3516	Gate Pkwy. On-Ramp - 2-lane segment	191	191	0	-0.1%	YES	0	YES	-	-	-	-	0.0	YES
	3517	Gate Pkwy. On-Ramp - 1-lane segment	191	193	2	1.1%	YES	2	YES	-	-	-	-	0.2	YES
	3518	Baymeadows Rd. Off-Ramp	748	771	23	3.1%	YES	-	-	3%	YES	-	-	0.8	YES
	3523	Baymeadows Rd. On-Ramp	304	303	-1	-0.4%	YES	-1	YES	-	-	-	-	0.1	YES
	3524	US-1 Off-Ramp	756	795	39	5.1%	YES	-	-	5%	YES	-	-	1.4	YES
	3530	US-1 On-Ramp - 2-lane segment	626	647	21	3.3%	YES	21	YES	-	-	-	-	0.8	YES
	3531	US-1 On-Ramp - 1-lane segment	626	644	18	2.8%	YES	18	YES	-	-	-	-	0.7	YES
	3532	I-95 NB Off-Ramp	409	438	29	7.2%	YES	29	YES	-	-	-	-	1.4	YES
	3533	I-95 SB Off-Ramp	809	854	45	5.6%	YES	-	-	6%	YES	-	-	1.6	YES
S.R. 9B	17001	Northbound S.R. 9B	852	859	7	0.8%	YES	-	-	1%	YES	-	-	0.2	YES
	19001	Southbound S.R. 9B	643	677	34	5.3%	YES	34	YES	-	-	-	-	1.3	YES
S.R. 202 (JTB) Eastbound	16001	Upstream of Gate Pkwy. off-ramp - Section 1	3,330	3,311	-19	-0.6%	YES	-	-	-	-	-19	YES	0.3	YES
	16002	Upstream of Gate Pkwy. off-ramp - Section 2	3,330	3,312	-18	-0.5%	YES	-	-	-	-	-18	YES	0.3	YES
	16003	Between Gate Pkwy. off- and on-ramps	2,452	2,456	4	0.2%	YES	-	-	0%	YES	-	-	0.1	YES
	16004	Between Gate Pkwy. on-ramp and I-295 off-ramp	3,026	3,045	19	0.6%	YES	-	-	-	-	19	YES	0.3	YES
	16005	Between I-295 off-ramp and Southbound I-295 on-ramp	2,130	2,136	6	0.3%	YES	-	-	0%	YES	-	-	0.1	YES
	16006	Southbound I-295 on-ramp merge	2,948	2,951	3	0.1%	YES	-	-	-	-	3	YES	0.0	YES
	16007	Downstream of Southbound I-295 on-ramp merge	2,948	2,887	-61	-2.1%	YES	-	-	-	-	-61	YES	1.1	YES
	16008	Between Northbound I-295 on-ramp and Kernan Blvd. off-ramp	4,400	4,430	30	0.7%	YES	-	-	-	-	30	YES	0.4	YES
	16009	Between Kernan Blvd. off- and on- ramp - 4-lane section	3,808	3,823	15	0.4%	YES	-	-	-	-	15	YES	0.2	YES
	16010	Between Kernan Blvd. off- and on- ramp - 3-lane section	3,808	3,825	17	0.5%	YES	-	-	-	-	17	YES	0.3	YES
	16011	Kernan Blvd. on-ramp merge - 4-lane section	4,113	4,139	26	0.6%	YES	-	-	-	-	26	YES	0.4	YES
	16012	Kernan Blvd. on-ramp merge - 3-lane section	4,113	4,080	-33	-0.8%	YES	-	-	-	-	-33	YES	0.5	YES
	16013	Downstream of Kernan Blvd. on-ramp	4,113	4,182	69	1.7%	YES	-	-	-	-	69	YES	1.1	YES
S.R. 202 (JTB) Eastbound - Ramps	16501	Gate Pkwy. Off-Ramp	878	863	-15	-1.7%	YES	-	-	-2%	YES	-	-	0.5	YES
	16505	Gate Pkwy. On-Ramp	574	590	16	2.7%	YES	16	YES	-	-	-	-	0.6	YES
	16506	Kernan Blvd. Off-Ramp	591	602	11	1.8%	YES	11	YES	-	-	-	-	0.4	YES
	16510	Kernan Blvd. On-Ramp	304	319	15	5.0%	YES	15	YES	-	-	-	-	0.9	YES
S.R. 202 (JTB) Westbound	18001	Upstream of Kernan Blvd. off-ramp - section 1	5,208	5,220	12	0.2%	YES	-	-	-	-	12	YES	0.2	YES
	18002	Upstream of Kernan Blvd. off-ramp - section 2	5,208	5,233	25	0.5%	YES	-	-	-	-	25	YES	0.3	YES
	18003	Between Kernan Blvd. off- and on-ramps	5,052	5,078	26	0.5%	YES	-	-	-	-	26	YES	0.4	YES
	18004	Between Kernan Blvd. on-ramp and I-295 off-ramp (combined)	6,634	6,684	50	0.8%	YES	-	-	-	-	50	YES	0.6	YES
	18005	Between I-295 off-ramp (combined) and Northbound I-295 on-ramp	4,417	4,456	39	0.9%	YES	-	-	-	-	39	YES	0.6	YES
	18006	Northbound I-295 on-ramp merge - 5-lane section	5,052	5,085	33	0.7%	YES	-	-	-	-	33	YES	0.5	YES
	18007	Northbound I-295 on-ramp merge - 4-lane section	5,052	5,087	35	0.7%	YES	-	-	-	-	35	YES	0.5	YES
	18008	Between Southbound I-295 on-ramp and Gate Pkwy. Off-ramp	6,191	6,298	107	1.7%	YES	-	-	-	-	107	YES	1.4	YES
	18009	Between Gate Pkwy. off- and on-ramps	4,695	4,792	97	2.1%	YES	-	-	-	-	97	YES	1.4	YES
	18010	Gate Pkwy. on-ramp merge - 5-lane section	5,434	5,536	102	1.9%	YES	-	-	-	-	102	YES	1.4	YES
	18011	Gate Pkwy. on-ramp merge - 4-lane section	5,434	5,603	169	3.1%	YES	-	-	-	-	169	YES	2.3	YES
S.R. 202 (JTB) Westbound - Ramps	18501	Kernan Blvd. Off-Ramp	157	155	-2	-1.1%	YES	-2	YES	-	-	-	-	0.1	YES
	18505	Kernan Blvd. On-Ramp	1,582	1,577	-5	-0.3%	YES	-	-	0%	YES	-	-	0.1	YES
	18506	Gate Pkwy. Off-Ramp	1,496	1,518	22	1.5%	YES	-	-	1%	YES	-	-	0.6	YES
	18511	Gate Pkwy. On-Ramp	739	744	5	0.7%	YES	-	-	1%	YES	-	-	0.2	YES

TABLE A3 CONTD.: Hourly flow comparisons – AM peak period - hour 3

Roadway	VISSIM Link Number	Location	Peak Hour Count Volume (vph)	VISSIM Model Volume (vph)	Difference	Within 15%	Criteria Met	Individual Link Flow: < 700 vph: Within 100vph	< 700 vph: Criteria Met	700 vph to 2,700 vph: Within 15%	700 vph to 2,700 vph: Criteria Met	> 2,700 vph: Within 400vph	> 2,700 vph: Criteria Met	GEH Statistic	GEH Criteria Met (<= 5)
US 1 (Philips Hwy.) Eastbound	4001	Begin Project to SR 115 (Southside Blvd.)	1,096	1,078	-18	-1.6%	YES	-	-	-2%	YES	-	-	0.5	YES
	4002		635	626	-9	-1.5%	YES	-9	YES	-	-	-	-	0.4	YES
	4006	SR 115 (Southside Blvd.) to I-295 SB off-ramp	1,017	1,027	10	0.9%	YES	-	-	1%	YES	-	-	0.3	YES
	4007		1,017	1,032	15	1.5%	YES	-	-	2%	YES	-	-	0.5	YES
	4008		1,017	1,028	11	1.1%	YES	-	-	1%	YES	-	-	0.3	YES
	4009		956	954	-2	-0.2%	YES	-	-	0%	YES	-	-	0.1	YES
	4011	I-295 SB off-ramp to Greenland Rd./I-295 NB off-ramp	1,400	1,437	37	2.7%	YES	-	-	3%	YES	-	-	1.0	YES
	4012		1,026	1,058	32	3.1%	YES	-	-	3%	YES	-	-	1.0	YES
	4015	Greenland Rd./I-295 NB off-ramp to Business Park Blvd.	1,565	1,585	20	1.3%	YES	-	-	1%	YES	-	-	0.5	YES
	4018	Business Park Blvd. to End Project	1,348	1,389	41	3.1%	YES	-	-	3%	YES	-	-	1.1	YES
US 1 (Philips Hwy.) Westbound	2001	Begin Project to Business Park Blvd.	1,296	1,300	4	0.3%	YES	-	-	0%	YES	-	-	0.1	YES
	2002		1,226	1,209	-17	-1.4%	YES	-	-	-1%	YES	-	-	0.5	YES
	2005	Business Park Blvd.to Greenland Rd./I-295 NB off-ramp	1,330	1,318	-12	-0.9%	YES	-	-	-1%	YES	-	-	0.3	YES
	2006		1,330	1,330	0	0.0%	YES	-	-	0%	YES	-	-	0.0	YES
	2010	Greenland Rd./I-295 NB off-ramp to I-295 SB off-ramp	1,913	1,897	-16	-0.9%	YES	-	-	-1%	YES	-	-	0.4	YES
	2011		1,348	1,350	2	0.2%	YES	-	-	0%	YES	-	-	0.1	YES
	2013	I-295 SB off-ramp to SR 115 (Southside Blvd.)	1,661	1,642	-19	-1.2%	YES	-	-	-1%	YES	-	-	0.5	YES
	2014		1,661	1,682	21	1.2%	YES	-	-	1%	YES	-	-	0.5	YES
	2018	SR 115 (Southside Blvd.) to End Project	1,278	1,310	32	2.5%	YES	-	-	3%	YES	-	-	0.9	YES
SR 152 (Baymeadows Rd.) Eastbound	6001	Begin Project to Pointmeadows Rd.	1,209	1,152	-57	-4.7%	YES	-	-	-5%	YES	-	-	1.7	YES
	6003	Pointmeadows Rd. to I-295 SB off-ramp	1,087	1,077	-10	-0.9%	YES	-	-	-1%	YES	-	-	0.3	YES
	6005	I-295 SB off-ramp to I-295 NB off-ramp	991	960	-31	-3.1%	YES	-	-	-3%	YES	-	-	1.0	YES
	6007		426	427	1	0.3%	YES	1	YES	-	-	-	-	0.1	YES
	6008	I-295 NB off-ramp to R.G. E. Skinner Blvd.	565	517	-48	-8.4%	YES	-48	YES	-	-	-	-	2.1	YES
	6011	R.G.E. Skinner Blvd. to End Project	478	490	12	2.6%	YES	12	YES	-	-	-	-	0.6	YES
SR 152 (Baymeadows Rd.) Westbound	8001	Begin Project to R.G.E. Skinner Blvd.	296	298	2	0.8%	YES	2	YES	-	-	-	-	0.1	YES
	8002		278	277	-1	-0.2%	YES	-1	YES	-	-	-	-	0.0	YES
	8005	R.G.E. Skinner Blvd. to I-295 NB off-ramp	461	452	-9	-2.1%	YES	-9	YES	-	-	-	-	0.4	YES
	8006	I-295 NB off-ramp to I-295 SB off-ramp	556	532	-24	-4.2%	YES	-24	YES	-	-	-	-	1.0	YES
	8007		426	414	-12	-2.7%	YES	-12	YES	-	-	-	-	0.6	YES
	8009	I-295 SB off-ramp to Pointmeadows Rd.	1,096	1,044	-52	-4.7%	YES	-	-	-5%	YES	-	-	1.6	YES
	8011	Pointmeadows Rd. to End Project	1,191	1,220	29	2.4%	YES	-	-	2%	YES	-	-	0.8	YES
Gate Pkwy. Eastbound	12001	Begin Project to Pointmeadows Rd.	574	571	-3	-0.4%	YES	-3	YES	-	-	-	-	0.1	YES
	12002		461	455	-6	-1.4%	YES	-6	YES	-	-	-	-	0.3	YES
	12004	Pointmeadows Rd. to I-295 SB off-ramp	696	678	-18	-2.5%	YES	-18	YES	-	-	-	-	0.7	YES
	12006	I-295 SB off-ramp to I-295 NB off-ramp	756	750	-6	-0.9%	YES	-	-	-1%	YES	-	-	0.2	YES
	12007		287	293	6	2.1%	YES	6	YES	-	-	-	-	0.4	YES
	12009	I-295 NB off-ramp to End Project	322	314	-8	-2.6%	YES	-8	YES	-	-	-	-	0.5	YES
	12010		322	331	9	2.7%	YES	9	YES	-	-	-	-	0.5	YES
Gate Pkwy. Westbound	14001	Begin Project to I-295 NB off-ramp	513	489	-24	-4.7%	YES	-24	YES	-	-	-	-	1.1	YES
	14002	I-295 NB off-ramp to I-295 SB off-ramp	956	938	-18	-1.9%	YES	-	-	-2%	YES	-	-	0.6	YES
	14003		939	936	-3	-0.4%	YES	-	-	0%	YES	-	-	0.1	YES
	14005	I-295 SB off-ramp to Pointmeadows Rd.	1,243	1,225	-18	-1.5%	YES	-	-	-1%	YES	-	-	0.5	YES
	14006		1,139	1,138	-1	-0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	14009	Pointmeadows Rd. to End Project	1,400	1,421	21	1.5%	YES	-	-	2%	YES	-	-	0.6	YES
Town Center Pkwy. Eastbound	20001	Begin Project to St. Johns Bluff Rd.	870	870	0	0.0%	YES	-	-	0%	YES	-	-	0.0	YES
	20002		617	620	3	0.5%	YES	3	YES	-	-	-	-	0.1	YES
	20005	St. Johns Bluff Rd. to I-295	904	905	1	0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	20006		148	147	-1	-0.8%	YES	-1	YES	-	-	-	-	0.1	YES
	20010	I-295 to Eco Dr.	583	588	5	0.8%	YES	5	YES	-	-	-	-	0.2	YES
	20012		409	345	-64	-15.6%	NO	-64	YES	-	-	-	-	3.3	YES
	20014	Eco Dr. to End Project	278	277	-1	-0.5%	YES	-1	YES	-	-	-	-	0.1	YES
Town Center Pkwy. Westbound	22001	Begin Project to Eco Dr.	148	143	-5	-3.2%	YES	-5	YES	-	-	-	-	0.4	YES
	22003	Eco Dr. to I-295	356	350	-6	-1.7%	YES	-6	YES	-	-	-	-	0.3	YES
	22004		356	344	-12	-3.4%	YES	-12	YES	-	-	-	-	0.7	YES
	22006		148	146	-2	-1.4%	YES	-2	YES	-	-	-	-	0.2	YES
	22008	I-295 to St. Johns Bluff Rd.	1,043	1,001	-42	-4.0%	YES	-	-	-4%	YES	-	-	1.3	YES
	22011	St. Johns Bluff Rd. to End Project	1,504	1,523	19	1.3%	YES	-	-	1%	YES	-	-	0.5	YES
Gate Pkwy. Southbound	35001	Begin Project to Town Center Pkwy.	852	865	13	1.6%	YES	-	-	2%	YES	-	-	0.5	YES
	35002		539	545	6	1.2%	YES	6	YES	-	-	-	-	0.3	YES
	35004	Town Center Pkwy. to Costco/Skinner Lake Dr.	1,374	1,389	15	1.1%	YES	-	-	1%	YES	-	-	0.4	YES
	35005		1,322	1,359	37	2.8%	YES	-	-	3%	YES	-	-	1.0	YES
	35007	Costco/Skinner Lake Dr. to I-295	1,504	1,540	36	2.4%	YES	-	-	2%	YES	-	-	0.9	YES
	35008	I-295 to Deerwood Park Blvd.	1,026	1,061	35	3.4%	YES	-	-	3%	YES	-	-	1.1	YES
	35011		1,817	1,848	31	1.7%	YES	-	-	2%	YES	-	-	0.7	YES
	35012	Deerwood Park Blvd. to End Project	722	749	27	3.8%	YES	-	-	4%	YES	-	-	1.0	YES
Gate Pkwy. Northbound	33001	Begin Project to Deerwood Park Blvd.	1,130	995	-135	-11.9%	YES	-	-	-12%	YES	-	-	4.1	YES
	33003	Deerwood Park Blvd. to I-295	1,035	1,042	7	0.7%	YES	-	-	1%	YES	-	-	0.2	YES
	33004	I-295 to Costco/Skinner Lake Dr.	556	553	-3	-0.5%	YES	-3	YES	-	-	-	-	0.1	YES
	33007		1,061	968	-93	-8.8%	YES	-	-	-9%	YES	-	-	2.9	YES
	33011	Costco/Skinner Lake Dr. to Town Center Pkwy.	1,678	1,681	3	0.2%	YES	-	-	0%	YES	-	-	0.1	YES
	33014	Town Center Pkwy. to End Project	1,939	2,017	78	4.0%	YES	-	-	4%	YES	-	-	1.8	YES
Kernan Blvd. Southbound	39001	Begin Project to 1st Tech Pkwy.	1,748	1,750	2	0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	39003	1st Tech Pkwy. to SR 202 (JTB)	296	422	126	42.6%	NO	126	NO	-	-	-	-	6.7	NO
	39008	SR 202 (JTB) to End. Project	17	13	-4	-24.5%	NO	-4	YES	-	-	-	-	1.1	YES
Kernan Blvd. Northbound	37001	Begin Project to SR 202 (JTB)	17	13	-4	-21.4%	NO	-4	YES	-	-	-	-	0.9	YES
	37005	SR 202 (JTB) to 1st Tech Pkwy.	652	716	64	9.8%	YES	64	YES	-	-	-	-	2.4	YES
	37007	1st Tech Pkwy. To End Project	670	687	17	2.6%	YES	17	YES	-	-	-	-	0.7	YES
Total			373,699	378,431	4,732										
Sum of all Link Flows				1.27%											
Sum of all Link Flows (Flows within 5%)			YES												
Total Count						199	195	55	54	84	84	60	60	199	198
Individual Links						98%		98%		100%		100%		99%	
Individual Links (Flows met for >85% Cases and GEH Statistic < 5 for 85% of Cases)						YES		YES		YES		YES		YES	

TABLE A4: Hourly flow comparisons – PM peak period - hour 1

Roadway	VISSIM Link Number	Location	Peak Hour Count Volume (vph)	VISSIM Model Volume (vph)	Difference	Within 15%	Criteria Met	Individual Link Flow: < 700 vph		700 vph to 2,700 vph		> 2,700 vph		GEH Statistic	GEH Criteria Met (<= 5)
								Within 100vph	Criteria Met	Within 15%	Criteria Met	Within 400vph	Criteria Met		
I-295 Northbound	1001	Upstream of I-95 on-ramps	1,424	1,428	4	0.3%	YES	-	-	0%	YES	-	-	0.1	YES
	1002	Upstream of S.R. 5 (US 1) off-ramp	2,611	2,615	4	0.1%	YES	-	-	0%	YES	-	-	0.1	YES
	1003	Between S.R. 5 (US 1) off- and on-ramps	2,000	2,003	3	0.2%	YES	-	-	0%	YES	-	-	0.1	YES
	1004	S.R. 5 (US 1) on-ramp merge	2,738	2,728	-10	-0.4%	YES	-	-	-	-	-10	YES	0.2	YES
	1005	Downstream of S.R. 5 (US 1) on-ramp merge	2,738	2,726	-12	-0.4%	YES	-	-	-	-	-12	YES	0.2	YES
	1006	3-lane to 2-lane merge - North of S.R. 5 (US 1)	2,738	2,730	-8	-0.3%	YES	-	-	-	-	-8	YES	0.2	YES
	1007	Upstream of S.R. 9B on-ramp	2,738	2,741	3	0.1%	YES	-	-	-	-	3	YES	0.1	YES
	1008	S.R. 9B on-ramp merge	3,365	3,374	9	0.3%	YES	-	-	-	-	9	YES	0.2	YES
	1009	Downstream of S.R. 9B on-ramp merge - 3-lane section	3,365	3,357	-8	-0.2%	YES	-	-	-	-	-8	YES	0.1	YES
	1010	Downstream of S.R. 9B on-ramp merge - 2-lane section	3,365	3,367	2	0.1%	YES	-	-	-	-	2	YES	0.0	YES
	1011	Upstream of S.R. 152 (Baymeadows Rd.) off-ramp	3,365	3,375	10	0.3%	YES	-	-	-	-	10	YES	0.2	YES
	1012	Between S.R. 152 (Baymeadows Rd.) off- and on-ramps	3,068	3,081	13	0.4%	YES	-	-	-	-	13	YES	0.2	YES
	1013	Between S.R. 152 on-ramp and Gate Pkwy. off-ramp	3,797	3,821	24	0.6%	YES	-	-	-	-	24	YES	0.4	YES
	1014	Between Gate Pkwy. off- and on-ramps	3,611	3,634	23	0.6%	YES	-	-	-	-	23	YES	0.4	YES
	1015	Between Gate Pkwy. on-ramp and S.R. 202 (JTB) off-ramp	4,136	4,163	27	0.6%	YES	-	-	-	-	27	YES	0.4	YES
	1016	Downstream of of S.R. 202 (JTB) off-ramp - 3-lane section	2,314	2,323	9	0.4%	YES	-	-	0%	YES	-	-	0.2	YES
	1017	Downstream of of S.R. 202 (JTB) off-ramp - 2-lane section	2,314	2,336	22	0.9%	YES	-	-	1%	YES	-	-	0.5	YES
	1018	Eastbound S.R. 202 (JTB) on-ramp merge	3,424	3,494	70	2.0%	YES	-	-	-	-	70	YES	1.2	YES
	1019	Downstream of Eastbound S.R. 202 (JTB) on-ramp merge	3,424	3,493	69	2.0%	YES	-	-	-	-	69	YES	1.2	YES
	1020	Between WB S.R. 202 (JTB) on-ramp and Town Center Pkwy. off-ramp	4,221	4,320	99	2.4%	YES	-	-	-	-	99	YES	1.5	YES
	1021	Between Town Center Pkwy. off- and on-ramps	3,696	3,796	100	2.7%	YES	-	-	-	-	100	YES	1.6	YES
	1022	Town Center Pkwy. on-ramp merge	4,467	4,589	122	2.7%	YES	-	-	-	-	122	YES	1.8	YES
I-295 Northbound - Ramps	1501	I-95 Southbound on-ramp	398	395	-3	-0.8%	YES	-3	YES	-	-	-	-	0.2	YES
	1502	I-95 Northbound on-ramp	788	791	3	0.4%	YES	-	-	0%	YES	-	-	0.1	YES
	1503	I-95 on-ramp (combined) - 2-lane section	1,187	1,186	-1	-0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	1504	I-95 on-ramp (combined) - 1-lane section	1,187	1,181	-6	-0.5%	YES	-	-	0%	YES	-	-	0.2	YES
	1505	US-1 Off-Ramp	610	618	8	1.3%	YES	8	YES	-	-	-	-	0.3	YES
	1513	US-1 On-Ramp	737	727	-10	-1.4%	YES	-	-	-1%	YES	-	-	0.4	YES
	1514	Baymeadows Rd. Off-Ramp	297	299	2	0.8%	YES	2	YES	-	-	-	-	0.1	YES
	1520	Baymeadows Rd. On-Ramp	729	739	10	1.3%	YES	-	-	1%	YES	-	-	0.4	YES
	1521	Gate Pkwy. Off-Ramp	186	187	1	0.8%	YES	1	YES	-	-	-	-	0.1	YES
	1527	Gate Pkwy. On-Ramp	526	528	2	0.4%	YES	2	YES	-	-	-	-	0.1	YES
	1528	SR 202 (JTB) Off-Ramp (combined)	1,822	1,837	15	0.8%	YES	-	-	1%	YES	-	-	0.4	YES
	1529	SR 202 (JTB) WB Off-Ramp	229	233	4	1.9%	YES	4	YES	-	-	-	-	0.3	YES
	1530	SR 202 (JTB) EB Off-Ramp	1,593	1,604	11	0.7%	YES	-	-	1%	YES	-	-	0.3	YES
	1531	SR 202 (JTB) On-Ramp - 2-lane section	2,212	2,230	18	0.8%	YES	-	-	1%	YES	-	-	0.4	YES
	1532	SR 202 (JTB) On-Ramp - 1-lane section	2,212	2,231	19	0.9%	YES	-	-	1%	YES	-	-	0.4	YES
	1533	On-Ramp from WB SR 202 (JTB)	797	811	14	1.8%	YES	-	-	2%	YES	-	-	0.5	YES
	1534	On-Ramp from EB SR 202 (JTB)	1,110	1,127	17	1.6%	YES	-	-	2%	YES	-	-	0.5	YES
	1535	Town Center Pkwy Off-Ramp	526	527	1	0.2%	YES	1	YES	-	-	-	-	0.1	YES
	1539	Town Center Pkwy On-Ramp	771	776	5	0.6%	YES	-	-	1%	YES	-	-	0.2	YES
I-295 Southbound	3001	Upstream of Town Center Pkwy. off-ramp	3,051	3,038	-13	-0.4%	YES	-	-	-	-	-13	YES	0.2	YES
	3002	Between Town Center Pkwy. off- and on-ramps	2,636	2,620	-16	-0.6%	YES	-	-	-1%	YES	-	-	0.3	YES
	3003	Between Town Center Pkwy. On-ramp and S.R. 202 (JTB) off-ramp (combined)	3,441	3,415	-26	-0.8%	YES	-	-	-	-	-26	YES	0.4	YES
	3004	Between S.R. 202 (JTB) off-ramp (combined) and Westbound S.R. 202 (JTB) on-ramp	2,229	2,212	-17	-0.7%	YES	-	-	-1%	YES	-	-	0.4	YES
	3005	Westbound S.R. 202 (JTB) on-ramp merge	3,645	3,615	-30	-0.8%	YES	-	-	-	-	-30	YES	0.5	YES
	3006	Downstream of Westbound S.R. 202 (JTB) on-ramp merge	3,645	3,535	-110	-3.0%	YES	-	-	-	-	-110	YES	1.8	YES
	3007	Between Eastbound S.R. 202 (JTB) on-ramp and Gate Pkwy. off-ramp	4,263	4,259	-4	-0.1%	YES	-	-	-	-	-4	YES	0.1	YES
	3008	Between Gate Pkwy off- and on- ramps - 4-lane section	3,484	3,497	13	0.4%	YES	-	-	-	-	13	YES	0.2	YES
	3009	Between Gate Pkwy off- and on- ramps - 3-lane section	3,484	3,554	70	2.0%	YES	-	-	-	-	70	YES	1.2	YES
	3010	Between Gate Pkwy off- and on- ramps - 2-lane section	3,484	3,602	118	3.4%	YES	-	-	-	-	118	YES	2.0	YES
	3011	Between Gate Pkwy. on-ramp and S.R. 152 (Baymeadows Rd.) off-ramp	4,272	4,409	137	3.2%	YES	-	-	-	-	137	YES	2.1	YES
	3012	Between S.R. 152 (Baymeadows Rd.) off- and on-ramps	3,611	3,722	111	3.1%	YES	-	-	-	-	111	YES	1.8	YES
	3013	S.R. 152 (Baymeadows Rd.) on-ramp merge	3,950	3,997	47	1.2%	YES	-	-	-	-	47	YES	0.7	YES
	3014	Downstream of S.R. 152 (Baymeadows Rd.) on-ramp merge	3,950	4,046	96	2.4%	YES	-	-	-	-	96	YES	1.5	YES
	3015	Upstream of S.R. 9B off-ramp - 2-lane section	3,950	4,052	102	2.6%	YES	-	-	-	-	102	YES	1.6	YES
	3016	Upstream of S.R. 9B off-ramp - 3-lane section	3,950	4,054	104	2.6%	YES	-	-	-	-	104	YES	1.6	YES
	3017	Downstraem of S.R. 9B off-ramp - 2-lane section	3,119	3,197	78	2.5%	YES	-	-	-	-	78	YES	1.4	YES
	3018	Downstraem of S.R. 9B off-ramp - 3-lane section	3,119	3,197	78	2.5%	YES	-	-	-	-	78	YES	1.4	YES
	3019	Upstream of S.R. 5 (US 1) off-ramp - 3-lane section	3,119	3,197	78	2.5%	YES	-	-	-	-	78	YES	1.4	YES
	3020	Upstream of S.R. 5 (US 1) off-ramp - 4-lane section	3,119	3,189	70	2.2%	YES	-	-	-	-	70	YES	1.2	YES
	3021	Between S.R. 5 (US 1) off- and on-ramps	2,678	2,741	63	2.4%	YES	-	-	2%	YES	-	-	1.2	YES
	3022	Downstream of S.R. 5 (US 1) on-ramp	3,323	3,397	74	2.2%	YES	-	-	-	-	74	YES	1.3	YES
	3023	Upstream of Northbound I-95 off-ramp	3,323	3,391	68	2.0%	YES	-	-	-	-	68	YES	1.2	YES
	3024	Downstream of Northbound I-95 off-ramp	3,145	3,225	80	2.5%	YES	-	-	-	-	80	YES	1.4	YES
	3025	Upstream of Southbound I-95 off-ramp - 3-lane section	3,145	3,228	83	2.6%	YES	-	-	-	-	83	YES	1.5	YES
	3026	Upstream of Southbound I-95 off-ramp - 4-lane section	3,145	3,225	80	2.6%	YES	-	-	-	-	80	YES	1.4	YES
	3027	Downstream of Southbound I-95 off-ramp - 3-lane section	2,000	2,041	41	2.1%	YES	-	-	2%	YES	-	-	0.9	YES
	3028	Downstream of Southbound I-95 off-ramp - 2-lane section	2,000	2,080	80	4.0%	YES	-	-	4%	YES	-	-	1.8	YES
I-295 Southbound - Ramps	3501	Town Center Pkwy Off-Ramp	415	414	-1	-0.3%	YES	-1	YES	-	-	-	-	0.1	YES
	3505	Town Center Pkwy On-Ramp	805	788	-17	-2.1%	YES	-	-	-2%	YES	-	-	0.6	YES
	3506	SR 202 (JTB) Off-Ramp (combined)	1,212	1,201	-11	-0.9%	YES	-	-	-1%	YES	-	-	0.3	YES
	3507	SR 202 (JTB) WB On-Ramp	644	633	-11	-1.8%	YES	-11	YES	-	-	-	-	0.4	YES
	3508	SR 202 (JTB) EB On-Ramp	568	569	1	0.2%	YES	1	YES	-	-	-	-	0.1	YES
	3509	On-Ramp from WB SR 202 (JTB)	1,415	1,420	5	0.4%	YES	-	-	0%	YES	-	-	0.1	YES
	3510	On-Ramp from EB SR 202 (combined)	1,729	1,739	10	0.6%	YES	-	-	1%	YES	-	-	0.2	YES
	3511	On-Ramp from EB SR 202 (JTB)	619	632	13	2.0%	YES	13	YES	-	-	-	-	0.5	YES
	3512	Gate Pkwy. Off-Ramp	780	778	-2	-0.3%	YES	-	-	0%	YES	-	-	0.1	YES
	3516	Gate Pkwy. On-Ramp - 2-lane segment	788	783	-5	-0.7%	YES	-	-	-1%	YES	-	-	0.2	YES
	3517	Gate Pkwy. On-Ramp - 1-lane segment	788	789	1	0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	3518	Baymeadows Rd. Off-Ramp	661	687	26	3.9%	YES	26	YES	-	-	-	-	1.0	YES
	3523	Baymeadows Rd. On-Ramp	339	338	-1	-0.2%	YES	-1	YES	-	-	-	-	0.0	YES
	3524	US-1 Off-Ramp	441	457	16	3.7%	YES	16	YES	-	-	-	-	0.8	YES
	3530	US-1 On-Ramp - 2-lane segment	644	643	-1	-0.1%	YES	-1	YES	-	-	-	-	0.0	YES
	3531	US-1 On-Ramp - 1-lane segment	644	641	-3	-0.4%	YES	-3	YES	-	-	-	-	0.1	YES
	3532	I-95 NB Off-Ramp	178	179	1	0.6%	YES	1	YES	-	-	-	-	0.1	YES
	3533	I-95 SB Off-Ramp	1,144	1,181	37	3.2%	YES	-	-	3%	YES	-	-	1.1	YES
S.R. 9B	17001	Northbound S.R. 9B	627	624	-3	-0.5%	YES	-3	YES	-	-	-	-	0.1	YES
	19001	Southbound S.R. 9B	831	859	28	3.3%	YES	-	-	3%	YES	-	-	0.9	YES
S.R. 202 (JTB) Eastbound	16001	Upstream of Gate Pkwy. off-ramp - Section 1	5,298	5,247	-51	-1.0%	YES	-	-	-	-	-51	YES	0.7	YES
	16002	Upstream of Gate Pkwy. off-ramp - Section 2	5,298	5,242	-56	-1.1%	YES	-	-	-	-	-56	YES	0.8	YES
	16003	Between Gate Pkwy. off- and on-ramps	4,577	4,545	-32	-0.7%	YES	-	-	-	-	-32	YES	0.5	YES
	16004	Between Gate Pkwy. on-ramp and I-295 off-ramp	6,035	6,022	-13	-0.2%	YES	-	-	-	-	-13	YES	0.2	YES
	16005	Between I-295 off-ramp and Southbound I-295 on-ramp	4,306	4,298	-8	-0.2%	YES	-	-	-	-	-8	YES	0.1	YES
	16006	Southbound I-295 on-ramp merge	4,874	4,842	-32	-0.7%	YES	-	-	-	-	-32	YES	0.5	YES
	16007	Downstream of Southbound I-295 on-ramp merge	4,874	4,738	-136	-2.8%	YES	-	-	-	-	-136	YES	2.0	YES
	16008	Between Northbound I-295 on-ramp and Kernan Blvd. off-ramp	6,467	6,479	12	0.2%	YES	-	-	-	-	12	YES	0.1	YES
	16009	Between Kernan Blvd. off- and on- ramp - 4-lane section	4,925	4,932	7	0.1%	YES	-	-	-	-	7	YES	0.1	YES
	16010	Between Kernan Blvd. off- and on- ramp - 3-lane section	4,925	4,925	0	0.0%	YES	-	-	-	-	0	YES	0.0	YES
	16011	Kernan Blvd. on-ramp merge - 4-lane section	5,077	5,066	-11	-0.2%	YES	-	-	-	-	-11	YES	0.1	YES
	16012	Kernan Blvd. on-ramp merge - 3-lane section	5,077	4,994	-83	-1.6%	YES	-	-	-	-	-83	YES	1.2	YES
	16013	Downstream of Kernan Blvd. on-ramp	5,077	5,114	37	0.7%	YES	-	-	-	-	37	YES	0.5	YES
S.R. 202 (JTB) Eastbound - Ramps	16501	Gate Pkwy. Off-Ramp	720	708	-12	-1.7%	YES	-	-	-2%	YES	-	-	0.5	YES
	16505	Gate Pkwy. On-Ramp	1,458	1,482	24	1.6%	YES	-	-	2%	YES	-	-	0.6	YES
	16506	Kernan Blvd. Off-Ramp	1,543	1,552	9	0.6%	YES	-	-	1%	YES	-	-	0.2	YES
	16510	Kernan Blvd. On-Ramp	153	155	2	1.1%	YES	2	YES	-	-	-	-	0.1	YES
S.R. 202 (JTB) Westbound	18001	Upstream of Kernan Blvd. off-ramp - section 1	4,009	4,016	7	0.2%	YES	-	-	-	-	7	YES	0.1	YES
	18002	Upstream of Kernan Blvd. off-ramp - section 2	4,009	4,021	12	0.3%	YES	-	-	-	-	12	YES	0.2	YES
	18003	Between Kernan Blvd. off- and on-ramps	3,712	3,721	9	0.2%	YES	-	-	-	-	9	YES	0.2	YES
	18004	Between Kernan Blvd. on-ramp and I-295 off-ramp (combined)	4,289	4,308	19	0.4%	YES	-	-	-	-	19	YES	0.3	YES
	18005	Between I-295 off-ramp (combined) and Northbound I-295 on-ramp	2,077	2,083	6	0.3%	YES	-	-	0%	YES	-	-	0.1	YES
	18006	Northbound I-295 on-ramp merge - 5-lane section	2,305	2,308	3	0.1%	YES	-	-	0%	YES	-	-	0.1	YES
	18007	Northbound I-295 on-ramp merge - 4-lane section	2,305	2,307	2	0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	18008	Between Southbound I-295 on-ramp and Gate Pkwy. Off-ramp	2,950	2,956	6	0.2%	YES	-	-	-	-	6	YES	0.1	YES
	18009	Between Gate Pkwy. off- and on-ramps	2,390	2,392	2	0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	18010	Gate Pkwy. on-ramp merge - 5-lane section	3,246	3,228	-18	-0.5%	YES	-	-	-	-	-18	YES	0.3	YES
	18011	Gate Pkwy. on-ramp merge - 4-lane section	3,246	3,269	23	0.7%	YES	-	-	-	-	23	YES	0.4	YES
S.R. 202 (JTB) Westbound - Ramps	18501	Kernan Blvd. Off-Ramp	297	292	-5	-1.8%	YES	-5	YES	-	-	-	-	0.3	YES
	18505	Kernan Blvd. On-Ramp	576	578	2	0.3%	YES	2	YES	-	-	-	-	0.1	YES
	18506	Gate Pkwy. Off-Ramp	559	562	3	0.6%	YES	3	YES	-	-	-	-	0.1	YES
	18511	Gate Pkwy. On-Ramp	856	839	-17	-2.0%	YES	-	-	-2%	YES	-	-	0.6	YES

TABLE A4 CONTD.: Hourly flow comparisons – PM peak period - hour 1

Roadway	VISSIM Link Number	Location	Peak Hour Count Volume (vph)	VISSIM Model Volume (vph)	Difference	Within 15%	Criteria Met	Individual Link Flow: < 700 vph Within 100vph	< 700 vph Criteria Met	700 vph to 2,700 vph Within 15%	700 vph to 2,700 vph Criteria Met	> 2,700 vph Within 400vph	> 2,700 vph Criteria Met	GEH Statistic	GEH Criteria Met (<= 5)
US 1 (Philips Hwy.) Eastbound	4001	Begin Project to SR 115 (Southside Blvd.)	1,246	1,220	-26	-2.1%	YES	-	-	-2%	YES	-	-	0.7	YES
	4002		898	892	-6	-0.7%	YES	-	-	-1%	YES	-	-	0.2	YES
	4006	SR 115 (Southside Blvd.) to I-295 SB off-ramp	1,619	1,614	-5	-0.3%	YES	-	-	0%	YES	-	-	0.1	YES
	4007		1,619	1,612	-7	-0.4%	YES	-	-	0%	YES	-	-	0.2	YES
	4008		1,619	1,604	-15	-0.9%	YES	-	-	-1%	YES	-	-	0.4	YES
	4009		1,458	1,422	-36	-2.5%	YES	-	-	-2%	YES	-	-	0.9	YES
	4011	I-295 SB off-ramp to Greenland Rd./I-295 NB off-ramp	1,805	1,781	-24	-1.3%	YES	-	-	-1%	YES	-	-	0.6	YES
	4012		890	863	-27	-3.1%	YES	-	-	-3%	YES	-	-	0.9	YES
	4015	Greenland Rd./I-295 NB off-ramp to Business Park Blvd.	1,297	1,246	-51	-3.9%	YES	-	-	-4%	YES	-	-	1.4	YES
	4018	Business Park Blvd. to End Project	1,263	1,256	-7	-0.6%	YES	-	-	-1%	YES	-	-	0.2	YES
US 1 (Philips Hwy.) Westbound	2001	Begin Project to Business Park Blvd.	1,314	1,322	8	0.6%	YES	-	-	1%	YES	-	-	0.2	YES
	2002		1,271	1,259	-12	-0.9%	YES	-	-	-1%	YES	-	-	0.3	YES
	2005	Business Park Blvd.to Greenland Rd./I-295 NB off-ramp	1,526	1,524	-2	-0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	2006		1,526	1,537	11	0.7%	YES	-	-	1%	YES	-	-	0.3	YES
	2010	Greenland Rd./I-295 NB off-ramp to I-295 SB off-ramp	1,382	1,320	-62	-4.5%	YES	-	-	-4%	YES	-	-	1.7	YES
	2011		898	860	-38	-4.2%	YES	-	-	-4%	YES	-	-	1.3	YES
	2013	I-295 SB off-ramp to SR 115 (Southside Blvd.)	992	949	-43	-4.3%	YES	-	-	-4%	YES	-	-	1.4	YES
	2014		992	981	-11	-1.1%	YES	-	-	-1%	YES	-	-	0.4	YES
	2018	SR 115 (Southside Blvd.) to End Project	1,068	1,054	-14	-1.3%	YES	-	-	-1%	YES	-	-	0.4	YES
SR 152 (Baymeadows Rd.) Eastbound	6001	Begin Project to Pointmeadows Rd.	1,161	1,110	-51	-4.4%	YES	-	-	-4%	YES	-	-	1.5	YES
	6003	Pointmeadows Rd. to I-295 SB off-ramp	1,068	1,049	-19	-1.8%	YES	-	-	-2%	YES	-	-	0.6	YES
	6005	I-295 SB off-ramp to I-295 NB off-ramp	975	937	-38	-3.9%	YES	-	-	-4%	YES	-	-	1.2	YES
	6007		322	322	0	-0.1%	YES	0	YES	-	-	-	-	0.0	YES
	6008	I-295 NB off-ramp to R.G. E. Skinner Blvd.	398	386	-12	-2.9%	YES	-12	YES	-	-	-	-	0.6	YES
	6011	R.G.E. Skinner Blvd. to End Project	288	299	11	3.8%	YES	11	YES	-	-	-	-	0.6	YES
SR 152 (Baymeadows Rd.) Westbound	8001	Begin Project to R.G.E. Skinner Blvd.	466	463	-3	-0.6%	YES	-3	YES	-	-	-	-	0.1	YES
	8002		424	419	-5	-1.2%	YES	-5	YES	-	-	-	-	0.2	YES
	8005	R.G.E. Skinner Blvd. to I-295 NB off-ramp	551	542	-9	-1.6%	YES	-9	YES	-	-	-	-	0.4	YES
	8006	I-295 NB off-ramp to I-295 SB off-ramp	644	619	-25	-3.8%	YES	-25	YES	-	-	-	-	1.0	YES
	8007		509	501	-8	-1.5%	YES	-8	YES	-	-	-	-	0.3	YES
	8009	I-295 SB off-ramp to Pointmeadows Rd.	1,060	999	-61	-5.8%	YES	-	-	-6%	YES	-	-	1.9	YES
	8011	Pointmeadows Rd. to End Project	1,178	1,204	26	2.2%	YES	-	-	2%	YES	-	-	0.8	YES
Gate Pkwy. Eastbound	12001	Begin Project to Pointmeadows Rd.	1,365	1,365	0	0.0%	YES	-	-	0%	YES	-	-	0.0	YES
	12002		1,110	1,096	-14	-1.3%	YES	-	-	-1%	YES	-	-	0.4	YES
	12004	Pointmeadows Rd. to I-295 SB off-ramp	1,212	1,141	-71	-5.9%	YES	-	-	-6%	YES	-	-	2.1	YES
	12006	I-295 SB off-ramp to I-295 NB off-ramp	780	767	-13	-1.7%	YES	-	-	-2%	YES	-	-	0.5	YES
	12007		483	488	5	1.0%	YES	5	YES	-	-	-	-	0.2	YES
	12009	I-295 NB off-ramp to End Project	500	499	-1	-0.2%	YES	-1	YES	-	-	-	-	0.0	YES
	12010		500	514	14	2.8%	YES	14	YES	-	-	-	-	0.6	YES
Gate Pkwy. Westbound	14001	Begin Project to I-295 NB off-ramp	314	298	-16	-5.1%	YES	-16	YES	-	-	-	-	0.9	YES
	14002	I-295 NB off-ramp to I-295 SB off-ramp	254	254	0	-0.1%	YES	0	YES	-	-	-	-	0.0	YES
	14003		220	224	4	1.9%	YES	4	YES	-	-	-	-	0.3	YES
	14005	I-295 SB off-ramp to Pointmeadows Rd.	678	637	-41	-6.0%	YES	-41	YES	-	-	-	-	1.6	YES
	14006		449	447	-2	-0.4%	YES	-2	YES	-	-	-	-	0.1	YES
	14009	Pointmeadows Rd. to End Project	559	561	2	0.4%	YES	2	YES	-	-	-	-	0.1	YES
Town Center Pkwy. Eastbound	20001	Begin Project to St. Johns Bluff Rd.	1,466	1,471	5	0.3%	YES	-	-	0%	YES	-	-	0.1	YES
	20002		1,017	1,007	-10	-1.0%	YES	-	-	-1%	YES	-	-	0.3	YES
	20005	St. Johns Bluff Rd. to I-295	1,297	1,290	-7	-0.5%	YES	-	-	-1%	YES	-	-	0.2	YES
	20006		144	139	-5	-3.2%	YES	-5	YES	-	-	-	-	0.4	YES
	20010	I-295 to Eco Dr.	348	343	-5	-1.4%	YES	-5	YES	-	-	-	-	0.3	YES
	20012		263	200	-63	-24.1%	NO	-63	YES	-	-	-	-	4.2	YES
	20014	Eco Dr. to End Project	144	142	-3	-1.7%	YES	-3	YES	-	-	-	-	0.2	YES
Town Center Pkwy. Westbound	22001	Begin Project to Eco Dr.	271	261	-10	-3.7%	YES	-10	YES	-	-	-	-	0.6	YES
	22003	Eco Dr. to I-295	568	546	-22	-3.9%	YES	-22	YES	-	-	-	-	0.9	YES
	22004		568	533	-35	-6.1%	YES	-35	YES	-	-	-	-	1.5	YES
	22006		144	136	-8	-5.7%	YES	-8	YES	-	-	-	-	0.7	YES
	22008	I-295 to St. Johns Bluff Rd.	602	641	39	6.5%	YES	39	YES	-	-	-	-	1.6	YES
	22011	St. Johns Bluff Rd. to End Project	848	849	1	0.1%	YES	-	-	0%	YES	-	-	0.0	YES
Gate Pkwy. Southbound	35001	Begin Project to Town Center Pkwy.	1,890	1,885	-5	-0.2%	YES	-	-	0%	YES	-	-	0.1	YES
	35002		1,187	1,163	-24	-2.0%	YES	-	-	-2%	YES	-	-	0.7	YES
	35004	Town Center Pkwy. to Costco/Skinner Lake Dr.	1,636	1,597	-39	-2.4%	YES	-	-	-2%	YES	-	-	1.0	YES
	35005		1,568	1,557	-11	-0.7%	YES	-	-	-1%	YES	-	-	0.3	YES
	35007	Costco/Skinner Lake Dr. to I-295	1,738	1,724	-14	-0.8%	YES	-	-	-1%	YES	-	-	0.3	YES
	35008	I-295 to Deerwood Park Blvd.	1,246	1,253	7	0.6%	YES	-	-	1%	YES	-	-	0.2	YES
	35011		1,009	1,009	0	0.0%	YES	-	-	0%	YES	-	-	0.0	YES
	35012	Deerwood Park Blvd. to End Project	1,102	1,115	13	1.1%	YES	-	-	1%	YES	-	-	0.4	YES
Gate Pkwy. Northbound	33001	Begin Project to Deerwood Park Blvd.	704	662	-42	-6.0%	YES	-	-	-6%	YES	-	-	1.6	YES
	33003	Deerwood Park Blvd. to I-295	1,771	1,766	-5	-0.3%	YES	-	-	0%	YES	-	-	0.1	YES
	33004	I-295 to Costco/Skinner Lake Dr.	653	631	-22	-3.4%	YES	-22	YES	-	-	-	-	0.9	YES
	33007		1,119	1,104	-15	-1.4%	YES	-	-	-1%	YES	-	-	0.5	YES
	33011	Costco/Skinner Lake Dr. to Town Center Pkwy.	1,339	1,318	-21	-1.6%	YES	-	-	-2%	YES	-	-	0.6	YES
	33014	Town Center Pkwy. to End Project	831	836	5	0.7%	YES	-	-	1%	YES	-	-	0.2	YES
Kernan Blvd. Southbound	39001	Begin Project to 1st Tech Pkwy.	653	653	0	0.0%	YES	0	YES	-	-	-	-	0.0	YES
	39003	1st Tech Pkwy. to SR 202 (JTB)	144	186	42	29.4%	NO	42	YES	-	-	-	-	3.3	YES
	39008	SR 202 (JTB) to End. Project	17	17	0	-1.3%	YES	0	YES	-	-	-	-	0.1	YES
Kernan Blvd. Northbound	37001	Begin Project to SR 202 (JTB)	17	14	-3	-20.1%	NO	-3	YES	-	-	-	-	0.9	YES
	37005	SR 202 (JTB) to 1st Tech Pkwy.	1,695	1,787	92	5.4%	YES	-	-	5%	YES	-	-	2.2	YES
	37007	1st Tech Pkwy. To End Project	1,704	1,726	22	1.3%	YES	-	-	1%	YES	-	-	0.5	YES
Total			373,752	374,848	1,096										
Sum of all Link Flows			0.29%												
Sum of all Link Flows (Flows within 5%)			YES												
Total Count						199	196	53	53	86	86	60	60	199	199
Individual Links						98%		100%		100%		100%		100%	
Individual Links (Flows met for >85% Cases and GEH Statistic < 5 for 85% of Cases)						YES		YES		YES		YES		YES	

TABLE A5: Hourly flow comparisons – PM peak period - hour 2

Roadway	VISSIM Link Number	Location	Peak Hour Count Volume (vph)	VISSIM Model Volume (vph)	Difference	Within 15%	Criteria Met	Individual Link Flow: < 700 vph Within 100vph	< 700 vph Criteria Met	700 vph to 2,700 vph Within 15%	700 vph to 2,700 vph Criteria Met	> 2,700 vph Within 400vph	> 2,700 vph Criteria Met	GEH Statistic	GEH Criteria Met (<= 5)
I-295 Northbound	1001	Upstream of I-95 on-ramps	1,680	1,701	21	1.3%	YES	-	-	1%	YES	-	-	0.5	YES
	1002	Upstream of S.R. 5 (US 1) off-ramp	3,080	3,098	18	0.6%	YES	-	-	-	-	18	YES	0.3	YES
	1003	Between S.R. 5 (US 1) off- and on-ramps	2,360	2,373	13	0.5%	YES	-	-	1%	YES	-	-	0.3	YES
	1004	S.R. 5 (US 1) on-ramp merge	3,230	3,227	-3	-0.1%	YES	-	-	-	-	-3	YES	0.1	YES
	1005	Downstream of S.R. 5 (US 1) on-ramp merge	3,230	3,225	-5	-0.2%	YES	-	-	-	-	-5	YES	0.1	YES
	1006	3-lane to 2-lane merge - North of S.R. 5 (US 1)	3,230	3,228	-2	-0.1%	YES	-	-	-	-	-2	YES	0.0	YES
	1007	Upstream of S.R. 9B on-ramp	3,230	3,217	-13	-0.4%	YES	-	-	-	-	-13	YES	0.2	YES
	1008	S.R. 9B on-ramp merge	3,970	3,944	-26	-0.7%	YES	-	-	-	-	-26	YES	0.4	YES
	1009	Downstream of S.R. 9B on-ramp merge - 3-lane section	3,970	3,936	-34	-0.8%	YES	-	-	-	-	-34	YES	0.5	YES
	1010	Downstream of S.R. 9B on-ramp merge - 2-lane section	3,970	3,948	-22	-0.6%	YES	-	-	-	-	-22	YES	0.4	YES
	1011	Upstream of S.R. 152 (Baymeadows Rd.) off-ramp	3,970	3,947	-23	-0.6%	YES	-	-	-	-	-23	YES	0.4	YES
	1012	Between S.R. 152 (Baymeadows Rd.) off- and on-ramps	3,620	3,587	-33	-0.9%	YES	-	-	-	-	-33	YES	0.5	YES
	1013	Between S.R. 152 on-ramp and Gate Pkwy. off-ramp	4,480	4,434	-46	-1.0%	YES	-	-	-	-	-46	YES	0.7	YES
	1014	Between Gate Pkwy. off- and on-ramps	4,260	4,202	-58	-1.4%	YES	-	-	-	-	-58	YES	0.9	YES
	1015	Between Gate Pkwy. on-ramp and S.R. 202 (JTB) off-ramp	4,880	4,814	-66	-1.4%	YES	-	-	-	-	-66	YES	0.9	YES
	1016	Downstream of of S.R. 202 (JTB) off-ramp - 3-lane section	2,730	2,664	-66	-2.4%	YES	-	-	-	-	-66	YES	1.3	YES
	1017	Downstream of of S.R. 202 (JTB) off-ramp - 2-lane section	2,730	2,617	-113	-4.1%	YES	-	-	-	-	-113	YES	2.2	YES
	1018	Eastbound S.R. 202 (JTB) on-ramp merge	4,040	3,714	-326	-8.1%	YES	-	-	-	-	-326	YES	5.2	NO
	1019	Downstream of Eastbound S.R. 202 (JTB) on-ramp merge	4,040	3,709	-331	-8.2%	YES	-	-	-	-	-331	YES	5.3	NO
	1020	Between WB S.R. 202 (JTB) on-ramp and Town Center Pkwy. off-ramp	4,980	4,662	-318	-6.4%	YES	-	-	-	-	-318	YES	4.6	YES
	1021	Between Town Center Pkwy. off- and on-ramps	4,360	4,078	-282	-6.5%	YES	-	-	-	-	-282	YES	4.3	YES
	1022	Town Center Pkwy. on-ramp merge	5,270	4,989	-281	-5.3%	YES	-	-	-	-	-281	YES	3.9	YES
I-295 Northbound - Ramps	1501	I-95 Southbound on-ramp	470	470	0	0.1%	YES	0	YES	-	-	-	-	0.0	YES
	1502	I-95 Northbound on-ramp	930	931	1	0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	1503	I-95 on-ramp (combined) - 2-lane section	1,400	1,401	1	0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	1504	I-95 on-ramp (combined) - 1-lane section	1,400	1,396	-5	-0.3%	YES	-	-	0%	YES	-	-	0.1	YES
	1505	US-1 Off-Ramp	720	726	6	0.0%	YES	-	-	1%	YES	-	-	0.2	YES
	1513	US-1 On-Ramp	870	846	-24	-2.7%	YES	-	-	-3%	YES	-	-	0.8	YES
	1514	Baymeadows Rd. Off-Ramp	350	350	0	0.0%	YES	0	YES	-	-	-	-	0.0	YES
	1520	Baymeadows Rd. On-Ramp	860	845	-15	-1.7%	YES	-	-	-2%	YES	-	-	0.5	YES
	1521	Gate Pkwy. Off-Ramp	220	223	3	1.6%	YES	3	YES	-	-	-	-	0.2	YES
	1527	Gate Pkwy. On-Ramp	620	623	3	0.5%	YES	3	YES	-	-	-	-	0.1	YES
	1528	SR 202 (JTB) Off-Ramp (combined)	2,150	2,103	-47	-2.2%	YES	-	-	-2%	YES	-	-	1.0	YES
	1529	SR 202 (JTB) WB Off-Ramp	270	263	-7	-2.6%	YES	-7	YES	-	-	-	-	0.4	YES
	1530	SR 202 (JTB) EB Off-Ramp	1,880	1,836	-44	-2.3%	YES	-	-	-2%	YES	-	-	1.0	YES
	1531	SR 202 (JTB) On-Ramp - 2-lane section	2,610	2,579	-31	-1.2%	YES	-	-	-1%	YES	-	-	0.6	YES
	1532	SR 202 (JTB) On-Ramp - 1-lane section	2,610	2,582	-28	-1.1%	YES	-	-	-1%	YES	-	-	0.6	YES
	1533	On-Ramp from WB SR 202 (JTB)	940	936	-4	-0.5%	YES	-	-	0%	YES	-	-	0.1	YES
	1534	On-Ramp from EB SR 202 (JTB)	1,310	1,176	-134	-10.2%	YES	-	-	-10%	YES	-	-	3.8	YES
	1535	Town Center Pkwy Off-Ramp	620	582	-38	-6.1%	YES	-38	YES	-	-	-	-	1.5	YES
	1539	Town Center Pkwy On-Ramp	910	898	-12	-1.3%	YES	-	-	-1%	YES	-	-	0.4	YES
I-295 Southbound	3001	Upstream of Town Center Pkwy. off-ramp	3,600	3,574	-26	-0.7%	YES	-	-	-	-	-26	YES	0.4	YES
	3002	Between Town Center Pkwy. off- and on-ramps	3,110	3,088	-22	-0.7%	YES	-	-	-	-	-22	YES	0.4	YES
	3003	Between Town Center Pkwy. On-ramp and S.R. 202 (JTB) off-ramp (combined)	4,060	4,026	-34	-0.8%	YES	-	-	-	-	-34	YES	0.5	YES
	3004	Between S.R. 202 (JTB) off-ramp (combined) and Westbound S.R. 202 (JTB) on-ramp	2,630	2,595	-35	-1.3%	YES	-	-	-1%	YES	-	-	0.7	YES
	3005	Westbound S.R. 202 (JTB) on-ramp merge	4,300	4,097	-203	-4.7%	YES	-	-	-	-	-203	YES	3.1	YES
	3006	Downstream of Westbound S.R. 202 (JTB) on-ramp merge	4,300	3,976	-324	-7.5%	YES	-	-	-	-	-324	YES	5.0	YES
	3007	Between Eastbound S.R. 202 (JTB) on-ramp and Gate Pkwy. off-ramp	5,030	4,666	-364	-7.2%	YES	-	-	-	-	-364	YES	5.2	NO
	3008	Between Gate Pkwy off- and on- ramps - 4-lane section	4,110	3,692	-418	-10.2%	YES	-	-	-	-	-418	NO	6.7	NO
	3009	Between Gate Pkwy off- and on- ramps - 3-lane section	4,110	3,562	-548	-13.3%	YES	-	-	-	-	-548	NO	8.8	NO
	3010	Between Gate Pkwy off- and on- ramps - 2-lane section	4,110	3,539	-571	-13.9%	YES	-	-	-	-	-571	NO	9.2	NO
	3011	Between Gate Pkwy. on-ramp and S.R. 152 (Baymeadows Rd.) off-ramp	5,040	4,463	-577	-11.5%	YES	-	-	-	-	-577	NO	8.4	NO
	3012	Between S.R. 152 (Baymeadows Rd.) off- and on-ramps	4,260	3,781	-479	-11.2%	YES	-	-	-	-	-479	NO	7.6	NO
	3013	S.R. 152 (Baymeadows Rd.) on-ramp merge	4,660	4,106	-554	-11.9%	YES	-	-	-	-	-554	NO	8.4	NO
	3014	Downstream of S.R. 152 (Baymeadows Rd.) on-ramp merge	4,660	4,159	-501	-10.8%	YES	-	-	-	-	-501	NO	7.6	NO
	3015	Upstream of S.R. 9B off-ramp - 2-lane section	4,660	4,163	-497	-10.7%	YES	-	-	-	-	-497	NO	7.5	NO
	3016	Upstream of S.R. 9B off-ramp - 3-lane section	4,660	4,173	-487	-10.5%	YES	-	-	-	-	-487	NO	7.3	NO
	3017	Downstraem of S.R. 9B off-ramp - 2-lane section	3,680	3,298	-382	-10.4%	YES	-	-	-	-	-382	YES	6.5	NO
	3018	Downstraem of S.R. 9B off-ramp - 3-lane section	3,680	3,298	-382	-10.4%	YES	-	-	-	-	-382	YES	6.5	NO
	3019	Upstream of S.R. 5 (US 1) off-ramp - 3-lane section	3,680	3,296	-384	-10.4%	YES	-	-	-	-	-384	YES	6.5	NO
	3020	Upstream of S.R. 5 (US 1) off-ramp - 4-lane section	3,680	3,288	-392	-10.7%	YES	-	-	-	-	-392	YES	6.6	NO
	3021	Between S.R. 5 (US 1) off- and on-ramps	3,160	2,832	-328	-10.4%	YES	-	-	-	-	-328	YES	6.0	NO
	3022	Downstream of S.R. 5 (US 1) on-ramp	3,920	3,543	-377	-9.6%	YES	-	-	-	-	-377	YES	6.2	NO
	3023	Upstream of Northbound I-95 off-ramp	3,920	3,517	-403	-10.3%	YES	-	-	-	-	-403	NO	6.6	NO
	3024	Downstream of Northbound I-95 off-ramp	3,710	3,332	-378	-10.2%	YES	-	-	-	-	-378	YES	6.4	NO
	3025	Upstream of Southbound I-95 off-ramp - 3-lane section	3,710	3,340	-370	-10.0%	YES	-	-	-	-	-370	YES	6.2	NO
	3026	Upstream of Southbound I-95 off-ramp - 4-lane section	3,710	3,338	-372	-10.0%	YES	-	-	-	-	-372	YES	6.3	NO
	3027	Downstream of Southbound I-95 off-ramp - 3-lane section	2,360	2,117	-243	-10.3%	YES	-	-	-10%	YES	-	-	5.1	NO
	3028	Downstream of Southbound I-95 off-ramp - 2-lane section	2,360	2,164	-196	-8.3%	YES	-	-	-8%	YES	-	-	4.1	YES
I-295 Southbound - Ramps	3501	Town Center Pkwy Off-Ramp	490	483	-7	-1.5%	YES	-7	YES	-	-	-	-	0.3	YES
	3505	Town Center Pkwy On-Ramp	950	934	-16	-1.7%	YES	-	-	-2%	YES	-	-	0.5	YES
	3506	SR 202 (JTB) Off-Ramp (combined)	1,430	1,427	-3	-0.2%	YES	-	-	0%	YES	-	-	0.1	YES
	3507	SR 202 (JTB) WB On-Ramp	760	763	3	0.5%	YES	-	-	0%	YES	-	-	0.1	YES
	3508	SR 202 (JTB) EB On-Ramp	670	662	-8	-1.2%	YES	-8	YES	-	-	-	-	0.3	YES
	3509	On-Ramp from WB SR 202 (JTB)	1,670	1,620	-50	-3.0%	YES	-	-	-3%	YES	-	-	1.2	YES
	3510	On-Ramp from EB SR 202 (combined)	2,040	1,942	-98	-4.8%	YES	-	-	-5%	YES	-	-	2.2	YES
	3511	On-Ramp from EB SR 202 (JTB)	730	678	-52	-7.1%	YES	-	-	-7%	YES	-	-	1.9	YES
	3512	Gate Pkwy. Off-Ramp	920	832	-88	-9.6%	YES	-	-	-10%	YES	-	-	3.0	YES
	3516	Gate Pkwy. On-Ramp - 2-lane segment	930	914	-16	-1.7%	YES	-	-	-2%	YES	-	-	0.5	YES
	3517	Gate Pkwy. On-Ramp - 1-lane segment	930	921	-9	-1.0%	YES	-	-	-1%	YES	-	-	0.3	YES
	3518	Baymeadows Rd. Off-Ramp	780	688	-92	-11.8%	YES	-	-	-12%	YES	-	-	3.4	YES
	3523	Baymeadows Rd. On-Ramp	400	388	-12	-2.9%	YES	-12	YES	-	-	-	-	0.6	YES
	3524	US-1 Off-Ramp	520	461	-59	-11.4%	YES	-59	YES	-	-	-	-	2.7	YES
	3530	US-1 On-Ramp - 2-lane segment	760	719	-41	-5.4%	YES	-	-	-5%	YES	-	-	1.5	YES
	3531	US-1 On-Ramp - 1-lane segment	760	712	-48	-6.3%	YES	-	-	-6%	YES	-	-	1.8	YES
	3532	I-95 NB Off-Ramp	210	195	-15	-7.0%	YES	-15	YES	-	-	-	-	1.0	YES
	3533	I-95 SB Off-Ramp	1,350	1,222	-128	-9.5%	YES	-	-	-10%	YES	-	-	3.6	YES
S.R. 9B	17001	Northbound S.R. 9B	740	739	-1	-0.2%	YES	-	-	0%	YES	-	-	0.0	YES
	19001	Southbound S.R. 9B	980	883	-97	-9.8%	YES	-	-	-10%	YES	-	-	3.2	YES
S.R. 202 (JTB) Eastbound	16001	Upstream of Gate Pkwy. off-ramp - Section 1	6,250	6,217	-33	-0.5%	YES	-	-	-	-	-33	YES	0.4	YES
	16002	Upstream of Gate Pkwy. off-ramp - Section 2	6,250	6,193	-57	-0.9%	YES	-	-	-	-	-57	YES	0.7	YES
	16003	Between Gate Pkwy. off- and on-ramps	5,400	5,348	-52	-1.0%	YES	-	-	-	-	-52	YES	0.7	YES
	16004	Between Gate Pkwy. on-ramp and I-295 off-ramp	7,120	6,945	-175	-2.5%	YES	-	-	-	-	-175	YES	2.1	YES
	16005	Between I-295 off-ramp and Southbound I-295 on-ramp	5,080	4,967	-113	-2.2%	YES	-	-	-	-	-113	YES	1.6	YES
	16006	Southbound I-295 on-ramp merge	5,750	5,591	-159	-2.8%	YES	-	-	-	-	-159	YES	2.1	YES
	16007	Downstream of Southbound I-295 on-ramp merge	5,750	5,465	-285	-5.0%	YES	-	-	-	-	-285	YES	3.8	YES
	16008	Between Northbound I-295 on-ramp and Kernan Blvd. off-ramp	7,630	7,441	-189	-2.5%	YES	-	-	-	-	-189	YES	2.2	YES
	16009	Between Kernan Blvd. off- and on- ramp - 4-lane section	5,810	5,658	-152	-2.6%	YES	-	-	-	-	-152	YES	2.0	YES
	16010	Between Kernan Blvd. off- and on- ramp - 3-lane section	5,810	5,655	-155	-2.7%	YES	-	-	-	-	-155	YES	2.0	YES
	16011	Kernan Blvd. on-ramp merge - 4-lane section	5,990	5,825	-165	-2.8%	YES	-	-	-	-	-165	YES	2.1	YES
	16012	Kernan Blvd. on-ramp merge - 3-lane section	5,990	5,730	-260	-4.3%	YES	-	-	-	-	-260	YES	3.4	YES
	16013	Downstream of Kernan Blvd. on-ramp	5,990	5,854	-136	-2.3%	YES	-	-	-	-	-136	YES	1.8	YES
S.R. 202 (JTB) Eastbound - Ramps	16501	Gate Pkwy. Off-Ramp	850	829	-21	-2.5%	YES	-	-	-3%	YES	-	-	0.7	YES
	16505	Gate Pkwy. On-Ramp	1,720	1,672	-48	-2.8%	YES	-	-	-3%	YES	-	-	1.2	YES
	16506	Kernan Blvd. Off-Ramp	1,820	1,740	-80	-4.4%	YES	-	-	-4%	YES	-	-	1.9	YES
	16510	Kernan Blvd. On-Ramp	180	188	8	4.3%	YES	8	YES	-	-	-	-	0.6	YES
S.R. 202 (JTB) Westbound	18001	Upstream of Kernan Blvd. off-ramp - section 1	4,730	4,752	22	0.5%	YES	-	-	-	-	22	YES	0.3	YES
	18002	Upstream of Kernan Blvd. off-ramp - section 2	4,730	4,736	6	0.1%	YES	-	-	-	-	6	YES	0.1	YES
	18003	Between Kernan Blvd. off- and on-ramps	4,380	4,368	-12	-0.3%	YES	-	-	-	-	-12	YES	0.2	YES
	18004	Between Kernan Blvd. on-ramp and I-295 off-ramp (combined)	5,060	5,039	-21	-0.4%	YES	-	-	-	-	-21	YES	0.3	YES
	18005	Between I-295 off-ramp (combined) and Northbound I-295 on-ramp	2,450	2,459	9	0.4%	YES	-	-	0%	YES	-	-	0.2	YES
	18006	Northbound I-295 on-ramp merge - 5-lane section	2,720	2,711	-9	-0.3%	YES	-	-	-	-	-9	YES	0.2	YES
	18007	Northbound I-295 on-ramp merge - 4-lane section	2,720	2,710	-10	-0.4%	YES	-	-	-	-	-10	YES	0.2	YES
	18008	Between Southbound I-295 on-ramp and Gate Pkwy. Off-ramp	3,480	3,490	10	0.3%	YES	-	-	-	-	10	YES	0.2	YES
	18009	Between Gate Pkwy. off- and on-ramps	2,820	2,830	10	0.4%	YES	-	-	-	-	10	YES	0.2	YES
	18010	Gate Pkwy. on-ramp merge - 5-lane section	3,830	3,819	-11	-0.3%	YES	-	-	-	-	-11	YES	0.2	YES
	18011	Gate Pkwy. on-ramp merge - 4-lane section	3,830	3,862	32	0.8%	YES	-	-	-	-	32	YES	0.5	YES
S.R. 202 (JTB) Westbound - Ramps	18501	Kernan Blvd. Off-Ramp	350	340	-10	-2.9%	YES	-10	YES	-	-	-	-	0.5	YES
	18505	Kernan Blvd. On-Ramp	680	666	-14	-2.0%	YES	-14	YES	-	-	-	-	0.5	YES
	18506	Gate Pkwy. Off-Ramp	660	653	-7	-1.0%	YES	-7	YES	-	-	-	-	0.3	YES
	18511	Gate Pkwy. On-Ramp	1,010	993	-17	-1.6%	YES	-	-	-2%	YES	-	-	0.5	YES

TABLE A5 CONTD.: Hourly flow comparisons – PM peak period - hour 2

Roadway	VISSIM Link Number	Location	Peak Hour Count Volume (vph)	VISSIM Model Volume (vph)	Difference	Within 15%	Criteria Met	Individual Link Flow: < 700 vph: Within 100vph	Individual Link Flow: < 700 vph: Criteria Met	Individual Link Flow: 700 vph to 2,700 vph: Within 15%	Individual Link Flow: 700 vph to 2,700 vph: Criteria Met	Individual Link Flow: > 2,700 vph: Within 400vph	Individual Link Flow: > 2,700 vph: Criteria Met	GEH Statistic	GEH Criteria Met (<= 5)
US 1 (Philips Hwy.) Eastbound	4001	Begin Project to SR 115 (Southside Blvd.)	1,470	1,448	-22	-1.5%	YES	-	-	-1%	YES	-	-	0.6	YES
	4002		1,060	1,058	-2	-0.2%	YES	-	-	0%	YES	-	-	0.1	YES
	4006	SR 115 (Southside Blvd.) to I-295 SB off-ramp	1,910	1,902	-8	-0.4%	YES	-	-	0%	YES	-	-	0.2	YES
	4007		1,910	1,919	9	0.5%	YES	-	-	0%	YES	-	-	0.2	YES
	4008		1,910	1,918	8	0.4%	YES	-	-	0%	YES	-	-	0.2	YES
	4009		1,720	1,723	3	0.2%	YES	-	-	0%	YES	-	-	0.1	YES
	4011	I-295 SB off-ramp to Greenland Rd./I-295 NB off-ramp	2,130	2,105	-25	-1.2%	YES	-	-	-1%	YES	-	-	0.5	YES
	4012		1,050	1,045	-5	-0.5%	YES	-	-	0%	YES	-	-	0.1	YES
	4015	Greenland Rd./I-295 NB off-ramp to Business Park Blvd.	1,530	1,494	-36	-2.3%	YES	-	-	-2%	YES	-	-	0.9	YES
	4018	Business Park Blvd. to End Project	1,490	1,485	-5	-0.4%	YES	-	-	0%	YES	-	-	0.1	YES
US 1 (Philips Hwy.) Westbound	2001	Begin Project to Business Park Blvd.	1,550	1,540	-10	-0.7%	YES	-	-	-1%	YES	-	-	0.3	YES
	2002		1,500	1,466	-34	-2.3%	YES	-	-	-2%	YES	-	-	0.9	YES
	2005	Business Park Blvd.to Greenland Rd./I-295 NB off-ramp	1,800	1,767	-33	-1.9%	YES	-	-	-2%	YES	-	-	0.8	YES
	2006		1,800	1,784	-16	-0.9%	YES	-	-	-1%	YES	-	-	0.4	YES
	2010	Greenland Rd./I-295 NB off-ramp to I-295 SB off-ramp	1,630	1,529	-102	-6.2%	YES	-	-	-6%	YES	-	-	2.6	YES
	2011		1,060	1,017	-43	-4.0%	YES	-	-	-4%	YES	-	-	1.3	YES
	2013	I-295 SB off-ramp to SR 115 (Southside Blvd.)	1,170	1,109	-61	-5.2%	YES	-	-	-5%	YES	-	-	1.8	YES
	2014		1,170	1,093	-77	-6.6%	YES	-	-	-7%	YES	-	-	2.3	YES
	2018	SR 115 (Southside Blvd.) to End Project	1,260	1,188	-72	-5.7%	YES	-	-	-6%	YES	-	-	2.1	YES
SR 152 (Baymeadows Rd.) Eastbound	6001	Begin Project to Pointmeadows Rd.	1,370	1,316	-54	-3.9%	YES	-	-	-4%	YES	-	-	1.5	YES
	6003	Pointmeadows Rd. to I-295 SB off-ramp	1,260	1,222	-38	-3.0%	YES	-	-	-3%	YES	-	-	1.1	YES
	6005	I-295 SB off-ramp to I-295 NB off-ramp	1,150	1,067	-83	-7.2%	YES	-	-	-7%	YES	-	-	2.5	YES
	6007		380	357	-23	-6.0%	YES	-23	YES	-	-	-	-	1.2	YES
	6008	I-295 NB off-ramp to R.G. E. Skinner Blvd.	470	429	-41	-8.8%	YES	-41	YES	-	-	-	-	1.9	YES
	6011	R.G.E. Skinner Blvd. to End Project	340	329	-11	-3.1%	YES	-11	YES	-	-	-	-	0.6	YES
SR 152 (Baymeadows Rd.) Westbound	8001	Begin Project to R.G.E. Skinner Blvd.	550	540	-10	-1.8%	YES	-10	YES	-	-	-	-	0.4	YES
	8002		500	484	-16	-3.2%	YES	-16	YES	-	-	-	-	0.7	YES
	8005	R.G.E. Skinner Blvd. to I-295 NB off-ramp	650	625	-25	-3.8%	YES	-25	YES	-	-	-	-	1.0	YES
	8006	I-295 NB off-ramp to I-295 SB off-ramp	760	718	-42	-5.5%	YES	-	-	-5%	YES	-	-	1.5	YES
	8007		600	582	-18	-3.1%	YES	-18	YES	-	-	-	-	0.8	YES
	8009	I-295 SB off-ramp to Pointmeadows Rd.	1,250	1,076	-174	-13.9%	YES	-	-	-14%	YES	-	-	5.1	NO
	8011	Pointmeadows Rd. to End Project	1,390	1,307	-83	-5.9%	YES	-	-	-6%	YES	-	-	2.2	YES
Gate Pkwy. Eastbound	12001	Begin Project to Pointmeadows Rd.	1,610	1,599	-11	-0.7%	YES	-	-	-1%	YES	-	-	0.3	YES
	12002		1,310	1,287	-23	-1.7%	YES	-	-	-2%	YES	-	-	0.6	YES
	12004	Pointmeadows Rd. to I-295 SB off-ramp	1,430	1,343	-87	-6.1%	YES	-	-	-6%	YES	-	-	2.3	YES
	12006	I-295 SB off-ramp to I-295 NB off-ramp	920	865	-55	-6.0%	YES	-	-	-6%	YES	-	-	1.8	YES
	12007		570	535	-35	-6.2%	YES	-35	YES	-	-	-	-	1.5	YES
	12009	I-295 NB off-ramp to End Project	590	546	-44	-7.4%	YES	-44	YES	-	-	-	-	1.8	YES
	12010		590	565	-25	-4.2%	YES	-25	YES	-	-	-	-	1.0	YES
Gate Pkwy. Westbound	14001	Begin Project to I-295 NB off-ramp	370	347	-23	-6.3%	YES	-23	YES	-	-	-	-	1.2	YES
	14002	I-295 NB off-ramp to I-295 SB off-ramp	300	301	1	0.4%	YES	1	YES	-	-	-	-	0.1	YES
	14003		260	264	4	1.6%	YES	4	YES	-	-	-	-	0.3	YES
	14005	I-295 SB off-ramp to Pointmeadows Rd.	800	707	-93	-11.7%	YES	-	-	-12%	YES	-	-	3.4	YES
	14006		530	493	-37	-7.0%	YES	-37	YES	-	-	-	-	1.6	YES
	14009	Pointmeadows Rd. to End Project	660	633	-27	-4.1%	YES	-27	YES	-	-	-	-	1.1	YES
Town Center Pkwy./UNF Dr. Eastbound	20001	Begin Project to St. Johns Bluff Rd.	1,730	1,731	1	0.0%	YES	-	-	0%	YES	-	-	0.0	YES
	20002		1,200	1,201	1	0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	20005	St. Johns Bluff Rd. to I-295	1,530	1,524	-6	-0.4%	YES	-	-	0%	YES	-	-	0.1	YES
	20006		170	170	-1	-0.3%	YES	-1	YES	-	-	-	-	0.0	YES
	20010	I-295 to Eco Dr.	410	403	-7	-1.6%	YES	-7	YES	-	-	-	-	0.3	YES
	20012		310	235	-75	-24.3%	NO	-75	YES	-	-	-	-	4.6	YES
	20014	Eco Dr. to End Project	170	162	-8	-4.6%	YES	-8	YES	-	-	-	-	0.6	YES
Town Center Pkwy./UNF Dr. Westbound	22001	Begin Project to Eco Dr.	320	309	-11	-3.4%	YES	-11	YES	-	-	-	-	0.6	YES
	22003	Eco Dr. to I-295	670	655	-15	-2.2%	YES	-15	YES	-	-	-	-	0.6	YES
	22004		670	640	-30	-4.5%	YES	-30	YES	-	-	-	-	1.2	YES
	22006		170	167	-3	-2.0%	YES	-3	YES	-	-	-	-	0.3	YES
	22008	I-295 to St. Johns Bluff Rd.	710	728	18	2.6%	YES	-	-	3%	YES	-	-	0.7	YES
	22011	St. Johns Bluff Rd. to End Project	1,000	979	-21	-2.1%	YES	-	-	-2%	YES	-	-	0.7	YES
Gate Pkwy. Southbound	35001	Begin Project to Town Center Pkwy.	2,230	2,238	8	0.3%	YES	-	-	0%	YES	-	-	0.2	YES
	35002		1,400	1,380	-20	-1.4%	YES	-	-	-1%	YES	-	-	0.5	YES
	35004	Town Center Pkwy. to Costco/Skinner Lake Dr.	1,930	1,890	-40	-2.1%	YES	-	-	-2%	YES	-	-	0.9	YES
	35005		1,850	1,837	-14	-0.7%	YES	-	-	-1%	YES	-	-	0.3	YES
	35007	Costco/Skinner Lake Dr. to I-295	2,050	2,029	-21	-1.0%	YES	-	-	-1%	YES	-	-	0.5	YES
	35008	I-295 to Deerwood Park Blvd.	1,470	1,447	-23	-1.6%	YES	-	-	-2%	YES	-	-	0.6	YES
	35011		1,190	1,168	-22	-1.9%	YES	-	-	-2%	YES	-	-	0.6	YES
	35012	Deerwood Park Blvd. to End Project	1,300	1,286	-14	-1.1%	YES	-	-	-1%	YES	-	-	0.4	YES
Gate Pkwy. Northbound	33001	Begin Project to Deerwood Park Blvd.	830	784	-46	-5.6%	YES	-	-	-6%	YES	-	-	1.6	YES
	33003	Deerwood Park Blvd. to I-295	2,090	2,038	-52	-2.5%	YES	-	-	-2%	YES	-	-	1.1	YES
	33004	I-295 to Costco/Skinner Lake Dr.	770	738	-32	-4.2%	YES	-	-	-4%	YES	-	-	1.2	YES
	33007		1,320	1,286	-34	-2.6%	YES	-	-	-3%	YES	-	-	0.9	YES
	33011	Costco/Skinner Lake Dr. to Town Center Pkwy.	1,580	1,542	-38	-2.4%	YES	-	-	-2%	YES	-	-	1.0	YES
	33014	Town Center Pkwy. to End Project	980	976	-4	-0.4%	YES	-	-	0%	YES	-	-	0.1	YES
Kernan Blvd. Southbound	39001	Begin Project to 1st Tech Pkwy.	770	767	-3	-0.4%	YES	-	-	0%	YES	-	-	0.1	YES
	39003	1st Tech Pkwy. to SR 202 (JTB)	170	227	57	33.4%	NO	57	YES	-	-	-	-	4.0	YES
	39008	SR 202 (JTB) to End. Project	20	9	-11	-53.1%	NO	-11	YES	-	-	-	-	2.8	YES
Kernan Blvd. Northbound	37001	Begin Project to SR 202 (JTB)	20	15	-5	-25.4%	NO	-5	YES	-	-	-	-	1.2	YES
	37005	SR 202 (JTB) to 1st Tech Pkwy.	2,000	1,988	-12	-0.6%	YES	-	-	-1%	YES	-	-	0.3	YES
	37007	1st Tech Pkwy. To End Project	2,010	1,919	-91	-4.5%	YES	-	-	-5%	YES	-	-	2.1	YES
		Total	**440,950**	**423,626**	**-17,324**										
		Sum of all Link Flows		**-3.93%**											
		Sum of all Link Flows (Flows within 5%)		**YES**											
		Total Count				199	195	41	41	90	90	68	58	199	175
		Individual Links				**98%**		**100%**		**100%**		**85%**		**88%**	
		Individual Links (Flows met for >85% Cases and GEH Statistic < 5 for 85% of Cases)				**YES**		**YES**		**YES**		**YES**		**YES**	

TABLE A6: Hourly flow comparisons – PM peak period - hour 3

Roadway	VISSIM Link Number	Location	Peak Hour Count Volume (vph)	VISSIM Model Volume (vph)	Difference	Within 15%	Criteria Met	Individual Link Flow < 700 vph		700 vph to 2,700 vph		> 2,700 vph		GEH Statistic	GEH Criteria Met (<= 5)
								Within 100vph	Criteria Met	Within 15%	Criteria Met	Within 400vph	Criteria Met		
I-295 Northbound	1001	Upstream of I-95 on-ramps	1,397	1,406	9	0.6%	YES	-	-	1%	YES	-	-	0.2	YES
	1002	Upstream of S.R. 5 (US 1) off-ramp	2,561	2,571	10	0.4%	YES	-	-	0%	YES	-	-	0.2	YES
	1003	Between S.R. 5 (US 1) off- and on-ramps	1,963	1,976	13	0.7%	YES	-	-	1%	YES	-	-	0.3	YES
	1004	S.R. 5 (US 1) on-ramp merge	2,686	2,686	0	0.0%	YES	-	-	0%	YES	-	-	0.0	YES
	1005	Downstream of S.R. 5 (US 1) on-ramp merge	2,686	2,684	-2	-0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	1006	3-lane to 2-lane merge - North of S.R. 5 (US 1)	2,686	2,690	4	0.1%	YES	-	-	0%	YES	-	-	0.1	YES
	1007	Upstream of S.R. 9B on-ramp	2,686	2,707	21	0.8%	YES	-	-	1%	YES	-	-	0.4	YES
	1008	S.R. 9B on-ramp merge	3,301	3,348	47	1.4%	YES	-	-	-	-	47	YES	0.8	YES
	1009	Downstream of S.R. 9B on-ramp merge - 3-lane section	3,301	3,339	38	1.1%	YES	-	-	-	-	38	YES	0.7	YES
	1010	Downstream of S.R. 9B on-ramp merge - 2-lane section	3,301	3,356	55	1.7%	YES	-	-	-	-	55	YES	1.0	YES
	1011	Upstream of S.R. 152 (Baymeadows Rd.) off-ramp	3,301	3,372	71	2.1%	YES	-	-	-	-	71	YES	1.2	YES
	1012	Between S.R. 152 (Baymeadows Rd.) off- and on-ramps	3,010	3,099	89	3.0%	YES	-	-	-	-	89	YES	1.6	YES
	1013	Between S.R. 152 on-ramp and Gate Pkwy. off-ramp	3,726	3,853	127	3.4%	YES	-	-	-	-	127	YES	2.1	YES
	1014	Between Gate Pkwy. off- and on-ramps	3,543	3,676	133	3.8%	YES	-	-	-	-	133	YES	2.2	YES
	1015	Between Gate Pkwy. on-ramp and S.R. 202 (JTB) off-ramp	4,058	4,207	149	3.7%	YES	-	-	-	-	149	YES	2.3	YES
	1016	Downstream of of S.R. 202 (JTB) off-ramp - 3-lane section	2,270	2,394	124	5.4%	YES	-	-	5%	YES	-	-	2.6	YES
	1017	Downstream of of S.R. 202 (JTB) off-ramp - 2-lane section	2,270	2,454	184	8.1%	YES	-	-	8%	YES	-	-	3.8	YES
	1018	Eastbound S.R. 202 (JTB) on-ramp merge	3,360	3,690	330	9.8%	YES	-	-	-	-	330	YES	5.6	NO
	1019	Downstream of Eastbound S.R. 202 (JTB) on-ramp merge	3,360	3,690	330	9.8%	YES	-	-	-	-	330	YES	5.6	NO
	1020	Between WB S.R. 202 (JTB) on-ramp and Town Center Pkwy. off-ramp	4,141	4,498	357	8.6%	YES	-	-	-	-	357	YES	5.4	NO
	1021	Between Town Center Pkwy. off- and on-ramps	3,626	3,946	320	8.8%	YES	-	-	-	-	320	YES	5.2	NO
	1022	Town Center Pkwy. on-ramp merge	4,383	4,739	356	8.1%	YES	-	-	-	-	356	YES	5.3	NO
I-295 Northbound - Ramps	1501	I-95 Southbound on-ramp	391	390	-1	-0.3%	YES	-1	YES	-	-	-	-	0.1	YES
	1502	I-95 Northbound on-ramp	773	774	1	0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	1503	I-95 on-ramp (combined) - 2-lane section	1,164	1,164	0	0.0%	YES	-	-	0%	YES	-	-	0.0	YES
	1504	I-95 on-ramp (combined) - 1-lane section	1,164	1,159	-5	-0.4%	YES	-	-	0%	YES	-	-	0.1	YES
	1505	US-1 Off-Ramp	599	602	3	0.5%	YES	3	YES	-	-	-	-	0.1	YES
	1513	US-1 On-Ramp	723	709	-14	-1.9%	YES	-	-	-2%	YES	-	-	0.5	YES
	1514	Baymeadows Rd. Off-Ramp	291	293	2	0.7%	YES	2	YES	-	-	-	-	0.1	YES
	1520	Baymeadows Rd. On-Ramp	715	732	17	2.4%	YES	-	-	2%	YES	-	-	0.6	YES
	1521	Gate Pkwy. Off-Ramp	183	191	8	4.1%	YES	8	YES	-	-	-	-	0.5	YES
	1527	Gate Pkwy. On-Ramp	516	521	5	1.0%	YES	5	YES	-	-	-	-	0.2	YES
	1528	SR 202 (JTB) Off-Ramp (combined)	1,788	1,856	68	3.8%	YES	-	-	4%	YES	-	-	1.6	YES
	1529	SR 202 (JTB) WB Off-Ramp	225	232	7	3.1%	YES	7	YES	-	-	-	-	0.5	YES
	1530	SR 202 (JTB) EB Off-Ramp	1,563	1,628	65	4.2%	YES	-	-	4%	YES	-	-	1.6	YES
	1531	SR 202 (JTB) On-Ramp - 2-lane section	2,170	2,196	26	1.2%	YES	-	-	1%	YES	-	-	0.6	YES
	1532	SR 202 (JTB) On-Ramp - 1-lane section	2,170	2,198	28	1.3%	YES	-	-	1%	YES	-	-	0.6	YES
	1533	On-Ramp from WB SR 202 (JTB)	782	790	8	1.1%	YES	-	-	1%	YES	-	-	0.3	YES
	1534	On-Ramp from EB SR 202 (JTB)	1,089	1,187	98	9.0%	YES	-	-	9%	YES	-	-	2.9	YES
	1535	Town Center Pkwy Off-Ramp	516	557	41	8.0%	YES	41	YES	-	-	-	-	1.8	YES
	1539	Town Center Pkwy On-Ramp	757	767	10	1.3%	YES	-	-	1%	YES	-	-	0.4	YES
I-295 Southbound	3001	Upstream of Town Center Pkwy. off-ramp	2,994	3,010	16	0.5%	YES	-	-	-	-	16	YES	0.3	YES
	3002	Between Town Center Pkwy. off- and on-ramps	2,586	2,615	29	1.1%	YES	-	-	1%	YES	-	-	0.6	YES
	3003	Between Town Center Pkwy. On-ramp and S.R. 202 (JTB) off-ramp (combined)	3,376	3,419	43	1.3%	YES	-	-	-	-	43	YES	0.7	YES
	3004	Between S.R. 202 (JTB) off-ramp (combined) and Westbound S.R. 202 (JTB) on-ramp	2,187	2,232	45	2.1%	YES	-	-	2%	YES	-	-	1.0	YES
	3005	Westbound S.R. 202 (JTB) on-ramp merge	3,576	3,732	156	4.4%	YES	-	-	-	-	156	YES	2.6	YES
	3006	Downstream of Westbound S.R. 202 (JTB) on-ramp merge	3,576	3,675	99	2.8%	YES	-	-	-	-	99	YES	1.6	YES
	3007	Between Eastbound S.R. 202 (JTB) on-ramp and Gate Pkwy. off-ramp	4,183	4,441	258	6.2%	YES	-	-	-	-	258	YES	3.9	YES
	3008	Between Gate Pkwy off- and on- ramps - 4-lane section	3,418	3,661	243	7.1%	YES	-	-	-	-	243	YES	4.1	YES
	3009	Between Gate Pkwy off- and on- ramps - 3-lane section	3,418	3,659	241	7.0%	YES	-	-	-	-	241	YES	4.0	YES
	3010	Between Gate Pkwy off- and on- ramps - 2-lane section	3,418	3,670	252	7.4%	YES	-	-	-	-	252	YES	4.2	YES
	3011	Between Gate Pkwy. on-ramp and S.R. 152 (Baymeadows Rd.) off-ramp	4,191	4,470	279	6.6%	YES	-	-	-	-	279	YES	4.2	YES
	3012	Between S.R. 152 (Baymeadows Rd.) off- and on-ramps	3,543	3,782	239	6.7%	YES	-	-	-	-	239	YES	3.9	YES
	3013	S.R. 152 (Baymeadows Rd.) on-ramp merge	3,875	4,053	178	4.6%	YES	-	-	-	-	178	YES	2.8	YES
	3014	Downstream of S.R. 152 (Baymeadows Rd.) on-ramp merge	3,875	4,106	231	6.0%	YES	-	-	-	-	231	YES	3.7	YES
	3015	Upstream of S.R. 9B off-ramp - 2-lane section	3,875	4,115	240	6.2%	YES	-	-	-	-	240	YES	3.8	YES
	3016	Upstream of S.R. 9B off-ramp - 3-lane section	3,875	4,124	249	6.4%	YES	-	-	-	-	249	YES	3.9	YES
	3017	Downstraem of S.R. 9B off-ramp - 2-lane section	3,060	3,259	199	6.5%	YES	-	-	-	-	199	YES	3.5	YES
	3018	Downstraem of S.R. 9B off-ramp - 3-lane section	3,060	3,261	201	6.6%	YES	-	-	-	-	201	YES	3.6	YES
	3019	Upstream of S.R. 5 (US 1) off-ramp - 3-lane section	3,060	3,259	199	6.5%	YES	-	-	-	-	199	YES	3.5	YES
	3020	Upstream of S.R. 5 (US 1) off-ramp - 4-lane section	3,060	3,253	193	6.3%	YES	-	-	-	-	193	YES	3.4	YES
	3021	Between S.R. 5 (US 1) off- and on-ramps	2,628	2,803	175	6.7%	YES	-	-	7%	YES	-	-	3.4	YES
	3022	Downstream of S.R. 5 (US 1) on-ramp	3,260	3,448	188	5.8%	YES	-	-	-	-	188	YES	3.2	YES
	3023	Upstream of Northbound I-95 off-ramp	3,260	3,437	177	5.4%	YES	-	-	-	-	177	YES	3.1	YES
	3024	Downstream of Northbound I-95 off-ramp	3,085	3,267	182	5.9%	YES	-	-	-	-	182	YES	3.2	YES
	3025	Upstream of Southbound I-95 off-ramp - 3-lane section	3,085	3,272	187	6.1%	YES	-	-	-	-	187	YES	3.3	YES
	3026	Upstream of Southbound I-95 off-ramp - 4-lane section	3,085	3,268	183	5.9%	YES	-	-	-	-	183	YES	3.2	YES
	3027	Downstream of Southbound I-95 off-ramp - 3-lane section	1,963	2,069	106	5.4%	YES	-	-	5%	YES	-	-	2.4	YES
	3028	Downstream of Southbound I-95 off-ramp - 2-lane section	1,963	2,110	147	7.5%	YES	-	-	7%	YES	-	-	3.3	YES
I-295 Southbound - Ramps	3501	Town Center Pkwy Off-Ramp	407	406	-1	-0.1%	YES	-1	YES	-	-	-	-	0.0	YES
	3505	Town Center Pkwy On-Ramp	790	791	1	0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	3506	SR 202 (JTB) Off-Ramp (combined)	1,189	1,189	0	0.0%	YES	-	-	0%	YES	-	-	0.0	YES
	3507	SR 202 (JTB) WB On-Ramp	632	638	6	1.0%	YES	6	YES	-	-	-	-	0.3	YES
	3508	SR 202 (JTB) EB On-Ramp	557	558	1	0.1%	YES	1	YES	-	-	-	-	0.0	YES
	3509	On-Ramp from WB SR 202 (JTB)	1,389	1,434	45	3.2%	YES	-	-	3%	YES	-	-	1.2	YES
	3510	On-Ramp from EB SR 202 (combined)	1,696	1,761	65	3.8%	YES	-	-	4%	YES	-	-	1.6	YES
	3511	On-Ramp from EB SR 202 (JTB)	607	620	13	2.2%	YES	13	YES	-	-	-	-	0.5	YES
	3512	Gate Pkwy. Off-Ramp	765	818	53	7.0%	YES	-	-	7%	YES	-	-	1.9	YES
	3516	Gate Pkwy. On-Ramp - 2-lane segment	773	775	2	0.2%	YES	-	-	0%	YES	-	-	0.1	YES
	3517	Gate Pkwy. On-Ramp - 1-lane segment	773	782	9	1.1%	YES	-	-	1%	YES	-	-	0.3	YES
	3518	Baymeadows Rd. Off-Ramp	649	686	37	5.7%	YES	37	YES	-	-	-	-	1.4	YES
	3523	Baymeadows Rd. On-Ramp	333	332	-1	-0.4%	YES	-1	YES	-	-	-	-	0.1	YES
	3524	US-1 Off-Ramp	432	457	25	5.8%	YES	25	YES	-	-	-	-	1.2	YES
	3530	US-1 On-Ramp - 2-lane segment	632	631	-1	-0.2%	YES	-1	YES	-	-	-	-	0.0	YES
	3531	US-1 On-Ramp - 1-lane segment	632	629	-3	-0.4%	YES	-3	YES	-	-	-	-	0.1	YES
	3532	I-95 NB Off-Ramp	175	187	12	6.7%	YES	12	YES	-	-	-	-	0.9	YES
	3533	I-95 SB Off-Ramp	1,123	1,197	74	6.6%	YES	-	-	7%	YES	-	-	2.2	YES
S.R. 9B	17001	Northbound S.R. 9B	615	620	5	0.8%	YES	5	YES	-	-	-	-	0.2	YES
	19001	Southbound S.R. 9B	815	867	52	6.3%	YES	-	-	6%	YES	-	-	1.8	YES
S.R. 202 (JTB) Eastbound	16001	Upstream of Gate Pkwy. off-ramp - Section 1	5,198	5,181	-17	-0.3%	YES	-	-	-	-	-17	YES	0.2	YES
	16002	Upstream of Gate Pkwy. off-ramp - Section 2	5,198	5,200	2	0.0%	YES	-	-	-	-	2	YES	0.0	YES
	16003	Between Gate Pkwy. off- and on-ramps	4,491	4,522	31	0.7%	YES	-	-	-	-	31	YES	0.5	YES
	16004	Between Gate Pkwy. on-ramp and I-295 off-ramp	5,921	6,027	106	1.8%	YES	-	-	-	-	106	YES	1.4	YES
	16005	Between I-295 off-ramp and Southbound I-295 on-ramp	4,225	4,312	87	2.1%	YES	-	-	-	-	87	YES	1.3	YES
	16006	Southbound I-295 on-ramp merge	4,782	4,861	79	1.7%	YES	-	-	-	-	79	YES	1.1	YES
	16007	Downstream of Southbound I-295 on-ramp merge	4,782	4,761	-21	-0.4%	YES	-	-	-	-	-21	YES	0.3	YES
	16008	Between Northbound I-295 on-ramp and Kernan Blvd. off-ramp	6,345	6,550	205	3.2%	YES	-	-	-	-	205	YES	2.6	YES
	16009	Between Kernan Blvd. off- and on- ramp - 4-lane section	4,832	4,987	155	3.2%	YES	-	-	-	-	155	YES	2.2	YES
	16010	Between Kernan Blvd. off- and on- ramp - 3-lane section	4,832	4,987	155	3.2%	YES	-	-	-	-	155	YES	2.2	YES
	16011	Kernan Blvd. on-ramp merge - 4-lane section	4,981	5,133	152	3.1%	YES	-	-	-	-	152	YES	2.1	YES
	16012	Kernan Blvd. on-ramp merge - 3-lane section	4,981	5,063	82	1.6%	YES	-	-	-	-	82	YES	1.2	YES
	16013	Downstream of Kernan Blvd. on-ramp	4,981	5,195	214	4.3%	YES	-	-	-	-	214	YES	3.0	YES
S.R. 202 (JTB) Eastbound - Ramps	16501	Gate Pkwy. Off-Ramp	707	706	-1	-0.2%	YES	-	-	0%	YES	-	-	0.1	YES
	16505	Gate Pkwy. On-Ramp	1,430	1,447	17	1.2%	YES	-	-	1%	YES	-	-	0.4	YES
	16506	Kernan Blvd. Off-Ramp	1,514	1,592	78	5.2%	YES	-	-	5%	YES	-	-	2.0	YES
	16510	Kernan Blvd. On-Ramp	150	151	1	0.5%	YES	1	YES	-	-	-	-	0.1	YES
S.R. 202 (JTB) Westbound	18001	Upstream of Kernan Blvd. off-ramp - section 1	3,933	3,947	14	0.3%	YES	-	-	-	-	14	YES	0.2	YES
	18002	Upstream of Kernan Blvd. off-ramp - section 2	3,933	3,966	33	0.8%	YES	-	-	-	-	33	YES	0.5	YES
	18003	Between Kernan Blvd. off- and on-ramps	3,642	3,671	29	0.8%	YES	-	-	-	-	29	YES	0.5	YES
	18004	Between Kernan Blvd. on-ramp and I-295 off-ramp (combined)	4,208	4,262	54	1.3%	YES	-	-	-	-	54	YES	0.8	YES
	18005	Between I-295 off-ramp (combined) and Northbound I-295 on-ramp	2,037	2,077	40	2.0%	YES	-	-	2%	YES	-	-	0.9	YES
	18006	Northbound I-295 on-ramp merge - 5-lane section	2,262	2,304	42	1.9%	YES	-	-	2%	YES	-	-	0.9	YES
	18007	Northbound I-295 on-ramp merge - 4-lane section	2,262	2,305	43	1.9%	YES	-	-	2%	YES	-	-	0.9	YES
	18008	Between Southbound I-295 on-ramp and Gate Pkwy. Off-ramp	2,894	2,965	71	2.5%	YES	-	-	-	-	71	YES	1.3	YES
	18009	Between Gate Pkwy. off- and on-ramps	2,345	2,415	70	3.0%	YES	-	-	3%	YES	-	-	1.4	YES
	18010	Gate Pkwy. on-ramp merge - 5-lane section	3,185	3,260	75	2.3%	YES	-	-	-	-	75	YES	1.3	YES
	18011	Gate Pkwy. on-ramp merge - 4-lane section	3,185	3,303	118	3.7%	YES	-	-	-	-	118	YES	2.1	YES
S.R. 202 (JTB) Westbound - Ramps	18501	Kernan Blvd. Off-Ramp	291	301	10	3.3%	YES	10	YES	-	-	-	-	0.6	YES
	18505	Kernan Blvd. On-Ramp	565	570	5	0.9%	YES	5	YES	-	-	-	-	0.2	YES
	18506	Gate Pkwy. Off-Ramp	549	555	6	1.2%	YES	6	YES	-	-	-	-	0.3	YES
	18511	Gate Pkwy. On-Ramp	840	843	3	0.3%	YES	-	-	0%	YES	-	-	0.1	YES

TABLE A6 CONTD.: Hourly flow comparisons – PM peak period - hour 3

Roadway	VISSIM Link Number	Location	Peak Hour Count Volume (vph)	VISSIM Model Volume (vph)	Difference	Within 15%	Criteria Met	Individual Link Flow						GEH Statistic	GEH Criteria Met (<= 5)
								< 700 vph		700 vph to 2,700 vph		> 2,700 vph			
								Within 100vph	Criteria Met	Within 15%	Criteria Met	Within 400vph	Criteria Met		
US 1 (Philips Hwy.) Eastbound	4001	Begin Project to SR 115 (Southside Blvd.)	1,222	1,206	-16	-1.3%	YES	-	-	-1%	YES	-	-	0.4	YES
	4002		881	873	-8	-0.9%	YES	-	-	-1%	YES	-	-	0.3	YES
	4006	SR 115 (Southside Blvd.) to I-295 SB off-ramp	1,588	1,595	7	0.4%	YES	-	-	0%	YES	-	-	0.2	YES
	4007		1,588	1,599	11	0.7%	YES	-	-	1%	YES	-	-	0.3	YES
	4008		1,588	1,596	8	0.5%	YES	-	-	1%	YES	-	-	0.2	YES
	4009		1,430	1,421	-9	-0.6%	YES	-	-	-1%	YES	-	-	0.2	YES
	4011	I-295 SB off-ramp to Greenland Rd./I-295 NB off-ramp	1,771	1,785	14	0.8%	YES	-	-	1%	YES	-	-	0.3	YES
	4012		873	887	14	1.5%	YES	-	-	2%	YES	-	-	0.5	YES
	4015	Greenland Rd./I-295 NB off-ramp to Business Park Blvd.	1,272	1,260	-12	-1.0%	YES	-	-	-1%	YES	-	-	0.4	YES
	4018	Business Park Blvd. to End Project	1,239	1,258	19	1.5%	YES	-	-	2%	YES	-	-	0.5	YES
US 1 (Philips Hwy.) Westbound	2001	Begin Project to Business Park Blvd.	1,289	1,292	3	0.3%	YES	-	-	0%	YES	-	-	0.1	YES
	2002		1,247	1,232	-15	-1.2%	YES	-	-	-1%	YES	-	-	0.4	YES
	2005	Business Park Blvd.to Greenland Rd./I-295 NB off-ramp	1,497	1,491	-6	-0.4%	YES	-	-	0%	YES	-	-	0.1	YES
	2006		1,497	1,505	8	0.6%	YES	-	-	1%	YES	-	-	0.2	YES
	2010	Greenland Rd./I-295 NB off-ramp to I-295 SB off-ramp	1,356	1,302	-54	-4.0%	YES	-	-	-4%	YES	-	-	1.5	YES
	2011		881	852	-29	-3.3%	YES	-	-	-3%	YES	-	-	1.0	YES
	2013	I-295 SB off-ramp to SR 115 (Southside Blvd.)	973	939	-34	-3.5%	YES	-	-	-3%	YES	-	-	1.1	YES
	2014		973	972	-1	-0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	2018	SR 115 (Southside Blvd.) to End Project	1,048	1,052	4	0.4%	YES	-	-	0%	YES	-	-	0.1	YES
SR 152 (Baymeadows Rd.) Eastbound	6001	Begin Project to Pointmeadows Rd.	1,139	1,096	-43	-3.8%	YES	-	-	-4%	YES	-	-	1.3	YES
	6003	Pointmeadows Rd. to I-295 SB off-ramp	1,048	1,035	-13	-1.2%	YES	-	-	-1%	YES	-	-	0.4	YES
	6005	I-295 SB off-ramp to I-295 NB off-ramp	956	931	-25	-2.7%	YES	-	-	-3%	YES	-	-	0.8	YES
	6007		316	324	8	2.6%	YES	8	YES	-	-	-	-	0.5	YES
	6008	I-295 NB off-ramp to R.G. E. Skinner Blvd.	391	382	-9	-2.2%	YES	-9	YES	-	-	-	-	0.4	YES
	6011	R.G.E. Skinner Blvd. to End Project	283	291	8	2.7%	YES	8	YES	-	-	-	-	0.4	YES
SR 152 (Baymeadows Rd.) Westbound	8001	Begin Project to R.G.E. Skinner Blvd.	457	458	1	0.2%	YES	1	YES	-	-	-	-	0.1	YES
	8002		416	413	-3	-0.8%	YES	-3	YES	-	-	-	-	0.2	YES
	8005	R.G.E. Skinner Blvd. to I-295 NB off-ramp	541	539	-2	-0.3%	YES	-2	YES	-	-	-	-	0.1	YES
	8006	I-295 NB off-ramp to I-295 SB off-ramp	632	622	-10	-1.6%	YES	-10	YES	-	-	-	-	0.4	YES
	8007		499	503	4	0.8%	YES	4	YES	-	-	-	-	0.2	YES
	8009	I-295 SB off-ramp to Pointmeadows Rd.	1,040	997	-43	-4.2%	YES	-	-	-4%	YES	-	-	1.4	YES
	8011	Pointmeadows Rd. to End Project	1,156	1,199	43	3.7%	YES	-	-	4%	YES	-	-	1.3	YES
Gate Pkwy. Eastbound	12001	Begin Project to Pointmeadows Rd.	1,339	1,335	-4	-0.3%	YES	-	-	0%	YES	-	-	0.1	YES
	12002		1,089	1,074	-15	-1.3%	YES	-	-	-1%	YES	-	-	0.4	YES
	12004	Pointmeadows Rd. to I-295 SB off-ramp	1,189	1,121	-68	-5.8%	YES	-	-	-6%	YES	-	-	2.0	YES
	12006	I-295 SB off-ramp to I-295 NB off-ramp	765	758	-7	-0.9%	YES	-	-	-1%	YES	-	-	0.3	YES
	12007		474	486	12	2.5%	YES	12	YES	-	-	-	-	0.5	YES
	12009	I-295 NB off-ramp to End Project	491	496	5	0.9%	YES	5	YES	-	-	-	-	0.2	YES
	12010		491	510	19	3.9%	YES	19	YES	-	-	-	-	0.9	YES
Gate Pkwy. Westbound	14001	Begin Project to I-295 NB off-ramp	308	295	-13	-4.3%	YES	-13	YES	-	-	-	-	0.8	YES
	14002	I-295 NB off-ramp to I-295 SB off-ramp	249	257	8	3.2%	YES	8	YES	-	-	-	-	0.5	YES
	14003		216	226	10	4.6%	YES	10	YES	-	-	-	-	0.7	YES
	14005	I-295 SB off-ramp to Pointmeadows Rd.	665	665	0	0.0%	YES	0	YES	-	-	-	-	0.0	YES
	14006		441	468	27	6.1%	YES	27	YES	-	-	-	-	1.3	YES
	14009	Pointmeadows Rd. to End Project	549	577	28	5.2%	YES	28	YES	-	-	-	-	1.2	YES
Town Center Pkwy. Eastbound	20001	Begin Project to St. Johns Bluff Rd.	1,439	1,452	13	0.9%	YES	-	-	1%	YES	-	-	0.3	YES
	20002		998	1,010	12	1.2%	YES	-	-	1%	YES	-	-	0.4	YES
	20005	St. Johns Bluff Rd. to I-295	1,272	1,285	13	1.0%	YES	-	-	1%	YES	-	-	0.4	YES
	20006		141	143	2	1.4%	YES	2	YES	-	-	-	-	0.2	YES
	20010	I-295 to Eco Dr.	341	357	16	4.7%	YES	16	YES	-	-	-	-	0.9	YES
	20012		258	206	-52	-20.2%	NO	-52	YES	-	-	-	-	3.4	YES
	20014	*Eco Dr. to End Project*	141	146	5	3.5%	YES	5	YES	-	-	-	-	0.4	YES
Town Center Pkwy. Westbound	22001	Begin Project to Eco Dr.	266	263	-3	-1.3%	YES	-3	YES	-	-	-	-	0.2	YES
	22003	Eco Dr. to I-295	557	553	-4	-0.7%	YES	-4	YES	-	-	-	-	0.2	YES
	22004		557	542	-15	-2.6%	YES	-15	YES	-	-	-	-	0.6	YES
	22006		141	143	2	1.3%	YES	2	YES	-	-	-	-	0.2	YES
	22008	I-295 to St. Johns Bluff Rd.	590	665	75	12.8%	YES	75	YES	-	-	-	-	3.0	YES
	22011	St. Johns Bluff Rd. to End Project	832	871	39	4.7%	YES	-	-	5%	YES	-	-	1.3	YES
Gate Pkwy. Southbound	35001	Begin Project to Town Center Pkwy.	1,854	1,856	2	0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	35002		1,164	1,152	-12	-1.0%	YES	-	-	-1%	YES	-	-	0.3	YES
	35004	*Town Center Pkwy. to Costco/Skinner Lake Dr.*	1,605	1,585	-20	-1.3%	YES	-	-	-1%	YES	-	-	0.5	YES
	35005		1,538	1,542	4	0.3%	YES	-	-	0%	YES	-	-	0.1	YES
	35007	Costco/Skinner Lake Dr. to I-295	1,705	1,711	6	0.4%	YES	-	-	0%	YES	-	-	0.2	YES
	35008	I-295 to Deerwood Park Blvd.	1,222	1,237	15	1.2%	YES	-	-	1%	YES	-	-	0.4	YES
	35011		990	989	-1	-0.1%	YES	-	-	0%	YES	-	-	0.0	YES
	35012	Deerwood Park Blvd. to End Project	1,081	1,077	-4	-0.4%	YES	-	-	0%	YES	-	-	0.1	YES
Gate Pkwy. Northbound	33001	Begin Project to Deerwood Park Blvd.	690	650	-40	-5.7%	YES	-40	YES	-	-	-	-	1.5	YES
	33003	Deerwood Park Blvd. to I-295	1,738	1,677	-61	-3.5%	YES	-	-	-4%	YES	-	-	1.5	YES
	33004	I-295 to Costco/Skinner Lake Dr.	640	611	-29	-4.5%	YES	-29	YES	-	-	-	-	1.2	YES
	33007		1,098	1,075	-23	-2.1%	YES	-	-	-2%	YES	-	-	0.7	YES
	33011	Costco/Skinner Lake Dr. to Town Center Pkwy.	1,314	1,300	-14	-1.1%	YES	-	-	-1%	YES	-	-	0.4	YES
	33014	Town Center Pkwy. to End Project	815	826	11	1.4%	YES	-	-	1%	YES	-	-	0.4	YES
Kernan Blvd. Southbound	39001	Begin Project to 1st Tech Pkwy.	640	635	-5	-0.7%	YES	-5	YES	-	-	-	-	0.2	YES
	39003	1st Tech Pkwy. to SR 202 (JTB)	141	181	40	28.0%	NO	40	YES	-	-	-	-	3.1	YES
	39008	SR 202 (JTB) to End. Project	17	22	5	27.6%	NO	5	YES	-	-	-	-	1.1	YES
Kernan Blvd. Northbound	37001	Begin Project to SR 202 (JTB)	17	13	-4	-20.6%	NO	-4	YES	-	-	-	-	0.9	YES
	37005	SR 202 (JTB) to 1st Tech Pkwy.	1,663	1,852	189	11.4%	YES	-	-	11%	YES	-	-	4.5	YES
	37007	1st Tech Pkwy. To End Project	1,672	1,784	112	6.7%	YES	-	-	7%	YES	-	-	2.7	YES
Total			**366,685**	**377,178**	**10,493**										
Sum of all Link Flows				**2.86%**											
Sum of all Link Flows (Flows within 5%)				YES											
Total Count						199	195	54	54	89	89	56	56	199	194
Individual Links						**98%**		**100%**		**100%**		**100%**		**97%**	
Individual Links (Flows met for >85% Cases and GEH Statistic < 5 for 85% of Cases)						YES		YES		YES		YES		YES	