Effects of Urban Street Environment on Operating Speeds

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FOREWORD

Speeding –exceeding the posted speed limit or driving too fast for conditions – is a contributing factor in approximately 30% of all fatal crashes. Speeding is a complex problem, involving the interaction of many factors including public attitudes, road user behavior, vehicle performance, roadway design and characteristics, posted speed limits, enforcement strategies and judicial decisions. This report provides a review of existing speed models (and modeling techniques), common methods used to evaluate driver's perception of the road environment, and possible factors that may influence a driver's speed choice. This report will be of interest to researchers and State and local agencies with responsibility for speed management activities.

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16. Abstract

Speeds on low-speed urban arterials often exceed their intended operating speeds resulting in potential safety problems since speed is directly related to crash severity, especially for pedestrian-involved crashes. This research develops and calibrates a method for estimating operating speeds based on drivers' perceptions of design features, environmental factors, and operational conditions on low-speed urban roadways. The operating speed model development includes the selection of study corridors that represent comprehensive urban street characteristics, the collection of supplemental data, and statistical model development. This report also includes a review of existing speed models and modeling techniques, common methods used to evaluate driver's perception of the road environment, and possible factors that may influence a driver's speed choice. Low-speed urban streets, as defined in this research, include urban local streets, collectors, and arterials with speed limits less or equal to 45 mph.

This study utilizes one year (2004) of data from the Commute Atlanta project, where drivers in the Atlanta, Georgia region freely drove their personal vehicles equipped with data collection equipment. Speed data for free-flow conditions, however, is not straightforward since there is no clear way to determine if a vehicle is operating under free-flow conditions. As a result, this project also includes the development of an extensive free-flow speed filter process.

Mixed models are utilized for the estimation of speed conditions. It was found that the use of one robust speed model is not practical for evaluating operating speeds for free-flow conditions for low-speed urban street locations since roadside features have a stronger affect on two-lane, two-way roads than on their four-lane counterparts. Thus, the effort considers two lane and four lane facilities separately and tangent and horizontal curve segments separately.

The results from this research effort will aid researchers and designers in pinpointing current problems with the design process and overcoming these limitations using design principals based on appropriate operating speeds that address driver's perception and reaction to the road environment. The resulting models will provide additional insight into driver selected speeds at urban locations. Future urban street speed model development should benefit from the information contained in this report as it will enable researchers to target specific variable sensitivities.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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1 INTRODUCTION

To date, designers of urban roads use a design speed concept in which a minimum "suitable" speed is used for the design of streets and highways. Often, this design speed is based on a proposed functional classification or a proposed speed limit that pays little regard to the actual speed drivers will select when utilizing the facility. The design speed does not address maximum operating speed issues, but simply assures that minimum design criteria are achieved. A survey performed by Mustyn and Sheppard [(1)] found that more than 75 percent of the drivers interviewed claimed to drive a speed they felt was appropriate for the road, regardless of the speed limit. Similarly, the European Transport Safety Council[(2)] found road characteristics determine what is physically possible for a vehicle, but they also influence "what seems appropriate to a driver." Clearly, an understanding of what influences these "driver-selected speeds" is essential in assuring safe design of transportation facilities.

1.1 Objective and Overview

The objective of this research is to develop and calibrate a method for estimating operating speeds based on drivers' perceptions of design features, environmental factors, and operational conditions on low-speed urban roadways where operating speed is defined as the highest overall speed at which a driver can travel on a given road under favorable weather conditions and under prevailing traffic [(3)].

This report provides a review of existing speed models (and modeling techniques), common methods used to evaluate driver's perception of the road environment, and possible factors that may influence a driver's speed choice. In addition, this report summarizes the available database for this evaluation of operating speeds for low-speed urban streets. Low-speed urban streets, as defined in this research, include urban local streets, collectors, and arterials with speed limits less or equal to 45 mph. Speeds on these facilities often exceed their intended operating speeds potentially resulting in potential safety problems since speed is directly related to crash severity, especially for pedestrian-involved crashes.

The operating speed model development includes the selection of study corridors that represent comprehensive urban street characteristics, the collection of supplemental data (i.e., vehicle trip data, road environment characteristics, and vehicle and driver characteristics), and statistical model development. This study utilizes data for one year (2004) where drivers in the Atlanta, Georgia region freely drove their personal vehicles equipped with data collection equipment. The equipment and data collection process were part of the Commute Atlanta project and provided to this project as a courtesy. Speed data for free-flow conditions, however, is not straightforward since there is no clear way to determine if a vehicle is operating under free-flow conditions. As a result, this project includes the development of an extensive free-flow speed filter process.

Mixed models are utilized for the estimation of speed conditions. The use of one robust speed model is not practical for evaluating operating speeds for free-flow conditions at low-speed urban street locations since roadside features have a stronger affect on two-lane, two-way roads than on their four-lane counterparts. Thus, the effort ultimately considers two-lane and four-lane facilities separately as well as tangent and horizontal curve segments separately. It will be seen that most of the variables performed in an intuitive manner. For example, better sight distance corresponded to higher operating speeds.

1.2 Summary

Ultimately, the results from this research effort will aid designers and researchers in pinpointing current problems with the design process and overcoming these limitations using design principals based on appropriate operating speeds that address driver's perception and reaction to the road environment. The resulting models will provide additional insight into driver selected speeds at urban locations. Future urban street speed model development should benefit from the information contained in this report as it will enable researchers to target specific variable sensitivities. The information contained in this report will also be helpful to practitioners to enable them to better estimate expected free-flow speeds at the design stages for proposed urban roads.

The organization of this report is as follows. Chapter 2 provides a detailed literature review of factors influencing speed choice, existing operating speed models, and methods of evaluating driver's perception of the road environment. Chapter 3 provides an overview of the data utilized for this study. Included in Chapter 3 is a review of the Georgia Department of Transportation (GDOT) geographical information system (GIS) roadway data, supplemental field data collection, the corridor selection process, and a description of the instrumented vehicle data set. Chapter 4 next provides a step-by-step detail for the processing of the vehicle trajectory data. Chapters 5 and 6 then present the operating speed data analysis and operating speed models. Finally, Chapter 7 presents a summary of conclusions and findings.

2 LITERATURE REVIEW

2.1 Introduction

Low speed urban streets, as defined in this work plan, include urban local streets, collectors, minor arterials, and principle arterials with speed limits less than or equal to 45 mph. Low speed urban streets are designed to provide both access and mobility while accommodating multiple road users such as bicyclists, motor vehicles, and pedestrians. Lower operating speeds are generally desired on low speed urban streets to help balance the intended roadway function and provide a safer environment. Speeds on these facilities often exceed the intended operating speeds of the roadways. This can cause potential safety problems since speed is directly related to crash severity, especially for pedestrian involved crashes.

This literature review explores documented factors influencing speed choice (e.g. geometric characteristic, traffic volumes, and traffic control devices), existing operating speed models, and methods of evaluating driver's perception of the road environment. The review is current as of 2002. Drawing from the findings of this review this research effort will develop and conduct a data collection effort and calibrate a method for estimating operating speeds based on drivers' perceptions of design features, environmental factors, and operational conditions on low-speed urban roadways.

2.2 Factors Influencing Speed Choice

The Highway Capacity Manual (HCM) [4] indicates that the speed of vehicles on urban streets is influenced by the street environment, interaction among vehicles, and traffic control. Table 1 further identifies these influencing factors, as described in the HCM.

Table 1. Factors Influencing Vehicle Speed on Urban Streets

Street Environment	Interaction Among Vehicles	Traffic Control
Geometric Characteristics of the Facility	Traffic Density	Induced delays to traffic stream (signals and signs)
Character of Roadside Activity	Proportion of Trucks and Buses	
Adjacent Land Use	Turning Movements	

The HCM suggested influencing factors are generally geometric and operational variables, the HCM factors all fall under the broader category of "Physical Road Characteristics," as identified in past research and summarized by Openlander [5]. The HCM does not directly address environmental conditions or driver characteristics for urban streets and vehicle characteristics are loosely considered in the HCM evaluation of vehicle interactions. Numerous studies have identified each of these categories - physical road characteristics, environmental influences, vehicle characteristics, and driver characteristics - for defining the factors influencing vehicle speeds. The following sections of this review will address each of these in turn.

2.2.1 Physical Road Characteristics

Oppenlander [5] reviewed several studies to identify variables that influence vehicle speed. He found that the roadway characteristics with the most significant influence on observed operating speed include horizontal curvature, functional classification, length of grade, gradient, number of lanes and surface type. Sight distance, lateral clearance and frequency of intersections were also determined to influence vehicle speeds. His list of factors is consistent with those identified in similar studies. The following provides a brief discussion of these and other identified factors.

2.2.1.1 Functional Classification/Road Type

A Policy on Geometric Design of Highways and Streets [3] by the American Association of State Highway and Transportation Officials (AASHTO) suggests urban and rural functional systems should be classified separately due to fundamentally different characteristics. AASHTO further defines urban areas as places within boundaries with a population of 5,000 or more. If the population is 50,000 or larger, these regions can be further classified as urbanized areas. A hierarchy of functional classification generally includes principal arterials, minor arterials, collectors, and local roads and streets.

The HCM [4] indicates the urban environment street classes should be as further separated as follows:

- High Speed -- urban street with low driveway/access-point density, separate left-turn lanes, and no parking. Roadside development is low density and the speed limit for high speed streets is typically 72 to 88 km/h (45 to 55 mi/h).
- Suburban -- street with low driveway/access-point density, separate left-turn lanes, and no parking. Roadside development is low to medium density, and speed limits range from 64 to 72 km/h (40 to 45 mi/h).
- Intermediate -- urban street with a moderate driveway/access-point density, may have some separate or continuous left-turn lanes, and parking is permitted for portions of the road. Roadside development is higher than suburban streets and speed limits range from 48 to 64 km/h (30 to 40 mi/h).
- Urban -- streets with a high driveway/access-point density, parking may be permitted, there are few separate left-turn lanes, and possible pedestrian presence. Roadside development is dense with commercial uses and speed limits are 40 to 56 km/h (25 to 35 mi/h).

In the past, most urban speed analysis focused on speed conditions at interrupted locations like signalized intersections. A few evaluated corridor speed characteristics. A study by Ericsson [6], for example, compared driving patterns between and within different street configurations, traffic conditions, and types of drivers. There were four street types involved in this study: main street in a residential area, local feeder road in a residential area, radial arterial towards the city center, and streets in the city center. The researchers found that average speed was significantly different for all investigated street types. The radial arterial towards the city center experienced the highest average speed whereas streets in the city center had the lowest speeds. Driving patterns varied greatly among the different street type. The findings of this experiment indicate that the greatest influence on an individual's driving pattern was type of street followed by driver type.

Gattis and Watts [(7)] analyzed the relationship between urban street width and vehicle speed for six two-lane urban streets in Fayetteville, Arkansas. The findings suggested that street width might play a small role in vehicle speed, but other factors such as street function might be more significant determinants of the average and 85th percentile speeds. In fact, they tentatively suggested that elevated speeds appeared to be associated with uninterrupted travel distance opportunities rather than road type and width.

2.2.1.2 Geometric Characteristics

Physical road and roadside characteristics directly impact the operating speed a driver selects. In general, past research has included the following eight "geometric" categories that strongly influence operating speed:

- Horizontal Curvature,
- Vertical Grade (and Length of Grade),
- Available Sight Distance,
- Number of Lanes,
- Surface Type and Condition,
- Number of Access Points (Intersections/Driveways),
- Lateral Clearance, and
- Land Use Type and Density.

Kanellaidis [(8)] surveyed drivers to determine the factors influencing their choice of speed on suburban road curves. A total of 207 Greek drivers were asked to rate 14 elements of the road environment as to how important the factors influence their speed choice on the suburban road curves. Sight distance was the most significant factor whereas free roadside space and speed limit signage influences were perceived to be minimal. Analysis of the survey data indicated that speed choice on curves can be described by four road-environment factors: separation of opposing traffic, cross-section characteristics, alignment, and signage.

Ottesen and Krammes [(9)] studied the operating speeds on 138 horizontal curves and 78 approach tangents for 29 rural highways in 5 states. The researchers concluded that in addition to degree of curvature (radius), the length of curvature and deflection angle also significantly influenced vehicle speeds on curve. Kanellaidis, et al. [(10)] investigated the relationship between operating speed on curves and various geometric design parameters, including radius of curvature, desired speed, superelevation rate, lane width, shoulder width, and length of curve. They determined that the operating speed was strongly related to the horizontal curvature and the driver's desired speed.

Warren [11] suggested the most significant roadway characteristics to be curvature, grade, length of grade, number of lanes, surface condition, sight distance, lateral clearance, number of intersections, and built-up areas near the roadway. Tignor and Warren [12] additionally reported that the number of access points and nearby commercial development have the greatest influence on vehicle speeds.

Rowan and Keese [13] studied the operating speeds within the urban environment in 1962. He observed a substantial speed reduction when sight distance was below 300 to 360 m (984 to 1180 ft) at a curbed urban cross section. Though the adjacent land use appeared to influence a speed reduction, lateral restrictions influenced speed reduction more significantly than development density.

Cooper, et al. [14] found that average vehicle speeds increased by 2 km/h (1.6 mi/h) after resurfacing major roads in the United Kingdom; no change in traffic speed occurred in locations where surface unevenness remained the same after resurfacing. Parker [15] found no change in speeds on two rural highways and a 5 km/h (3 mi/h) increase on two urban streets that were resurfaced and subsequently subjected to an increased speed limit.

The European Transport Safety Council [2] reported that width, gradient, alignment and layout, and the consistency of these variables were the determinants of speed choice on a particular stretch of road. Road characteristics determine what is physically possible for a vehicle, but they also influence "what seems appropriate to a driver." In this regard, the interaction of all roadway geometric variables appears to play a more significant role upon driver selected speed than any one individual feature.

Tenkink [16] performed an experiment where subjects in a driving simulator drove a winding road. Each "driver" was asked to identify the highest possible safe speed. In one experiment, the researchers evaluated the subject's response to lead vehicle speed. "It concluded that uncertainty about the ability to respond adequately to lead vehicles, rather than uncertainty about roadway preview, dominates speed choice at these sight distances."

The AASHTO *Roadside Design Guide* [17] encourages the use of operating speeds during free-flow conditions for designing urban roadside features. The guideline indicates that more severe crashes can occur during high-speed conditions, and the nature of the urban environment deems it likely that during high traffic volume conditions the operating speed will be lower due to the interaction of vehicles. The guideline also encourages designers to perform individual site studies before establishing restrictions regarding roadside environment design since the clear roadside concept is rarely attainable in a dense urban setting.

2.2.1.3 Traffic Volume

The influence of increasing traffic volume levels on operating speed is intuitive. Simply put, the more vehicles there are in a traffic stream, the less likely it is that a driver can freely select his or her desired speed. Similarly the interaction of vehicles (e.g., slow vehicle turning into a driveway) directly influences the speed of vehicles in the vicinity. As a result, the free-flow speed is commonly assumed to best represent the driver's preferred operating speed, as seen in the HCM. Free-flow speed on an urban street is the speed that a vehicle travels under low-volume conditions [(4)]. The HCM further suggests the free-flow speed should be measured mid-block and as far as possible from the nearest signalized or stop-controlled intersection [(4)].

Studies where the researchers observed prevailing speed, rather than just free-flow speed, support the influence of traffic volume on overall speed. Polus, et al. [(18)] evaluated the effect of traffic and geometric measures on highway vehicle speeds. The study determined that the average curvature, average hilliness, and traffic volume each had a moderate negative correlation with the average running speed. Drivers' selected speeds were higher during low traffic volume conditions. During heavy traffic flow, speeds were lower due to the influence of other vehicles in the traffic stream. This influence of prevailing traffic conditions was also observed by Ericsson [(6)].

2.2.1.4 Influence of Traffic Control Devices

> *"The purpose of traffic control devices, as well as the principles for their use, is to promote highway safety and efficiency by providing for the orderly movement of all road users on streets and highways throughout the nation."* [(19)]

Traffic control devices are implemented to regulate, direct, or advise drivers. The Manual of Uniform Traffic Control Devices (MUTCD) [(19)] emphasized that vehicle speed should be carefully considered when implementing various traffic control strategies. The regulatory posted speed limit is the traffic control device most frequently used as an indicator of operating speed. However, several studies determined that posted speed limit is not an effective traffic control device for regulation of vehicle speed. Mustyn and Sheppard [(1)] indicate more than 75 percent of drivers claim they drive at a speed that traffic and road conditions permit, regardless of the posted speed limit. Although the drivers interviewed for the study tended to consider speeding to be one of the primary causes of crashes, they did not consider driving 16 km/h (10 mi/h) over the limit to be dangerous. Most of those interviewed did consider driving 32 km/h (20 mi/h) over the limit to be a serious offense.

Garber and Gadiraju [(20)] studied speed variance for 36 roadway locations, including intersections, arterials, and rural collectors. While all 36 roadways had the same posted speed (55 mph) they represented a cross section of design speeds (design speeds were obtained from department of transportation plan sets). Their results suggested that drivers increased speed as geometric characteristics improved regardless of posted speed limit. A similar study by Leish and Leish [(21)] pointed to the fact that drivers selected their speeds according to the highway ahead and may exceed both the speed limit and the design speed.

Parker [15] evaluated the influence of raising and lowering posted speed limits on driver behavior for urban and rural unlimited access roadways for 98 sites in 22 states. He found that the changing speed limits had no significant influence on driver speeds. He concluded that drivers determine speed according to their perception of the road. This perception is not changed due to the posted speed limit.

Other studies, however, have inconclusive observations about the level of influence of posted speed limits on driver behavior. Fitzpatrick et al. [22] investigated geometric, roadside, and traffic control device variables and their influence on driver behavior for major suburban four-lane arterials. They observed that the only significant variable for influencing speed on tangent sections of road was the posted speed limit. In addition to posted speed, deflection angle and access density influenced speed on curve sections. Zwahlen [23] found that advisory speed signs on curves are not generally heeded by drivers and may even produce the opposite effect for which they are intended.

Other traffic control devices have little impact on driver selected speeds. Várhelyi [24] studied drivers' speed behavior at zebra pedestrian crossings. He suggested that the willingness of drivers to give priority to pedestrians at the zebra crossing was low, and that drivers did not observe the law concerning speed behavior at the zebra crossings.

2.2.1.5 Traffic Calming Techniques

"There's more to life than increasing its speed."
Mahatma Gandhi

The above quotation embraces the concept of traffic calming. Traffic calming is the implementation of unique traffic control strategies that reduce traffic and lower vehicle speeds in residential and local service regions. Traffic calming strategies may range from physical modifications (chokers, speed humps, etc.) to increased enforcement, modified road use (on-street parking, bicycle lanes, etc.), and time-based exclusions. Several researchers have evaluated feasible traffic calming strategies and their impact on operating speed.

Ewing [25] explains that speed impacts of traffic calming measures depend primarily on geometrics and device spacing. His report, *Traffic Calming State of the Practice*, identifies numerous speed studies where before/after evaluation of calming devices resulted in speed reductions. Representative examples of traffic calming strategies that resulted in reduced speeds summarized in his report include speed humps, raised intersections, traffic circles, narrowings, and diagonal diverters.

Amour [26] determined that the presence of an enforcement symbol (e.g., a police car) might reduce the vehicle speeds on an urban road. He also demonstrated it was possible to produce a memory effect of police presence in an urban situation, but showed that drivers returned to their normal driving behavior very soon after passing a police vehicle.

Roadway restrictions are effective traffic calming strategies. Many residential streets are considerably wider than necessary for prevailing traffic conditions. Officials in Anne Arundel County, Maryland, painted parking lane lines without centerline striping on residential streets. This visually narrowed the street and reduced vehicle speed by 4.8 to 6.4 km/h (3 to 4 mi/h) [(27)]. It is important to note, however, that opponents of this strategy suggest the visually narrowed street directs vehicles into the path of approaching traffic and introduces safety hazards.

Comte and Jamson [(28)] used a driving simulator to investigate the effectiveness of speed-reducing measures ranging from intrusive devices (speed limiter or in-car advice) to informational devices such as variable message signs or transverse bars. All speed-reducing measures evaluated proved to be effective, with speed limiters proving to be the most influential.

Barbosa, et al. [(29)] investigated the influence of varying combinations of traffic calming measures on vehicle speeds by evaluating differences in speed profiles. Five roads in the City of York located in the United Kingdom were selected for this case study. The study focused on traffic calming measures, including speed humps, speed cushions, and chicanes implemented in sequence. The researchers concluded that calming measures of the same design tended to produce similar influences on speeds and the effectiveness of the measures in reducing speed decreased under higher entry speed conditions.

Stop signs are the most publicly requested regulatory measures to slow traffic on streets. Many studies indicate, however, this strategy has a weak or negligible effect on overall traffic speeds. (Basically, drivers who do slow their speed at the intersection generally pick up speed quickly in mid-block locations to compensate for the "lost time.") Before-after speed studies conducted in the City of Troy, Michigan, indicated that stop signs were not effective in controlling speeds and compliance with these stop signs was extremely poor [(30)].

2.2.2 Physical Environment Characteristics

Lighting conditions (e.g., daylight, dawn, dark) and environmental influences like heavy rain or snow may influence driver's speeds. Very few studies address specifically light or weather constraints, and most of the past studies focused on rural road locations.

The *Roadside Design Guide* [(17)] indicates that operating speeds on urban and suburban roads have greater variation by time of day than rural roads. During the lower volume and higher speed period of 7 p.m. to 7 a.m. (generally corresponding to nighttime conditions) there is a greater percentage of crashes due to the higher speeds and greater speed variances.

Liang et al. [(31)] evaluated the effect of visibility and other environmental factors on driver speed. They determined that drivers reduced their speeds during poor environmental conditions such as heavy rainfall or high winds. This reduction was accompanied by a higher variation in speeds.

Lamm et al. [32] compared vehicle speeds during dry and wet conditions on two-lane rural highways in New York. This research team concluded that operating speeds on dry pavements were not statistically different from operating speeds on wet pavements.

2.2.3 Vehicle Characteristics

Very little research exists on the speed characteristics of individual vehicle types in a general traffic stream. A common segregation of vehicles is into the categories of passenger cars, heavy vehicles, buses, and recreational vehicles. For emission analysis, vehicle fleet characteristics are further defined based on number of axles and age of the vehicle. For speed analysis, due to the random nature of the data collection, the most common means of evaluating vehicle characteristics is to simply separate heavy vehicles from all other vehicles and study their behavior independently. The existing speed model section of this chapter summarizes several methods for estimating operating speeds. Table A-1 in the appendix depicts these specific models for a rural environment. It is interesting to note that the predominant approach to speed modeling is to limit the study to passenger cars only. In the rural environment, only one researcher summarized elected to model truck behavior and that was at the exclusion of the passenger cars. Table A-2 depicts similar urban speed models. In this environment a variety of vehicle fleet characteristics were included in the models. The isolation of specific speed influences beyond the broad categories of truck versus car does not appear in the available literature.

2.2.4 Driver Characteristics

Many previous studies concentrated on the relationship between drivers' speed selection and road/vehicle characteristics without considering other important factors such as personal characteristics and drivers' perception of the roadway environment.

2.2.4.1 Judgment

A speed management Transportation Research Board report [34] stated:

> "In many speed zones, it is common practice to establish the speed limit near the 85th percentile speed, that is, the speed at or below which 85 percent of drivers travel in free-flow conditions at representative locations on the highway or roadway section. This approach assumes that most drivers are capable of judging the speed at which they can safely travel."

This speed approach is not recommended for urban roads, however, because of the mix of road users, high traffic volume, and level of roadside activity. Perception of safe speed is influenced by judgment of vehicle capability, anticipation of roadway conditions (further influenced by familiarity with the route), fatigue or similar factors, and judgment of speed on crash probability and severity. Most drivers do not perceive the act of driving as life-threatening. They believe themselves to be good drivers, and they often misjudge vehicle speed. People use the following information in determining driving speed:

> "characteristics of the road; the amount of traffic on the road; weather conditions and time of day; the speed limit and its enforcement; the length and purpose of the trip; the vehicle's operating characteristics, such as handling and stopping as well as fuel consumption and emissions; and driver-related factors, such as the propensity to take risks and the pleasure associated with driving fast." (TRB Report) (34)

2.2.4.2 Personal Characteristics

Kang (35) analyzed Korean drivers' speed selection behavior by taking into account such factors as personal, vehicle, attitudinal and trip characteristics. He concluded that male drivers with higher income tended to drive faster, experienced drivers drove at a higher speed than others, and trip distance and frequent use of the road were also important factors for speed selection behavior.

Poe, et al. (36) studied the relationship of operating speed to roadway design speeds for low-speed urban streets. In this study, both driver and vehicle characteristics were evaluated. They observed that gender, number of passengers, and passenger vehicle types were not significant. The analysis indicated that senior drivers traveled about 2 km/h (1.2 mi/h) slower than young drivers.

2.2.4.3 Attitudes

Based on data from Swedish drivers on roads with speed limits of 90 km/h (55 mi/h), researchers investigated drivers' attitudes towards speeding and influences from other road users on the drivers' speed choice. Haglund and Åberg (39) suggested that drivers might influence the driving patterns of others and that they might adjust their own speed in accordance with their estimate of the behavior of other drivers.

2.2.4.4 Experience

Elslande and Faucher-Alberton (40) found that in most situations, experienced individuals can use knowledge of a task to enhance performance. However, it is possible for experienced individuals to become overconfident, and particularly in a driving task, to encounter more risky situations because of it. Drivers use consistent behavior in an environment, even if their vision is impaired by some object. The automaticity of driving prevents them from executing a complete visual search of the environment. Also, drivers sometimes fail to update information. They ignore cues that present information indicating a change to their expectancies. These problems can be characterized as perceptive negligence, interpretational errors, or temporary breakdown of observation.

2.2.4.5 Response

Perceptual countermeasures can be used to influence driver perception of safe speed. These include patterned road surfaces, center and edge-line treatment, lane-width reduction, curvature enhancements, and delineators (guideposts) (34).

Scallen and Carmody (33) investigated the effects of roadway design on human behavior in Tofte, Minnesota. They found that white pavement treatments produced more moderate speeds and large speed changes, and landscape architecture treatments on the medians and roadside also produced desirable effects in driver's selection of speeds.

Poe, et al. (36) also investigated how the perspective view of horizontal curves might influence the relationship between perceived speed, operating speed, and geometric design speed. Their results indicated that the visual perspective view of a horizontal curve might be an important factor in the selection of an appropriate speed on horizontal curves. This suggests that a three-dimensional approach to horizontal curve design for low-speed alignments would assist in design consistency.

Hassan and Easa (37) suggested that combined horizontal and vertical alignment could cause a distorted perception of the horizontal curvature and could affect the drivers' choice of operating speed on horizontal curves. They determined that horizontal curvature looked consistently sharper when overlapped with a crest vertical curve and consistently flatter when overlapped with a sag vertical curve. Gibreel and Easa (38) also found that the overlapping vertical alignment could influence the driver's choice of speed on horizontal curves. They found that drivers adopt higher operating speeds on horizontal curves combined with sag vertical curves compared to the speeds on horizontal curves combined with crest vertical curves.

Alison Smiley (41) found that a driver's main cue for speed comes from peripheral vision. When peripheral vision is eliminated, drivers use only the central field of view to determine speed. If peripheral stimuli are close by, then drivers feel that they are going faster than if they encounter a wide-open situation. Dr. Smiley pointed out that speed was most influenced by geometric demands (i.e., sight distance, sharpness of curves, grades, etc.).

Bartmann et al. (42) also examined the effects of driving speed and route characteristics on the visual field. As speed increases, the visual field, from which the driver gathers information, decreases. Thus, peripheral vision gets greatly reduced at higher speeds, taking away a number of relevant driving cues. Six subjects wore eye movement helmets and were asked to drive on three different road types at varying speeds. On the urban street they were asked to drive at 50 km/h (31 mi/h) and 30 km/h (18 mi/h). Relevant eye fixations fell in the following categories: mirror, traffic control devices, traffic, and road related. The researchers concluded that urban street driving at higher speed corresponds to greater relevant object fixation. Driving speed influences perceptual behavior depending on road type.

2.2.4.5.1 National Highway Traffic Safety Administration Survey

In 1997, the National Highway Traffic Safety Administration (NHTSA) [43] commissioned a national survey of the driving public. The survey was conducted by telephone by the national survey research organization, Schulman, Ronca and Bucuvalas, Inc. A total of 6,000 interviews were completed with a participation rate of 73.5 percent. Six basic speed-related questions were presented to the subjects:

(1) Drivers were asked how important a series of factors were in selecting the speed at which they drive.

- The most important factor was the weather condition. Five out of six drivers felt weather was extremely important and another 10 percent felt it was moderately important.

- The second most important factor in the minds of drivers is the posted speed limit. This factor was rated as extremely important by 54-percent of the respondents and as moderately important by an additional 35-percent.

- The third most important factor was past experience on the road. This factor was rated as extremely or moderately important by 84 percent of the people surveyed.

- Traffic density, likelihood of being stopped by police, and the speed of other traffic were also identified as important speed influences by 75 percent of the interviewed drivers.

(2) Drivers felt the maximum safe speed for residential streets, whether in urban or rural settings, was 40 to 56 km/h (25 to 35 mi/h). The maximum safe speed for non-interstate urban roads was 72 to 88 km/h (45 to 55 mi/h).

(3) Drivers were asked why they consider speeds greater than the maximum speed to be unsafe on residential streets.

- Almost four in five residential road drivers mentioned the presence of people (non-drivers) -- primarily children, schools and playgrounds -- in close proximity to the roads as the primary reason that driving faster would be unsafe.

- The second most often reason cited concerned individual reaction times and the ability of the vehicle to stop quickly.

- The next greatest concern, cited by about one in six drivers, centered around traffic patterns, primarily heavy traffic and merging.

- Other categories suggested safety, road conditions, weather conditions, and presence of other vehicles.

(4) Drivers who reported that they drove faster now than they had one year ago were asked why they were driving faster. More than half the drivers said they were driving faster as a result of increased speed limits. The second most common reason suggested for driving faster was the increased experience of the driver. Improved traffic flow conditions were also suggested.

(5) Drivers who reported they were driving slower also were asked to elaborate on the reasons. Two drivers in five identified driver-related issues, primarily the maturity of the driver. Safety concerns were the reason for driving slower for one driver in three. About half of these concerns were related to more cautious driving behavior. One driver in 14 was driving slower to avoid crashes and 6-percent were driving slower because they had been in a crash. Many of the slower drivers reported driving at reduced speeds due to vehicle-related factors, primarily having children or other family members in the car. Finally, respondents identified heightened police enforcement as a reason for driving more slowly.

Those drivers who reported that other drivers were more aggressive in their area than during the previous year were asked why they thought the other drivers had changed. Nearly 23-percent said that drivers drive more aggressively now because they are hurried, rushed or behind schedule. About an equal number of respondents (22-percent) attribute the increased aggressiveness of driving in their areas to traffic flow, particularly increased traffic volume and congestion. Two groups of drivers were singled out as contributing to increased aggressive driving -- young drivers and careless or inconsiderate drivers. Several respondents blamed higher speed limits as a contributing factor for increases in aggressive driving in their areas. The presence of fewer visible police was also suggested as a factor in increased aggressive driving.

2.3 Review of Existing Operating Speed Models

Existing operating speed models primarily focus on rural environments where drivers encounter uninterrupted traffic flow conditions and minimal variability. Limited research to date exists for urban environment speed estimation. Operating speed in urban areas may be influenced by a vast array of land use development issues, numerous road geometric features, and varying driver or vehicle characteristics not consistent with the rural environment. As a result, rural speed models and their "critical influences" on operating speed are initially reviewed in this summary to help identify factors transferable from rural speed models to a future urban speed model.

2.3.1 General statistic – 85^{th} percentile speed

The 85^{th} percentile speed is the general statistic used to describe operating speeds when assessing the influence of the driver's environment on speed selection. The 85th percentile speed is the speed at or below which 85-percent of the vehicles in the traffic stream travel. This speed measure is the most common factor used to set speed limits on existing roads in the United States and is internationally accepted as a reasonable representation of operating speed; however, conditions under which the 85th percentile speed are measured strongly influence perceived significant variables. For example, if a researcher elects to assess the influence of roadside trees on operating speed and only collects speed data during peak hour conditions, it is likely the prevailing traffic will exert a strong influence on the observed 85th percentile speed and minimize the influence of extraneous roadside features. It is reasonable to then consider the 85th percentile speed for only free-flowing vehicles. Again the peak hour influence may confound the tree influence. Drivers may be in a hurry to return home or retrieve their children from school. As a result, the time of day may influence the driver's behavior. It is necessary, therefore, to identify a comprehensive model that captures variables beyond physical road features and to study operating speeds for a variety of road, driver, and environment configurations.

2.3.2 Operating Speed Models for Rural Highways

As previously indicated, the existing speed models are divided into rural and urban conditions. Within the rural environment, researchers typically separately evaluate speed for roads with horizontal geometric controls (e.g., curves versus tangents) from roads with vertical controls; however, several models also exist that evaluate a corridor that includes the combined influences of horizontal and vertical influences collectively. Table A-1 summarizes several of these representative rural operating speed models.

2.3.2.1 Models for Rural Horizontal Geometric Controls

Estimation of speeds on curves may be easier than the prediction of speeds on tangent sections due to of the correlation of speeds to a few defined and limited variables, such as radius and superelevation rate. On tangent sections, however, the vehicle speed is dependent on a wide variety of roadway characteristics including the tangent length, cross-sectional elements, vertical alignment, general terrain, sight distance, and driver’s attitude. Many available models, therefore, focus on the prediction of speeds at horizontal curve locations.

Many researchers have developed similar models for the estimation of the 85th percentile speed for rural roads at horizontal curves. For a variety of speed limits, vertical grades, and vehicle types (primarily passenger cars or heavy vehicles), several studies identified the primary independent variable influencing operating speed to be only the radius of the curve (or a surrogate measure such as degree of curve or inverse of the radius) [10, 44, 45, 46, 47, 48, 49].

McLean [50] observed that the 85th percentile curve speeds were dominantly influenced by both the driver's desired speed and the curve radius. Lamm et al., [51] expanding on work performed in 1988, [44] suggested the lane width, shoulder width, and traffic volume explain approximately 5.5-percent of the variation in operating speeds over a simple speed model that only considers curve radius.

The speed model developed by Ottesen and Krammes [9] added the horizontal curve length and the approach speed tangent to the model (in addition to the radius). This model approach is only useful if approach tangent speeds are actually measured.

Andueza [52] developed a rural speed model that included radii for consecutive curves, tangent length before the curve, and a minimum sight distance for the horizontal curve. Donnell et al. [53] developed rural heavy vehicle curve speed models that included both the length and grade of approaching and departing tangents, the radius, and curve length.

Many researchers determined that a vehicle's speed changes as it traverses a sharp horizontal curve and the vehicle does not maintain a constant speed. Similarly, the influence of boundary horizontal curves extends to short tangent sections between the curves. Liapis et al. [54] analyzed the speed behavior of passenger cars at 20 on- and off-ramps in rural Greece, and concluded the 85th percentile speed is dependent on the superelevation rate (directly correlated with curve radius) and the curvature change rate. They identified this curvature rate of change by adding the angular change in the horizontal alignment and then dividing by the length of the highway section studied.

Polus et al. [55] developed four speed models for tangents located between horizontal curves. They categorized the horizontal geometry as one of four conditions:

- Group 1 -- sharp curve radii and short connecting tangent,
- Group 2 -- sharp curve radii and moderate length tangent,
- Group 3 -- moderate curve radii and moderate length tangent, and
- Group 4 -- flat curve radii with long tangent.

The research team determined for group 1 operating speed, only the radii of the curves proved significant; however, for group 2 the length of tangent was also significant. Due to limited available data sets, their speed models for groups 3 and 4 were inconclusive. Preliminary models appeared to depend on factors similar to those for group 2, but the researchers cautioned that characteristics such as cross-section, vertical longitudinal slope, and vertical curve rate of change (if vertical curvature is present) also may influence operating speeds.

2.3.2.2 Models for Rural Vertical Geometric Controls

Roadway parabolic vertical curves can be either crest curves or sag curves. Whereas, sag curves generally do not physically constrict a driver's line of sight, an abrupt crest vertical curve may impede the driver's sight distance. Jesson et al. [56]

evaluated operating speeds for crest vertical curves at rural two-lane highways in Nebraska. They separated their study corridors into two categories: crest vertical curves with limited sight distance, and crest vertical curves without sight distance constraints. This research team tested the significance of numerous variables, including:

- the approach grade,
- the algebraic difference for the vertical curve,
- the length of the vertical curve,
- the vertical curve rate of change,
- the inferred design speed (per accepted design standards),
- the average daily traffic (ADT),
- heavy vehicle percentage,
- posted speed limit,
- width of roadway and shoulder, and
- type of shoulder.

Only the posted speed limit, approach grade, and ADT proved significant for crest vertical curves with limited sight distance. For crest vertical curves with adequate visibility, they developed a similar speed model but with the only significant independent variable as the ADT and posted speed limit.

Fitzpatrick et al. [(49)] similarly evaluated crest vertical curves at horizontal tangent locations. They determined that the operating speed is essentially the driver's assumed desired speed for unlimited sight distance locations, whereas the vertical curve rate of change proved to be the only significant variable for the 85th percentile speed at limited sight distance crest curve locations. This research team further evaluated the speed for sag vertical curves at horizontal tangent locations and again concluded the speed represented a driver's selected speed at these locations.

2.3.2.3 Models for Locations with Combined Horizontal & Vertical Controls

In a study by Gibreel at al., [(38)] the authors developed speed models for combined horizontal and vertical conditions. They evaluated (a) a horizontal curve combined with a sag vertical curve, and (b) a horizontal curve combined with a crest vertical curve. In an effort to identify discrete influence locations, the research team collected speed data at five locations in the vicinity of the curve. They determined that for the three-dimensional road conditions evaluated, significant variables influencing the 85th percentile speed include:

- radius of horizontal curve,
- deflection angle of horizontal curve,
- horizontal distance between the point of horizontal intersection and the point of vertical intersection,
- length of vertical curve (or rate of curvature),
- vertical gradients,
- algebraic difference in grades, and

- superelevation rate.

Fitzpatrick et al. [49] evaluated speeds for the combined effect of a horizontal curve with a sag vertical curve. They similarly evaluated operating speeds for a horizontal curve combined with a limited sight distance crest vertical curve. For both conditions, only the inverse of the horizontal curve radius proved to be a significant factor for operating speeds.

2.3.3 Operating Speed Models for Urban Roadways

The urban street environment is characterized by a variety of influences that may conceivably influence the operating speed for a facility. As a result, horizontal curvature alone cannot define the anticipated speed for an urban street as it did for many of the speed models for the rural environment. Numerous roadside features and access points create a complex driving environment. Poe et al. [57] determined that access and land use characteristics have a direct influence on operating speed. For example, higher access density contributes to lower operating speeds due to the increased interaction with vehicles from driveways, intersections, median areas, and parking.

Fitzpatrick et al. [58] evaluated operating speeds for curve sections on suburban roadways. The roads in this study were four-lane divided sections with moderate approach densities and signal spacing. The research team used approach density as a surrogate for roadside development. Only data for free-flow passenger cars, pickup trucks, and vans were included in this study. One variable used in the evaluation was an inferred design speed that generally represented road design constraints (e.g., available sight distance for crest vertical curvature conditions). For horizontal curve locations, the speed models resulted in a curvilinear regression equation with two significant independent variables -- horizontal curve radius and approach density. For crest vertical curve locations, the inferred design speed proved to be the only significant variable for predicting operating speed. It is important to note, all crest curve locations included in the study were characterized by limited sight distance, so the resulting speed model may not be applicable to unrestricted sight distance vertical conditions.

Bonneson [59] studied vehicle speeds on horizontal curves at 55 sites in eight states. These sites included urban low-speed, urban high-speed roadways, rural low-speed and high-speed roadways, and turning roadways. He developed a curve speed model to identify the relationship between curve speed, approach speed, radius, and superelevation. He also developed a side friction model to explain the relationship between the approach speed, speed reduction, and side friction demand at horizontal curves. Minimum radii and design superelevation rates were key variables in the development of the side friction model. The curve speed model included curve speed, approach speed, radius, and superelevation rate. It is important to note a collinearity exists between the radius and the superelevation rate, so application of model using both variables may lend a bias toward the curve geometry.

A study by Poe et al. (36) identified the geometric roadway elements, land-use characteristics, and traffic engineering elements that influenced vehicle speeds on low-speed urban street. Poe, Tarris, and Mason performed an analysis to determine the relationship between 85^{th} percentile speeds and geometric, roadside, driver, and traffic control variables. They considered the following variables during model development:

- Geometric measures (e.g., curve radius, grade, sight distance),
- Cross-section (e.g., lane width, road configuration),
- Roadside (e.g., access density, land use, roadside lateral obstructions),
- Traffic control devices (e.g., speed limit, pavement marking), and
- Driver / vehicle (e.g., gender, age, number of passengers, vehicle type).

The best speed estimation models resulted in the following general form:

$$\text{Speed} = \beta_0 + \beta_1(\text{Alignment}) + \beta_2(\text{Cross Section}) + \beta_3(\text{Roadside}) + \beta_4(\text{Traffic Control})$$

The researchers collected free-flowing speed data at designated locations along a corridor. In addition, they determined basic road geometry. Field observation teams, positioned next to the road, attempted to document information about each vehicle and driver. This study is the only United States field study identified where researchers attempted to include driver and vehicle influences (other than presence of heavy trucks) into a speed model.

2.3.4 Analysis of Existing Models

The existing speed models range from a simple linear regression model with a dependent variable of speed and a significant independent variable of the horizontal curve radius up to complex curvilinear regression equations. The majority of the existing speed models attempt to quantify operating speed based primarily on physical conditions such as road geometric design and, in the urban environment, roadside development. Many of the significant variables influencing speed selection may not be included in previous models simply due to the complexity of data collection issues. For example, an experienced driver may traverse a sharp horizontal curve at a much faster speed than that of a novice driver. By using the 85th percentile speed as a representative measure for operating speed, analysts are simply attempting to identify the operating speed threshold under which 85-percent of the drivers in the traffic stream elect to travel. Generally, these models represent roads with dry pavement and daylight conditions. Again, these are typical data collection controls established to maintain consistency between limited data sites.

Poe and Mason (60) suggest that, at a minimum, a mixed-model analysis should be performed for speed estimation. Mixed models account for the influences of both random and fixed effects. A fixed effect may be represented by geometric elements that do not fluctuate from day-to-day. Similarly, a random effect represents a random sample of a larger population. For example, a study of 50 vehicles is generally based on the assumption that the 50 randomly selected vehicles are representative of the larger traffic stream. A mixed model can be used if the following assumptions are met:

- Data are normally distributed,
- Means (expected values) of the data are linear in terms of a certain set of parameters, and
- The variances and covariance of the data are in terms of a different set of parameters and are in a format that can be modeled.

A basic assumption echoed throughout the available speed model research is that operating speed can be modeled using some sort of regression analysis. When unpredictable elements are included in the data set such as weather, driver type, time of day, visibility, or a combination of these non-geometric elements it is likely that simple statistical procedures may not adequately represent the operating speed selection. As a result, more appropriate models may include speed profiles for specific conditions, a set of speed curves, or perhaps decision trees with a variety of confounding variables (combined influences). The simple linear regression model for the rural horizontal curve conditions does not appear to be a realistic model candidate for the complex urban environment.

2.4 Methods to Evaluate Driver's Perception of the Road Environment

Human factors research has implemented many techniques for evaluating a driver's perception of the road environment. In general, analysis methods focus on three basic evaluation techniques: driver simulator studies, static two-dimensional or dynamic three-dimensional "office" studies, and human field studies. Often a combination of these three techniques may be used to validate results obtained in the easily controlled simulator environment to those applicable to the uncontrolled real world environment.

2.4.1 Simulator Studies

The use of simulators for testing, training, and evaluation of driver reactions to their environments dates back many years and is the most common method implemented to evaluate drivers’ perception of the road environment. In a simulated environment, a researcher can hold many variables constant while altering one item to evaluate the driver's reaction to a single variable. For example, a road environment cluttered with many obstacles such as traffic signs, pavement markings, driveways, roadside attractions, and physical road geometric features makes the assessment of a single variable (say driveways) difficult due to the confounding influence of all the extraneous environment "noise." A simulator can be used to hold all variables constant and simply alter the driveway density to determine the reaction a driver may have to this single feature. Assessment of the "drivers" in the simulator environment can be performed in a variety of ways. The subjects can simply complete surveys about how they respond to an environment, or the subjects can interact with the simulator environment and respond to individual stimuli.

Klee et al. [61] provided a preliminary validation of a driving simulator at the University of Central Florida (UCF). Thirty volunteers were asked to drive an instrumental car along a section of road on the UCF campus. The vehicle was equipped with a distance measurement instrument that recorded instantaneous speed, cumulative distance, and elapsed time at designated points along the route. The drivers were then asked to drive in the UCF driving simulator. This simulator consists of a complete vehicle cab with a wraparound screen for displaying computer-generated images of the identical campus road and surroundings. Speed data from the field and simulator were analyzed using conventional statistical tests to determine whether drivers responded differently in the simulator compared with their response during the real driving experience. Results of the statistical analysis indicated that drivers behaved similarly at 10 of 16 designed locations along the road. Confidence intervals for the difference between the simulator and the field mean speeds indicated a tendency of drivers to travel at slower speeds in the simulator.

A pilot study performed by Lockwood [62] evaluated traffic calming features using the Transport Research Laboratory (TRL) driving simulator. The TRL simulator produced small variations in the pitch of the car, the noise of the car, and the noise of passing traffic. The simulator continuously recorded the driver-selected speeds throughout each experiment. Lockwood examined the validity of the simulator by comparing the results with those of public-road trials through three local villages. He demonstrated that the effects of signing/marking measures as perceived by drivers could be broadly reproduced in the TRL simulator. This indicated that the simulator was valuable in supplementing the results of road trials, in particular, for comparing the effects of a wider range of measures. The simulator also offered additional data not easily obtainable from conventional road trials.

In a study by Scallen and Carmody [33], the researchers developed a wraparound driving simulator to test the driver response to roadside patterns and environments. They developed a computer model for an actual segment of urban highway planned for reconstruction in Tofte, Minnesota. This experiment marked the first use of a driving simulator as part of the Minnesota Department of Transportation highway design process. The designers could visualize the project and test drive various options prior to completing plans and construction documents. The purpose of this simulator experiment, therefore, was to assist evaluation of visual cues in the environment rather than directly evaluating driver reaction to those cues.

2.4.2 Static Two-Dimensional and Dynamic Three-Dimensional Methods

Although driving simulators provide a "near world" experiment, they are also expensive to construct and maintain. Other laboratory models provide accurate analysis for a variety of driver perception conditions. Common alternative evaluation methods include static two-dimensional (2-D) and dynamic three-dimensional (3-D) methods of presentation.

Zakowska [63] investigated forty drivers' perception of road curves. More specifically, he intended to test two experimental research methods used for road view evaluation and to evaluate the effect of visual information from both a static and dynamic road view on driver perception of road curves. Each "driver" was shown two series of corresponding road pictures, one dynamic and one static. They were asked to give ratings of the presented road situation (the approaching zone of the horizontal curve and the curve itself). Their ratings reflected their subjective perception of level of curvature and curve angle for each curve. This research determined that drivers were able to discriminate different levels of curvature and angle of curves. The perception of curvature is more sensitive to geometric curve properties for a dynamic presentation than for a static presentation.
In a study by Hassan and Easa, [37] computer animation was employed as a 3-D presentation method of the road perspective, and was found to produce a realistic view of the road.

2.4.3 Human Field Studies

Driver perception and reaction, though often tested in a simulated environment, should also be tested in a physical environment. Historically, to assure safety to experimental subjects, the use of a closed test track to evaluate driver perception of traffic control devices provides useful information. Since a driver is actually operating the vehicle and the vehicle is actually in motion, a more accurate indication of how a driver will respond to stimuli in the road environment can be evaluated in the test track environment. Unfortunately, past studies show that after a test driver traverses a limited length track several times, the driver adapts to this road environment and may not react as he or she would if encountering the stimuli in the open road environment. Nevertheless, test track data is useful for evaluation of conspicuity and sight distance variables that evaluate human performance characteristics.

A representative example of a test track study is the evaluation of driver braking performance for stopping sight distance performed by Fambro et al. [64]. In this experiment, nine employees of the Texas Transportation Institute, three of whom were expert drivers, participated in a closed-course braking study. Subjects were instructed to drive the equipped vehicle through a test course at a specific speed. The drivers were given a "count down" signal for braking and were also provided a random "surprise" braking signal. To reduce the driver expectation for the "surprise" signal, approximately 20-percent of the time the drivers were not given any signal. The researchers evaluated a variety of conditions including anti-lock brake performance, and driver's braking distance for wet versus dry conditions.

Open road field studies provide the most accurate information for driver behavior and perception of the road environment. In the past, however, most open road studies were performed with a few equipped vehicles that were driven by a small sample of test drivers. The equipped vehicles generally include speed and distance measurement evaluation devices and the driver typically is instructed to traverse a designated route.

As an example, researchers at the Georgia Institute of Technology recently performed an open road field study. They evaluated rush hour driver speed and travel characteristics on the Interstate-75 freeway corridor in the Atlanta metropolitan region [65]. Two "floating" cars equipped with distance measure devices and accelerometers were dispatched during peak hour travel conditions concurrent to traffic volume field data collection. The drivers of the cars were instructed to enter the freeway while maintaining a consistent distance behind the vehicle in front of them. Upon entering the freeway, the driver was instructed to change one lane to the left (of a six-lane road section where one lane is a designated high occupancy vehicle lane), and then seek the first white car in the traffic stream. The driver was then instructed to follow that car and duplicate the movement and speed behavior of the car. In this way, the influence of the test driver could be minimized in evaluating typical driver behavior. Since one of the lanes on the freeway was a high-occupancy lane, the test vehicle included two occupants. One occupant drove the vehicle while the second occupant operated an on-board computer. One limitation of this study is that the first white vehicle identified from the second lane from the right rarely was in the high-occupancy lane. As a result, driver performance was generally limited to conventional freeway lanes.

Though the open field study technique is the most accurate method for truly identifying driver perception and reaction to the road environment, it is characterized by experimental bias. For example, often only one or two equipped vehicles are available. These vehicles are assumed to be representative of the traffic stream. As a result, vehicles that do not have similar performance characteristics as the equipped vehicles are not adequately evaluated. In the United States, a recent trend in the automobile industry is a sharp increase in the number of sports utility vehicles in the traffic stream. These vehicles place the driver in a higher riding position and do not necessarily perform in a manner consistent with the standard passenger car. Another possible bias to the open road field test scenario is the limited number of test drivers. In general, research studies often focus on one type of driver such as the elderly driver or the college student. This limitation restricts evaluation of the driver type. In addition, test drivers are alert to possible stimuli because they are aware they are subjects of a test. As a result, reaction time may not be representative of the typical driver who is not expecting similar stimuli. Finally, field tests are often performed at a designated time of day, weather condition, and traffic condition. This selection of restricted variables is intentional experimental design to enable researchers to perform tests during reasonably uniform conditions. This restriction is necessary because field tests are expensive and the resulting data set has only a limited number of data points for which to evaluate the driver's behavior. The ideal open road test would minimize these experimental limitations to assure representative findings for the driving population.

2.5 Summary

This chapter presented in-depth review of the current literature on factors influencing speed choice, existing operating speed models, and methods to evaluate the driver's perception of the road environment. As seen many researchers have determined that current design speed approaches for low speed urban streets often result in operating speeds higher than their associated design speeds. This observation suggests that the conventional design speed approach may not be appropriate for urban street environments. The design speed approach incorporates a significant factor of safety to provide a road that functions well for all drivers, and performs well during inclement weather and varying lighting conditions. This resulting minimum design speed value may be lower than the speed a driver is likely to expect or select. Therefore, it is not surprising that many drivers feel comfortable traveling at speeds higher than the roadway's design speeds during favorable conditions.

Of significant note identified fundamental flaws in the design speed concept approach included that design speed applies primarily to horizontal and vertical curves rather than to the tangents between these curves and the current design speed approach does not set limitations on the maximum allowable operating speed. To overcome the shortfalls of the design speed approach, it may be beneficial to incorporate an operating speed feedback loop into the design speed concept. Under this approach, the geometric elements of roadways are selected based on their influences on the desired operating speeds. Such an approach requires operating speed models for different road environments. As seen numerous previous studies have developed operating speed models, however, most of them have concentrated on high speed, rural highways. As a result, highway designers and planners have very little information about the influence of the low speed street environment on operating speeds.

This review has re-enforced the introductory remarks that designers of urban roads use a design speed concept in which a minimum suitable speed is used for the design of streets and highways, paying little regard to the actual speed drivers will select when utilizing the facility. The design speed does not address maximum operating speed issues, but simply assures that minimum design criteria are achieved. In chapter 3 the data utilized in this study to model drivers *selected* speeds is presented. Followed by data processing (chapter 4), operating speed data analysis (chapter 5), operating speed model development (chapter 6), and summary of findings (chapter 7).

3 DATA COLLECTION

3.1 Introduction

This study analyzes selected drivers' vehicle trajectory data for a one year period, from January 2004 to December 2004. Trajectory data is collected using an in-vehicle global positioning system (GPS) developed, deployed, and maintained as part of the Commute Atlanta Project, funded by the Federal Highway Administration (FHWA). The instrumented vehicles collect second-by-second position, i.e., latitude and longitude, and vehicle speed data. Other known variables are driver characteristics (e.g., gender, age) and vehicle type.

Potential study corridors are limited to those self-selected corridors traversed by the Commute Atlanta study drivers. The trajectory data was pre-processed by the Commute Atlanta project team before being distributed to the operating speed project. Data points within a 50-feet buffer area encompassing a study corridor were provided for trajectory analysis. In additional to the Commute Atlanta data, the research team also collected roadside environment features for the corridors under study.

This chapter provides a further overview of the data utilized for this operating speed study. Included in this chapter is a review of GPS and the in-vehicle instrumentation, the integration of the GDOT geographical information system (GIS) roadway data, the corridor selection process, and supplemental field data collection. Chapter 4 then provides step-by-step detail for the trajectory data processing.

3.2 In-Vehicle Equipment Data Collection

3.2.1 Introduction to Global Positioning System

GPS is a satellite-based navigation system consisting of 24 satellites orbiting the earth at an altitude of approximately 11,000 miles. GPS was initially developed for military services by the United States Department of Defense (DOD). However, GPS is now widely used for civilian applications. For example, in transportation engineering, GPS is widely used in studies of travel time, route choice, car following, and drivers' speed behaviors.

GPS has three components: the space segment, the control segment, and the user segment. The space segment consists of the 24 satellites that emit high-frequency radio waves. The control segment consists of five ground stations located around the world that monitor the GPS satellites and upload information from the ground. The user segment is the GPS receivers, which detect, decode, and process GPS satellite signals.

GPS determines a location by calculating the distances between the receiver and 4 or more satellites. GPS measures distance by measuring the travel time of radio waves from the satellites to the receiver. Assuming the positions of the satellites are known, the location of the receiver can be calculated by determining the distance from each satellite to the receiver.

3.2.2 Data Collection Equipment

The in-vehicle data collection equipment consists of a computerized processing unit, power system, cellular transceiver, GPS, and other sensors. The data collection equipment turns on and off automatically with the vehicle ignition. Recorded data are automatically transferred to a data server at the Georgia Institute of Technology over a wireless connection at periodic intervals. Figure 1 graphically depicts the GPS data collection system.

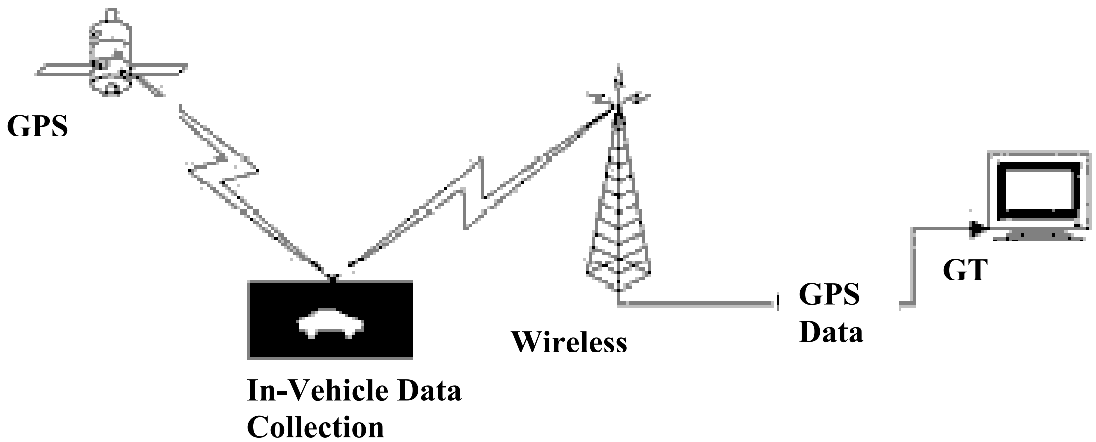

Figure 1. GPS Data Collection System

3.2.3 Speed Data from In-Vehicle Data Collection Equipment

As stated, the in-vehicle data collection is supported as part of the on-going FHWA Commute Atlanta instrumented vehicle project currently underway at the Georgia Institute of Technology. The portion of the Commute Atlanta database used for this operating speed project includes one-second interval (1 Hz) GPS data records for the entire year of 2004. The GPS receivers provide speed accuracy within 1.6 km (1 mph) for 95 percent of the time.

Table 2 presents an example of GPS speed data. The location and speed data are recorded at a rate of 1 Hz. For example, the last record in Table 2 indicates that this vehicle was traveling at 21.49 km/h (13.43 mph), at a latitude value of 33.80997, at a longitude value of -84.392974, at GMT time 14:53:24, and on April 30th in 2004.

Table 2. Example Speed Data from In-Vehicle GPS Data Collection Equipment

Date	Time	Latitude	Longitude	Speed (km/h)
20040430	145312	33.810060	-84.392663	0.00
20040430	145313	33.810061	-84.392668	0.02
20040430	145314	33.810061	-84.392675	0.06
20040430	145315	33.810063	-84.392680	0.02
20040430	145316	33.810063	-84.392685	0.43
20040430	145317	33.810060	-84.392686	1.62
20040430	145318	33.810066	-84.392710	7.01
20040430	145319	33.810072	-84.392748	12.43
20040430	145320	33.810070	-84.392796	15.71
20040430	145321	33.810063	-84.392845	16.35
20040430	145322	33.810046	-84.392893	17.20
20040430	145323	33.810013	-84.392938	19.52
20040430	145324	33.809970	-84.392974	21.49

The collected GPS data records were overlaid with a GIS digital road network map based on the latitude and longitude information so that the researchers know where, when, and how fast the drivers were driving. The task of associating the GPS data records to the GIS digital map was completed by the Commute Atlanta project team. Figure 2 shows a trip example overlaid onto a GIS road network.

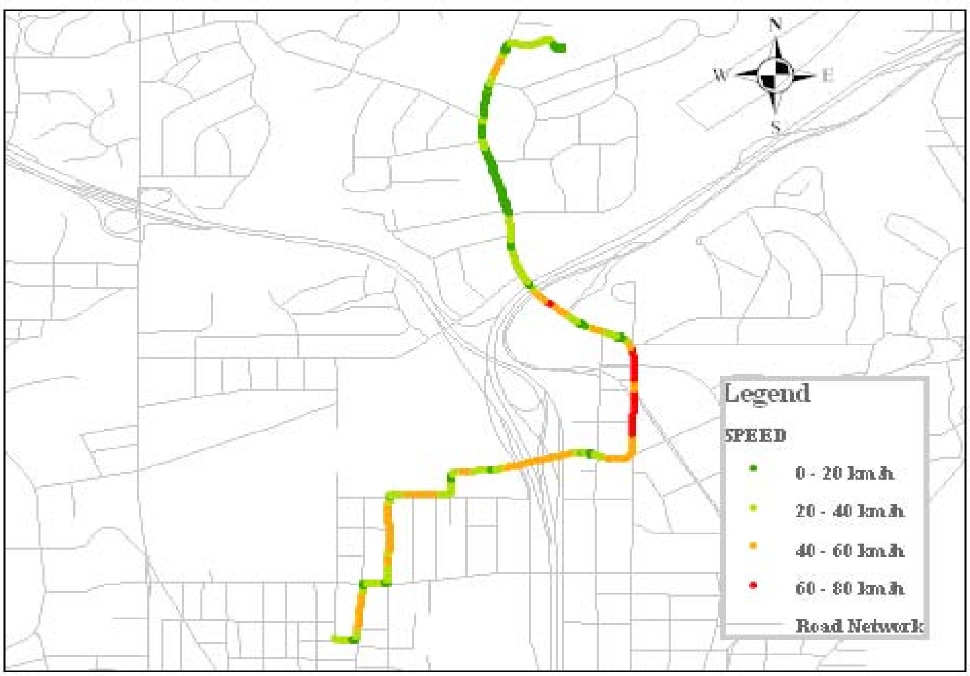

Figure 2. Example Trip Overlaid with GIS Road Network

3.3 GIS Road Network Database

The overlaid GPS data points have an associated road segment identification number (Link ID in Figure 3), which correspond to the route identification number in the GDOT Road Characteristics file (RC file). Utilizing the common Link ID, the research team was able to correlate the instrumented vehicle data overlaid on the GPS map to the GDOT RC file.

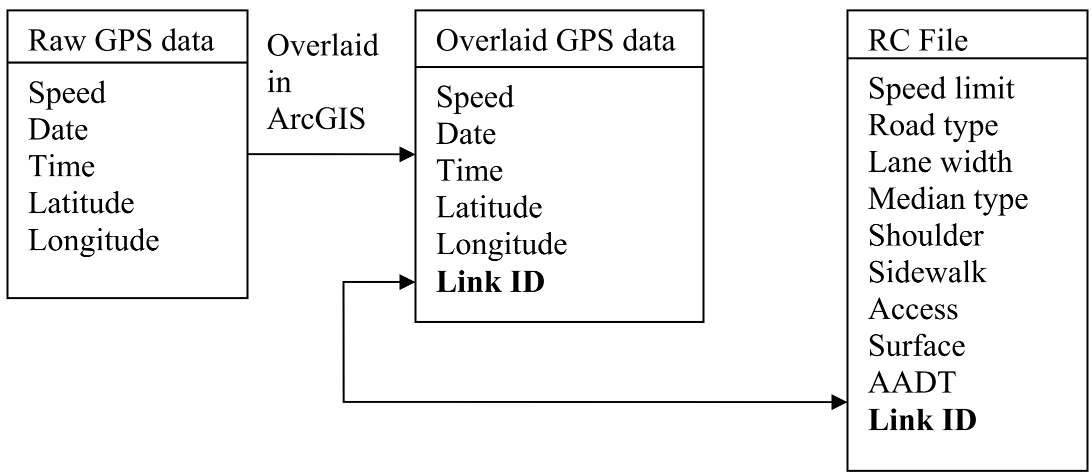

Figure 3. Relationship between GPS Data and Road Characteristics

The common Link ID is the GDOT RCLINK number that uniquely identifies each route of the road network. The RCLINK number is a 10-digit GDOT route identification number that provides a relational link between route features and their RC File. Each route consists of several road segments identified by a milepoint number. This milepoint represents the mile measurement along a route recorded to the nearest 1/100th of a mile. The road segments are delimited by intersections, ramps, and other physical discontinuities. An example road network is shown in Figure 4.

The research team extracted the road network characteristics database used for this project from the larger GDOT RC File. This final 13 county database includes road features for the public road network in the 13 county metro Atlanta area. Each road segment record includes 61 attributes that describe the road characteristics such as road type, number of lanes, lane width, median type, and speed limit. Each record is identified by a unique combination of RCLINK and MILEPONT number and corresponds to one unique link in the road base map. Chapter 4 will provide a detailed description of how the research team integrated the individual vehicle trajectory data points with the roadway attributes found in the RC file. For the final set of corridors utilized in this study (final corridor selection is given in Chapter 4) the GDOT RC data is field verified.

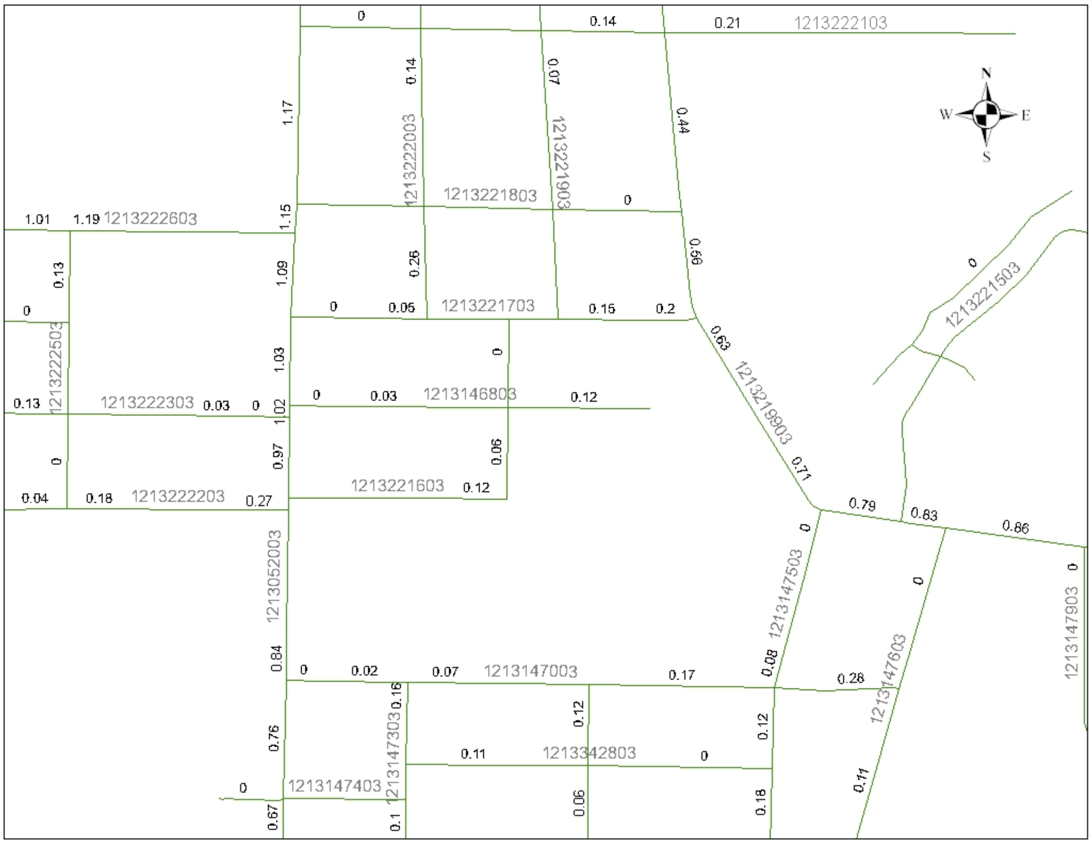

Figure 4. Example Digital Road Network

3.4 Characteristics of Study Drivers

The research team compared the study drivers' age and gender distribution with the U.S. census data of licensed drivers in 2003. The characteristics of selected drivers are reasonably representative of the general population in the United States. The authors also compared the vehicle type distributions. The sample set has a smaller percentage of minivans and pickups and a larger percentage of passenger cars and SUVs than the general population, as shown in Table 3.

Table 3. Study Driver and Vehicle Characteristics

	Sample Population (3)	U.S. Census Data
Gender		
Female	55.7%	50.1% (1)
Male	44.3%	49.9% (1)
Age Distribution		
Age less than 18	3.3%	4.7% (1)
Age between 18 and 45	41.9%	47.6% (1)
Age between 45 and 60	37.5%	27.1% (1)
Age larger than 60	17.2%	20.6% (1)
Vehicle Type		
Passenger Car	61.7%	56.8% (2)
Minivan	7.6%	9.1% (2)
SUV	19.8%	11.9% (2)
Pickup	11.0%	18.3% (2)

(1) Source: Age and Gender Distribution of U.S. Licensed Drivers, 2003, U.S. Department of Transportation, Federal Highway Administration, *Highway Statistics 2003*.
(2) Source: The 2001 National Household Travel Survey, vehicle file, U.S. Department of Transportation
(3) Note: Sample percentages are for instrumented vehicle data utilized in this operating speed study, all drivers in the Commute Atlanta Study may not be included in this data set and the Commute Atlanta Data distributions may differ slightly.

3.5 Corridor Selection

3.5.1 Determination of Appropriate Study Corridor Length

For the purposes of this study, a corridor is defined as the roadway section between two intersections. The corridor is characterized by uninterrupted flow for the study road, i.e., no stop-control traffic device such as a traffic signal or a stop sign is present on the corridor mainline. A side street intersecting the corridor may be sign controlled (stop or yield) or uncontrolled. If a study corridor is delimited by two intersections with traffic control devices, the corridor must be sufficiently long to enable drivers to reach their desired speeds. If a study corridor is delimited by two intersections without traffic control devices, there is no minimum length requirement, but the study corridor must be located at a sufficient distance from any adjacent traffic control devices. Figure 5 demonstrates a typical study corridor.

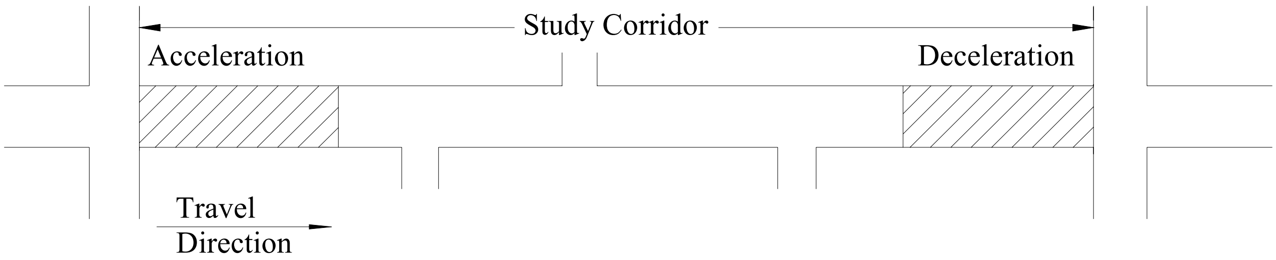

Figure 5. Example Study Corridor Layout

Several previous studies have indicated that the selected study corridors should be long enough or sufficiently distant from the adjacent traffic control devices such that a portion of the driver's trip on the roadway is not influenced by the acceleration and deceleration zones. Poe et al. (57) investigated the relationship between the urban road environment and vehicle speeds. In this study, the researchers defined a typical corridor as the entire roadway between the traffic control devices on both ends. The corridors were typically 1 to 2 km (3280 ft to 6560 ft) long. Fitzpatrick et al. (22) evaluated the design factors that affected vehicle speeds on suburban streets. They defined the straight section/corridor as a straight portion of a suburban arterial between horizontal curves and/or traffic control devices. The straight sections selected were at least 200 m (656 ft) from an adjacent horizontal curve and 300 m (984 ft) from adjacent signal or stop sign. The length of these sections ranged from 149 to 1398 m (489 to 4585 ft). Another study by Fitzpatrick et al. (58) investigated the operating speed on suburban arterials. In this study, there were at least 200 m (656 ft) between the study site and a signalized intersection to eliminate the effect of traffic control devices on vehicle speeds. Polus et al. (18) suggested that the study site should be at least 500 m (1,640 ft) from any intersection to avoid the effect of traffic control devices on vehicle speeds. Schurr et al. (73) studied the relationship between design, operating, and posted speeds at horizontal curves on rural two-lane highways in Nebraska. They suggested at least 300 m (984 ft) from the study site to any intersection or other elements that may affect operating speeds.

These previous urban studies indicated that the selected corridor should be located between two intersections and generally the corridor should exclude certain distances for each intersection in an effort to remove the influence of the traffic signal or similar traffic control devices on driver selected speeds. If the corridor includes the intersections, drivers may choose the vehicle speeds according to the status of traffic control devices at the intersection rather than the road environment. Speeds should also be measured for vehicles in traffic streams under free-flow conditions to avoid the impact of traffic flow characteristics on specific vehicle speeds. These previous studies generally indicated selected corridor lengths and, if included, separation distances from proximate intersections. They did not, however, delve into the question of how to determine an adequate distance from the intersection influence regions or how to determine a minimum study corridor length so that drivers could reach their desired speeds without the influence of traffic control devices.

Members of the research team also evaluated vehicle accelerations and decelerations for these lower-speed urban streets [66, 67]. The research results provide guidance in determination of the minimum length of the studied corridors between two intersections with traffic signals or stop signs so that the selected streets are long enough that drivers are able to select and achieve their desired corridor speeds without the influence of adjacent traffic control devices. The length of a selected study corridor should be at least equal to the length of acceleration zone plus the length of deceleration zone so that drivers are able to accelerate to their desired speed under free-flow conditions. Table 4 provides an estimate of the minimum corridor length required to accommodate the acceleration and deceleration zones for various speed limits on a corridor with stop sign control at the corridor end points [66, 67]. These lengths were utilized in the initial screening of corridors in the Atlanta region.

Table 4. Minimum Length for Study Corridors

Speed Limit kph, (mph)	Approximate Minimum Corridor Length, m (ft)
40 (25)	213 (700)
48 (30)	274 (900)
56 (35)	335 (1100)
64 (40)	457 (1500)
72 (45)	488 (1600)

However, as the speed a driver desires to achieve by the end of his/her acceleration and the speed at which a driver begins his/her deceleration is generally unknown, this research effort developed a heuristic by which acceleration and deceleration distances could be estimated for each individual corridor, based on the GPS trajectory data for that corridor. Thus, the final determination of the sufficiency of a corridor's length utilizes acceleration and deceleration zones calibrated to that corridors trajectory data. The complete method to determine acceleration and deceleration zones is described in Chapter 4, Data Processing.

3.5.2 Corridor Selection Criteria

The research team developed and applied the following criteria for the corridor selection process to assure, to the highest extent possible, that the sampling observations, i.e., trips in this case, fairly capture the driver's behavior from the population, that is to say, they create an unbiased data set for the desired roadway features.

1. The study focuses on the low speed urban street, thus the speed limit on the selected streets should be lower than or equal to 45 mph.

2. To help to ensure that the developed speed model is representative of roadways throughout the Metro Atlanta region, the corridors should be distributed throughout the 11 sub-regions of the Metro Atlanta area defined for this effort. The 11 sub-regions (N1, NE1, SE1, SW1, NW1, N2, NE2, SE2, S, SW2, and NW2) utilize the freeway structure as boundaries, see Figure 6. Sub-Area System Map.

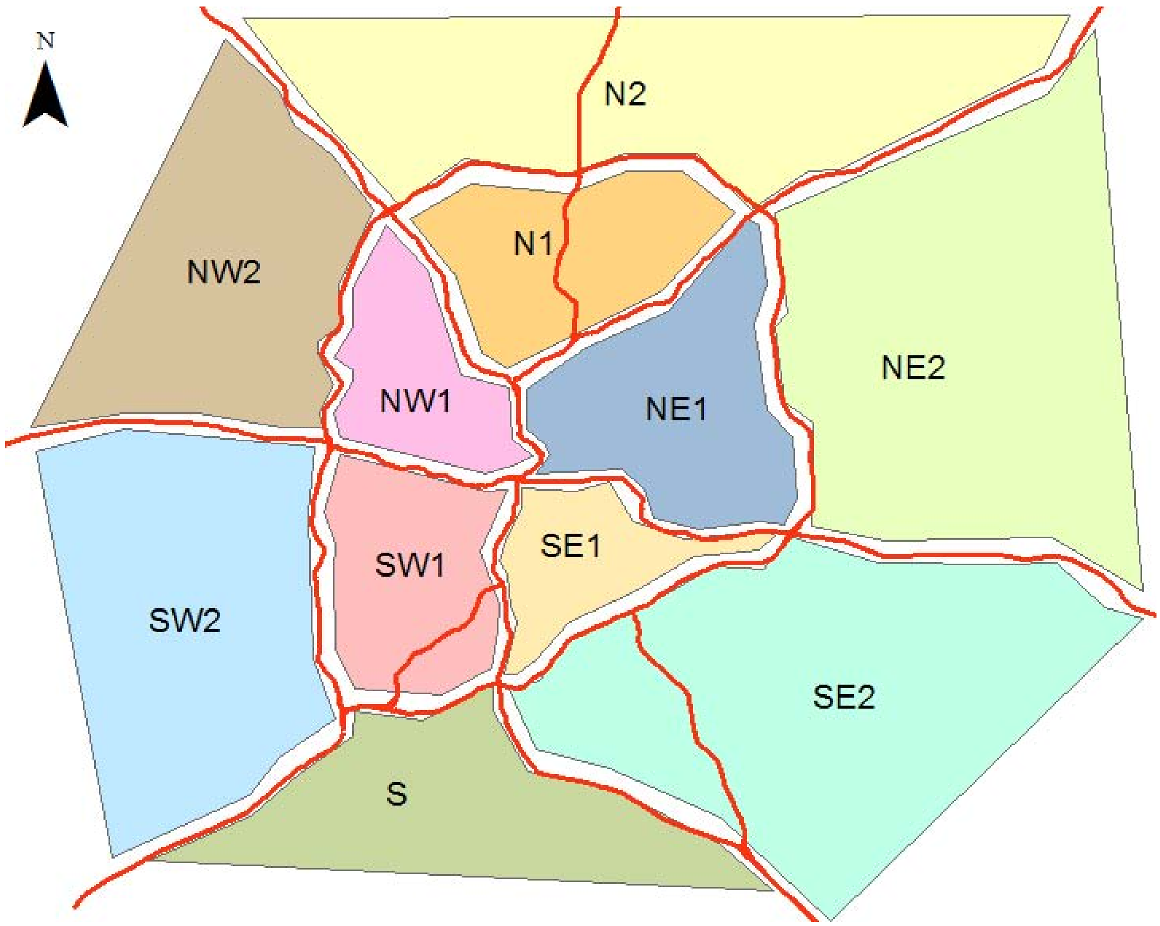

Figure 6. Sub-Area System Map

3. The number of selected corridors should be distributed among the low speed urban street functional classes to ensure that the speed data from different functional classification roadways will be included in the speed model. In other words, the corridors should represent a variety of road geometry, roadside environments, land uses, cross-sectional characteristics, and posted speed limits. This research utilizes the GDOT function classification code depicted in Table 5. For this research effort three classifications are included in the low speed urban street category, Minor Arterial (16), Collector Street (17), and Local (19). The research team excluded Urban principal arterial (14) roads as they are characterized by frequent traffic control devices and congested conditions. Road Functional Classification can be obtained from GDOT's Road Characteristics database under the field name "FUNC_CLASS".

Table 5. GDOT's Functional Classification Codes

Functional Classification Code	Description
Rural	
1	Interstate principal arterial
2	Principal arterial
6	Minor arterial
7	Major collector
8	NFA Minor Collector
9	Local
Urban	
11	Interstate Principal arterial
12	Urban freeway and expressway
14	Urban principal arterial
16	Minor arterial street
17	Collector street
19	Local

4. Candidate corridors should maintain a balance between the number of drivers, number of trips, and number of data points.

5. If the selected corridors are bounded by stop sign or signal controlled intersections, the length of the corridor should be sufficient to ensure that drivers reach their desired speed under free-flow conditions. The initial minimum distance for each speed limit category is based on Wang et al., [66, 67] as shown in Table 4.

Criterion 1 was met directly by limiting the selected corridors to those with a speed limit of 45 mph or less. Criteria 2, 3, 4 and 5 required application of the following process:

- The number of trips on each RCLINK (i.e., roadway segment as defined in the GDOT RC data base) was determined.

- The RCLINKs were ranked by number of trips on each link, from high to low. The research team selected the top one hundred RCLINKs from each Functional Classification (16, 17, and 19). From this point each list of one-hundred corridors is processed separately. Considering each Functional Class separately allows for the distribution of the study corridors among the functional classifications, i.e., corridor selection criterion 3. Otherwise the highest included roadway classification (16-Minor arterial) would dominate the data collection effort as these corridors tend to have a higher density of instrumented vehicles.

- For each functional classification list of corridors the number of trips made by individual drivers is considered. To limit the influence of any one driver on the modeling results, it is desirable that selected corridors have more than one driver and the total trips per driver are relatively balanced, i.e., corridor selection criterion 4. For example, a corridor with two drivers, each with a total of 50 trips, is preferred over a corridor with 98 trips from one driver and 2 trips from the other driver. To achieve this balance the research team calculated the average and standard deviation for the number of trips made by all drivers on each corridor. From these two parameters the coefficient of variation (defined as the ratio of the standard deviation to the mean) for each corridor is determined:

$$c_v = \frac{\sigma}{\mu}$$

  The one-hundred RCLINKs for each functional classification were then sorted according to the coefficient of variation. The RCLINKs were prioritized such that the lower the coefficient of variation, the higher the priority. The corridor prioritization may be inspected visually using GIS software, color-coding the top one-hundred RCLINK's in each road classification based on their coefficient values, i.e., corridors with lower coefficient have a darker color than the ones with higher coefficient. Figure 7 shows a selection of RCLINKS that are included in the top one-hundred lists for the Minor arterial (blue), Collector Street (green), and Local Street (orange) classifications. In addition, dark color links have higher priority than light color links.

- Through visual inspection of the corridors the team selected candidate corridors according to their priority and distribution among the 11 sub regions outlined in corridor section criterion 2. Corridors were also eliminated that did not meet the initial minimum length requirement between traffic control devices, i.e., corridor selection criterion 5.

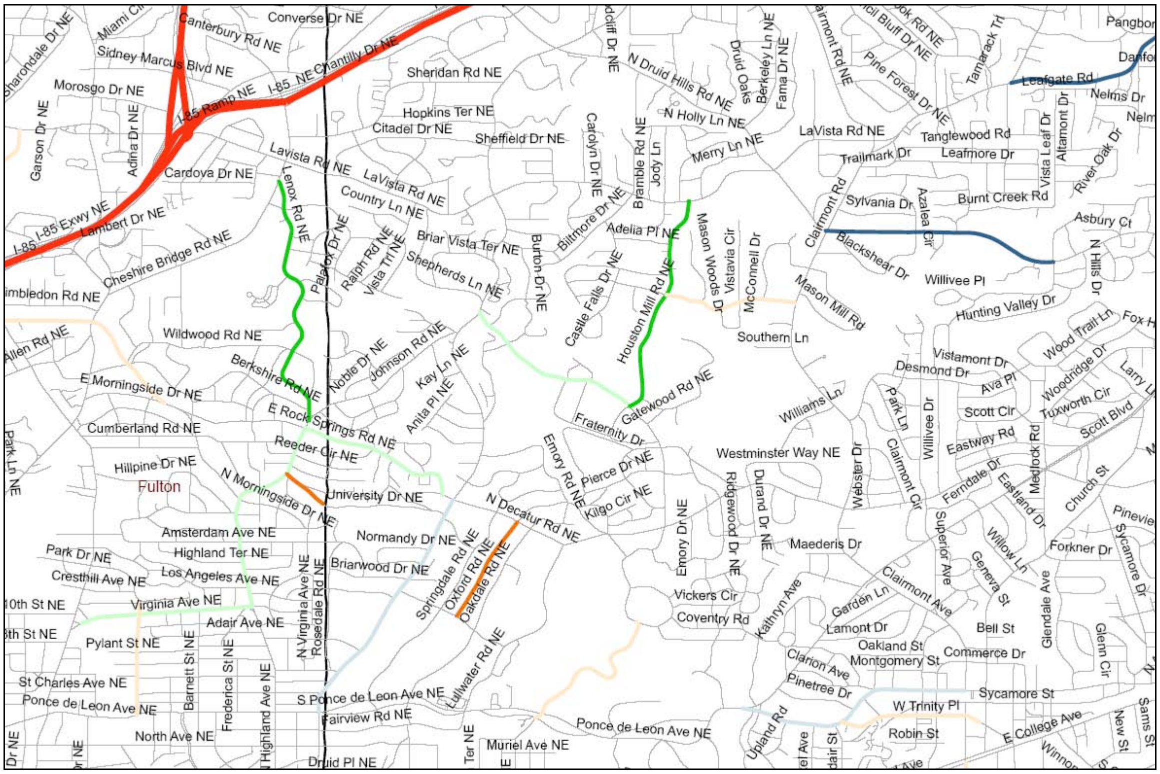

Figure 7. Color-coded RCLINKS

3.5.3 Corridor Selection Result

Ninety-two corridors were initially selected for data analysis and modeling. Out of these initial 92 corridors, 33 are Minor Arterials (36%), 32 are Collector Streets (35%), and 27 are Local Streets (29%). Figure 8 and Figure 9 illustrate the distribution of selected corridors. The quantity in the box found in each sub region in Figure 8 indicates the number of selected corridors in that sub region. It is noted that sub-regions SW1 and SE1 are under represented due to low availability of GPS data in these two sub-regions. This lower availability of GPS data is primarily explained by the sparser density of households in these regions participating in the Commute Atlanta Project. The distribution of households is depicted in Figure 10.

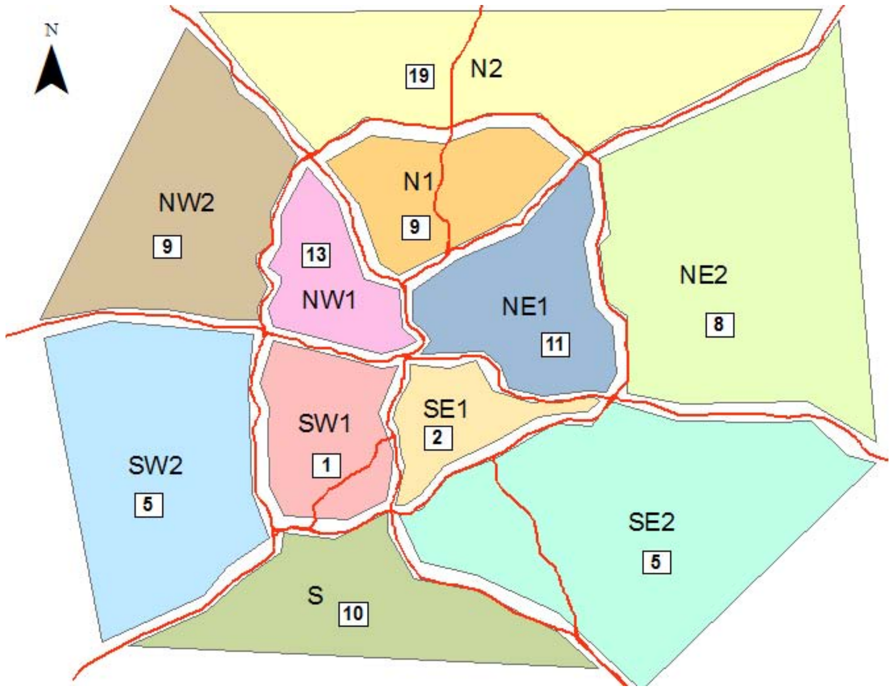

Figure 8. Sub-Area System Map with Candidate Number of Corridors (# indicates number of corridors in region)

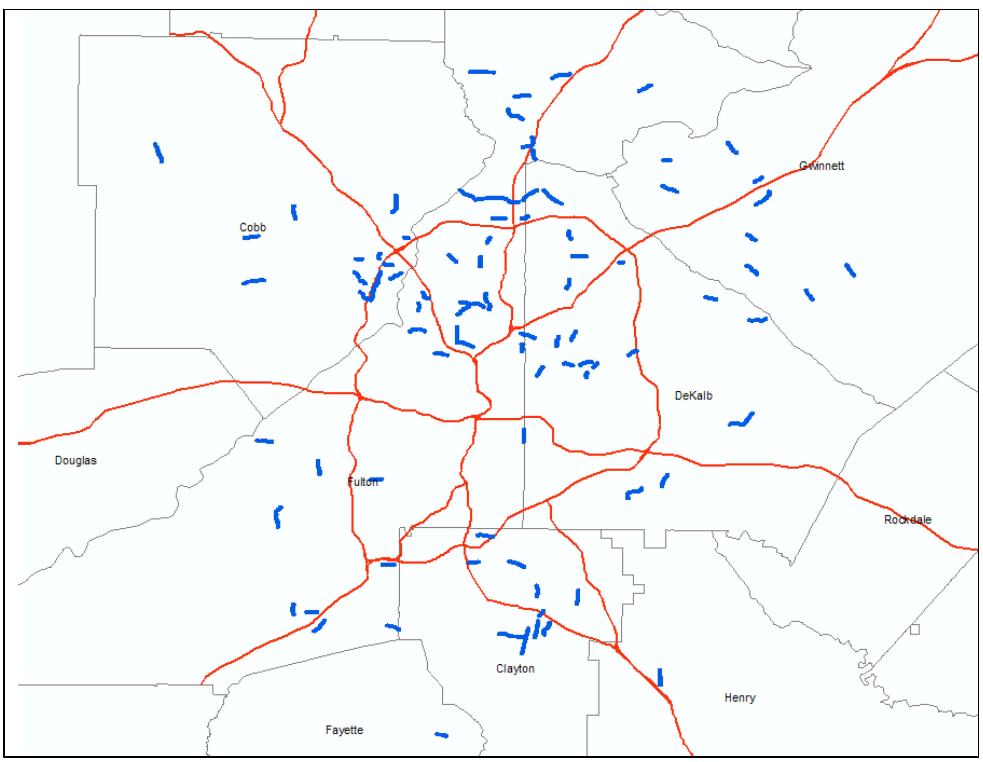

Figure 9. Locations of the 92 Selected Corridors

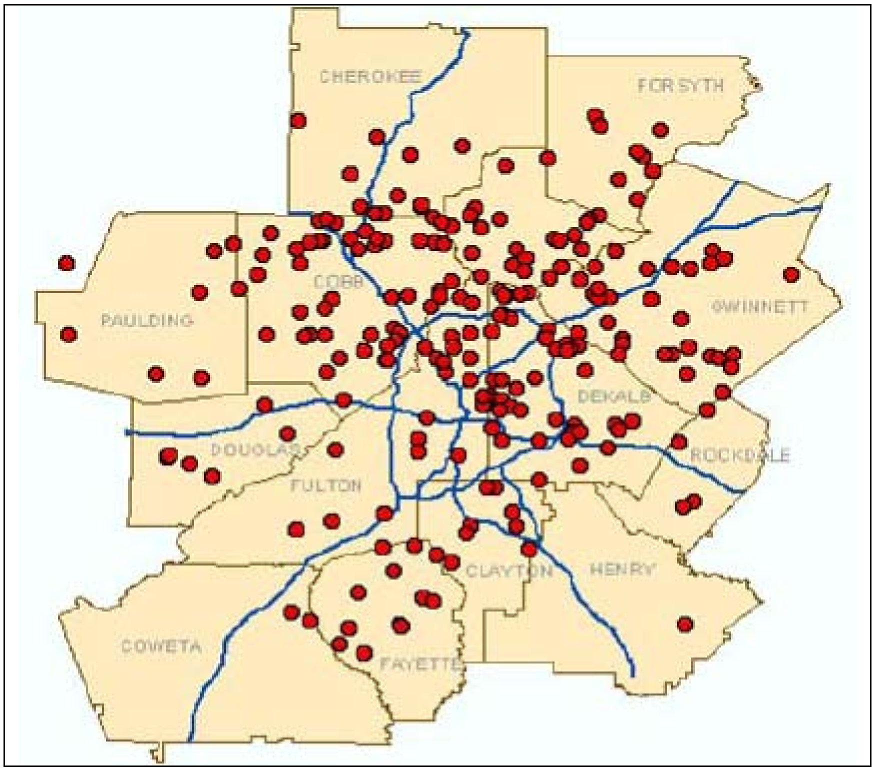

Figure 10. Locations of the Commute Atlanta Project Participating Households

3.6 Physical Field Data Collection

Roadside environment parameters such as number of utility poles, number of mailboxes, offset from the right edge of traveling lane to roadside objects, grade/slope, and other specific road environment characteristics are measured in the field. In general, the data collection process included the four following steps:

1) *Evaluate Initial corridors*: Upon arrival at a corridor (selected according to the above described corridor selection procedure) the researchers identified the starting and ending points. The corridor length was measured to verify it satisfied the minimum values recommended in Table 4. It was also verified that the corridor had a consistent cross-section with no mainline stop-control traffic control devices.

2) *Record roadside features*: The research team created video recording of the corridor roadway and roadside in each direction of travel. For locations with a raised median an additional video recording to identify median features was captured. To assist in project data organization the first video recording trip of a corridor is always south to north or west to east. Figure 11 depicts the video recording travel procedure for an east-west corridor with a median.

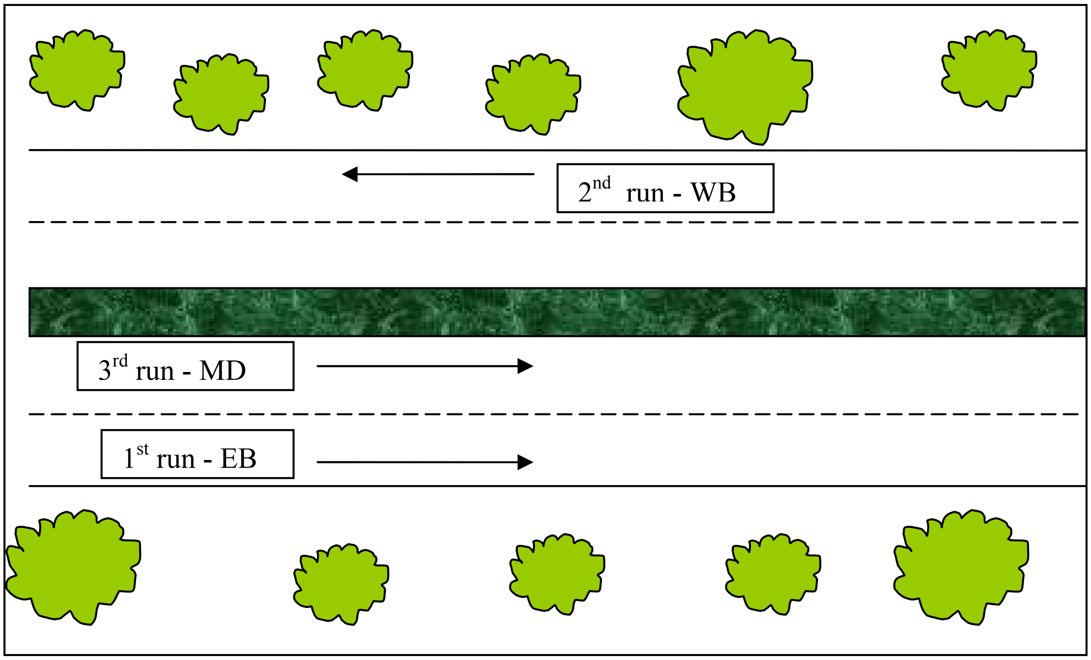

Figure 11. Example of Video Recording "Runs" on East-West corridor

The camera was oriented such that roadside features within 7.6 to 9.2 m (25-30 ft) from the edge of pavement on the right side were within view (Figure 12). The research team recorded a static movie title image as shown in Figure 13 at the beginning or each video trip to identify the corridor number, location, and direction of travel.

Figure 12. Typical Camera Orientation

ID: 51	Direction: NB
Street Name: Jonesboro Rd	
From: Battle Creek Rd	
To: SR 138	

Figure 13. Example of Video Label for Northbound Corridor 51

3) *Create roadway grade profile*: Members of the research team measured the grade/slope of the roadway at a number of spot locations along each corridor. Grade information is acquired near the starting and ending points of the corridor (determined by visual inspection), at representative locations on a grade, near the beginning and ending points of vertical curves, and the low or high points for sags or crest curves. The field data team always measured grade from south to north or west to east. Figure 14 depicts a sample road profile schematic.

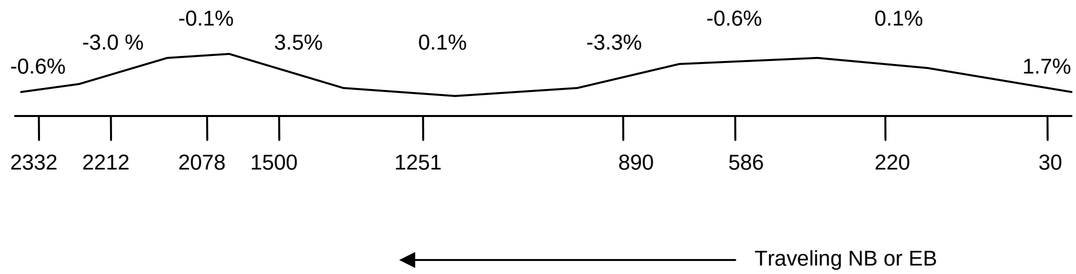

Figure 14. Example of Road Profile Sketch

4) *Determine roadway and road features geometrics:* The research team measured the offset of trees, utility poles, mailboxes, street signs, or other objects that may influence operating speed. The offset distance is measured from the object to the white solid pavement edge line, or the face of a raised curb at locations without a painted edge line. The measurement accuracy is to the nearest foot, e.g., 12'4" was recorded as 12'.

   The research team approximated horizontal curvature data using scaled aerials and a coordinate geometry program. Two team members independently estimated each horizontal curve radius and the research team used a composite value of these estimations in this study.

3.7 Summary

Chapter 3 has provided an overview of the data utilized for this study. Included in this chapter is a review of GPS and the in-vehicle instrumentation, the integration of the GDOT geographical information system (GIS) roadway data, the corridor selection process, and supplemental field data collection. It was shown that this study utilizes selected drivers' vehicle trajectory data for the year 2004. Trajectory data is collected using data from a fleet of vehicles equipped with GPS. The vehicle trajectory data collection is maintained as part of the Commute Atlanta Project, funded by the Federal Highway Administration (FHWA). Chapter 4 will next provide step-by-step detail for the trajectory data processing.

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4 DATA PROCESSING

As stated in Chapter 3, this study analyzes the selected drivers' vehicle trajectory data over a one year period, from January 2004 to December 2004 using data collected with in-vehicle GPS. This data was pre-processed by the Commute Atlanta project team before being distributed to the operating speed project. Only data points that fall inside a 50-feet buffer area encompassing a study corridor were provided for trajectory analysis. Potential study corridors are limited to those with available speed data. The Commute Atlanta team provided data for the candidate corridors in twelve data files, one file for each month of the year. Each file contained all of the vehicle trips that occurred on all of the corridors for the respective month. The research team created fourteen data processing steps to prepare the final data set for use in the statistical modeling.

4.1 Data Processing Steps

4.1.1 Step 1 -- Data Formatting

Step 1 removes instrumented vehicle data attributes included in the FHWA Commute Atlanta raw data that may potentially allow for the identification an individual study participant. The data attributes removed include driver's gender, driver's age range, vehicle type, and vehicle model year. The data attributes remaining after this step include the field names identified in Table 6. Table 7 depicts the number of instrumented vehicle data points (i.e., total seconds of data) in each monthly raw data file. Over the entire 12 month period there were a total of 6,616,991 second-by-second data points, roughly equivalent to 1,838 hours of travel time.

Upon completion of the thirteen data processing steps, some of the removed attributes were returned to the data set, such as gender and age, to allow for improved statistical modeling. However, the returned data is encoded such that study participant anonymity remains assured.

The RCLINK identifier listed in Table 6 represents a unique ID assigned to each route in the GDOT roadway system RC file and is composed of the codes depicted in Table 8. For example, a section of Clairmont Road in Atlanta is coded as RCLINK 089 1 0155 00. This is interpreted as DeKalb County (089), a State Route (1), and GDOT route number (0155). The last two digits, 00, also indicate this link is a State Route. Note the RCLINK number for a route will change should the route cross a county boundary. Along with the RCLINK number each route is subdivided into segments distinguished by mile points.

Figure 15 depicts an example of how a route is composed of several small segments identified by RCLINK and mile points. The color coding in this figure depicts segment BEG_MP (beginning mile point) along a link. The GDOT RCFILE contains recorded roadway characteristics for each RCLINK and mile point segment.

Table 6. Descriptions for Field Names Utilized from the Commute Atlanta Data Files

Field Name	**Description**
TRIP ID	Trip ID Identification code with date and time identifying trip start.
DATE	Date in format yyyymmdd. For example, 20040605 is June 5th 2004.
TIME	Greenwich Mean Time (GMT) in 24-hour clock format hhmmss. For example, value of "143230" means 14:32:30.
LAT	Latitude of the coordinate location (6-digit precision)
LONG	Longitude of the coordinate location (6-digit precision)
SPEED	Travel speed in miles per hour (2-digit precision)
HEAD	Direction of travel, measured clockwise from North bearing.
SAT	Number of satellites in view (acceptable when SAT >= 4)
PDOP	Position Dilution of Precision (acceptable when 1<= PDOP <= 8)
RCLINK	GDOT Route Identification Number. Provides relational link between route features and the RCFILE (see Table 8).
BEG_MP	Mile point along route demarking the beginning mile point for a segment, measured as a distance from the Route 0.00 mile point.

Table 7. Quantity of Instrumented Vehicle Data in Received Commute Atlanta Data Files

MONTH (Format: YearMonth)	**DATA POINTS (seconds)**
200401	600,742
200402	563,190
200403	647,008
200404	639,303
200405	599,600
200406	569,480
200407	526,109
200408	532,007
200409	533,661
200410	491,073
200411	428,709
200412	486,109
TOTAL	6,616,991

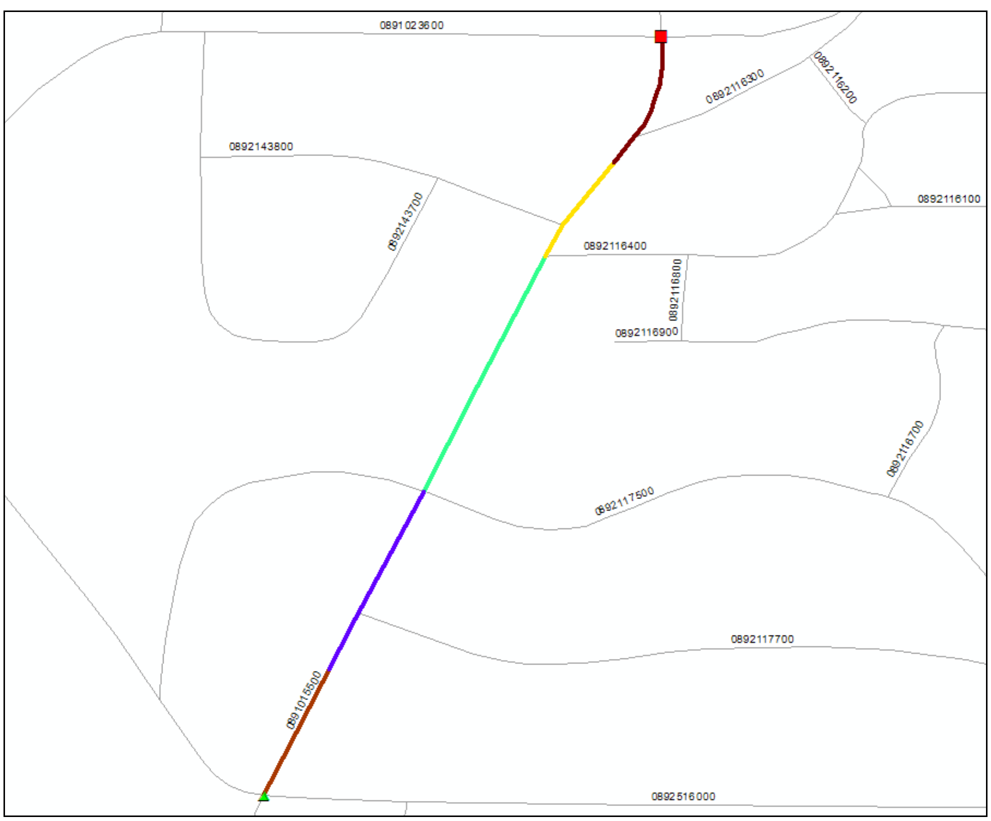


Figure 15. RCLINK Representing Segment of Clairmont Road (Highlighted)

Table 8. RCLINK Code Definition

Position	Description
1-3	County FIPS Code
4	GDOT Route Type 0 – Unknown Road 1 – State Route 2 – County Route 3 – City Route 4 – Col Route 5 – Unofficial Route 6 – Ramp/ Interchange 7 – Private Road 8 – Public Road 9 – Collector-Distributor Roads
5-8	Actual number of the road
9-10	00 – State Route or County Route, none of the following NO – North SO – South EA – East WE – West AL – Alternate BY – Bypass SP – Spur CO – Connector LO – Loop TO – Toll DU – Dual Mileage AD – Alternate Dual BD – Business Dual BC – Bypass Connector CD – Connector Dual SD – Spur Dual NN – City Suffix Number

4.1.2 Step 2 -- Create Road Link ID's

Step 2 creates a unique tag (Link ID) for each road segment. The Link ID is a combination of the RCLINK and beginning mile point value for the subject road segment. A data field is then added to every second of instrumented vehicle data containing the Link ID associated with the vehicle location. In later steps this Link ID is utilized to efficiently relate data between the instrumented vehicle data files and RCFILE database

4.1.3 Step 3 -- Assign Instrumented Vehicle Data to Selected Corridors

As previously indicated, the instrumented vehicle data from the FHWA Commute Atlanta project is grouped by month, i.e., January 2004, February 2004, to December 2004, for a total of 12 data files. Step 3 sorts the data into a corridor based file system, each file containing the entire 12 months of data for the subject corridor. This sorting is accomplished using the Link ID (created in step 2) to assign each instrumented vehicle data point from the monthly data files to its associated corridor file. This sorting resulted in ninety-two files, one for each corridor under consideration. Figure 16 illustrates the original and new file structure.

This step included a visual inspection of the data for each corridor. The GPS data was superimposed on a GIS map and the data point locations were inspected for consistency with the corridor locations. GPS points located significantly far from the corresponding corridor were removed from the data set. Figure 17 provides an illustration of the GPS data points located along Corridor No. 33, Dunwoody Place between Northridge Parkway to Roberts Drive. The data in each of the 92 files at the end of this step includes both directions of travel (where both exist), i.e., northbound and southbound, or eastbound and westbound.

Table 9 provides a sense of the corridor files sizes, listing the number of data points, number of drivers, and number of trips for the first 15 corridors.

ORIGINAL VEHICLE DATA
"Jan 2004" data file
"Feb 2004" data file
…
"Dec 2004" data file

⇒

REGROUPED VEHICLE DATA
"Corridor # 1" data file
.
.
"Corridor # 92" data file

Figure 16. Data File Structure at Start and End of Step 3: Assign Instrumented Vehicle Data to Selected Corridors

Figure 17. Plotted GPS data for Corridor No. 33, Dunwoody Place

Table 9. Records, Drivers, and Trips by Corridor

Corridor ID	Number of Points	Number of Drivers	Number of Trips
1	127,289	289	1,313
2	147,168	330	1,627
3	241,962	310	2,339
4	137,563	328	1,563
5	27,563	250	897
6	67,662	326	1,432
7	44,984	235	1,254
8	69,689	235	2,422
10	149,916	346	2,026
11	63,115	294	2,440
12	138,033	242	1,866
13	20,375	204	735
14	83,743	315	1,156
15	59,043	280	654
.			
..			

4.1.4 Step 4 -- Sorting and Removing Duplicates

Step 4 performs a multilevel sort of the data in each file, first sorting by TRIP_ID, then DATE, then TIME. At the conclusion of Step 4, duplicate records are identified and removed. Approximately 0.04% of data were detected as duplicate and removed by this filter.

4.1.5 Step 5 -- Detecting sub-trips in a trip

The Commute Atlanta Project defines a trip as the duration from engine on to engine off. The Trip ID will change only when the driver turns off the engine. Thus, if a vehicle leaves a corridor and then returns without turning off the vehicle (for example to drop of a passenger), this is identified as a single trip in the step 4 output.

For this project it is necessary to define each trip as a period of continuous travel over a corridor. To accomplish this two fields are appended to each data point. The first is the gap time (T_TIME), defined as the time in seconds from data point (i-1) to data point (i) of each vehicle trip, i.e., the time between consecutive instrumented vehicle data points. The second additional field is sub-trip ID (SUB). This field is used to identify the separation of a single vehicle trip into multiple trips. A trip is divided into multiple trips when a gap between consecutive points larger than a predetermined value is identified. A sensitivity analysis of GPS data found that a 10 second gap threshold effectively captures multiple trip occurrences while avoiding breaking a single trip into multiple trips due to a momentary loss of the GPS signal. Figure 18 depicts an example of dividing a single trip as defined in the Commute Atlanta data into multiple continuous travel trips. In this example a single trip (i.e. engine-on to engine-off) crosses the link twice, once eastbound and once westbound. For the modeling analysis this is effectively two trips. Where the 10 second gap threshold is met (i.e. the time the vehicle is not on corridor between crossings) this trip will be divided into two trips for analysis.

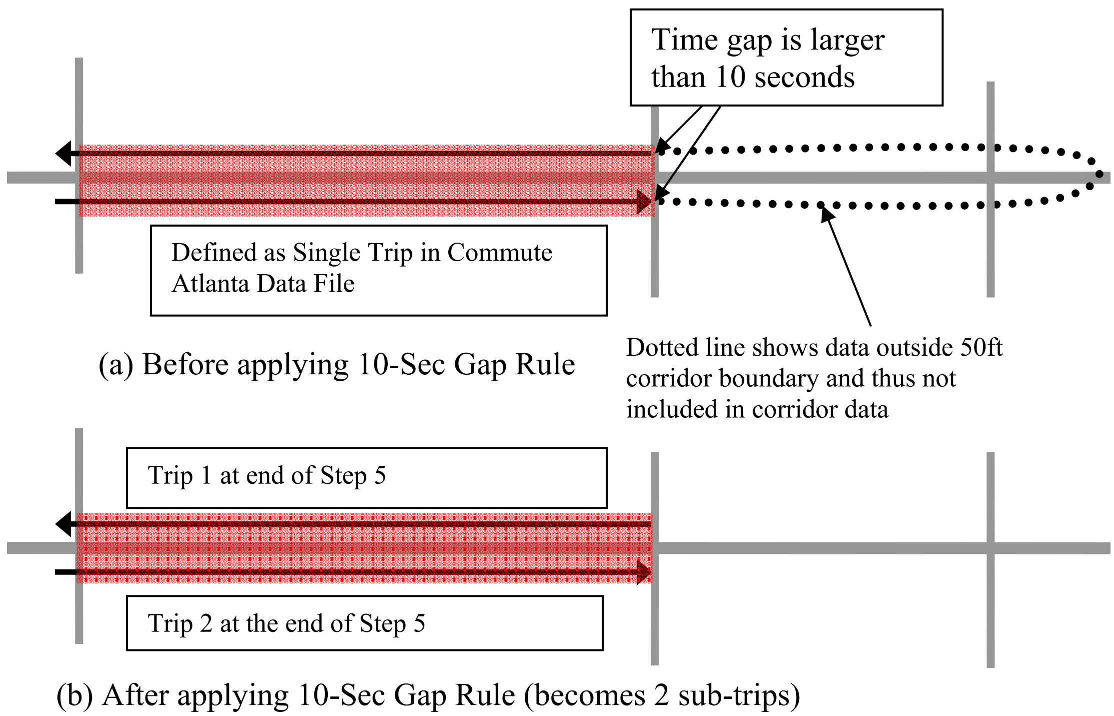


Figure 18. Example Scenario for Application of the 10-second Gap Rule

4.1.6 Step 6 -- Check for Complete Trips

Trip inclusion in the final modeling stage also requires that the trip traverse the entire corridor length. The research team noted that some vehicles either entered or departed the roadway at an internal corridor point, such as a driveway, gas stations, etc., and thus did not traverse the entire corridor length. These trips were defined as incomplete trips and removed from additional analysis. The Step 6 filter checked whether a trip passed within 100 ft of the corridor endpoints. If so, the trip was considered a complete trip. Otherwise, the trip was discarded and not considered in subsequent analysis. Figure 19 shows the driving activity data for a trip traveling westbound. As this trip passed within 100-feet of both endpoints, it is considered a complete trip. Figure 20 illustrates driving activity of another trip also traveling westbound. This trip passed through one endpoint but not the other. Thus, this trip is considered an incomplete trip and is removed from subsequent analysis.

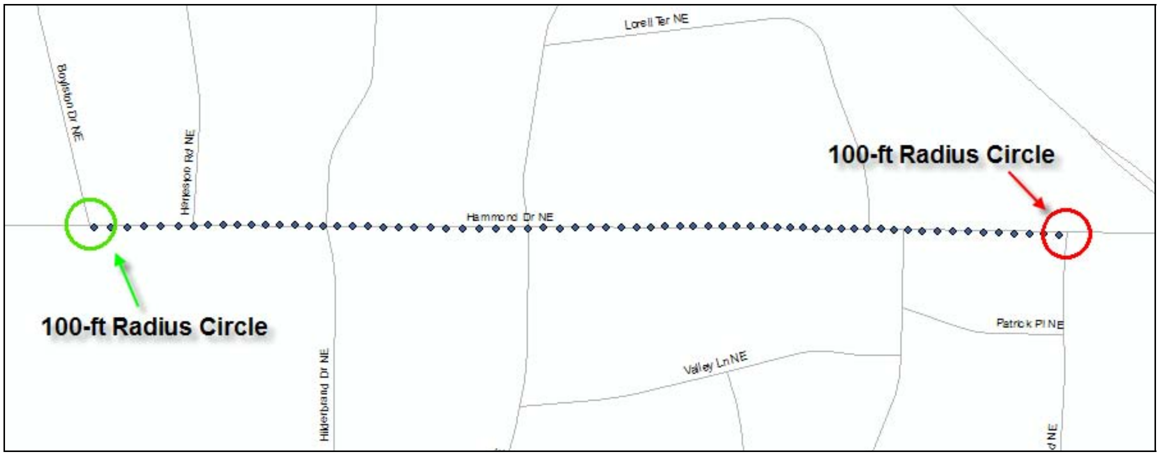


Figure 19. Example of a Complete Trip on Hammond Drive

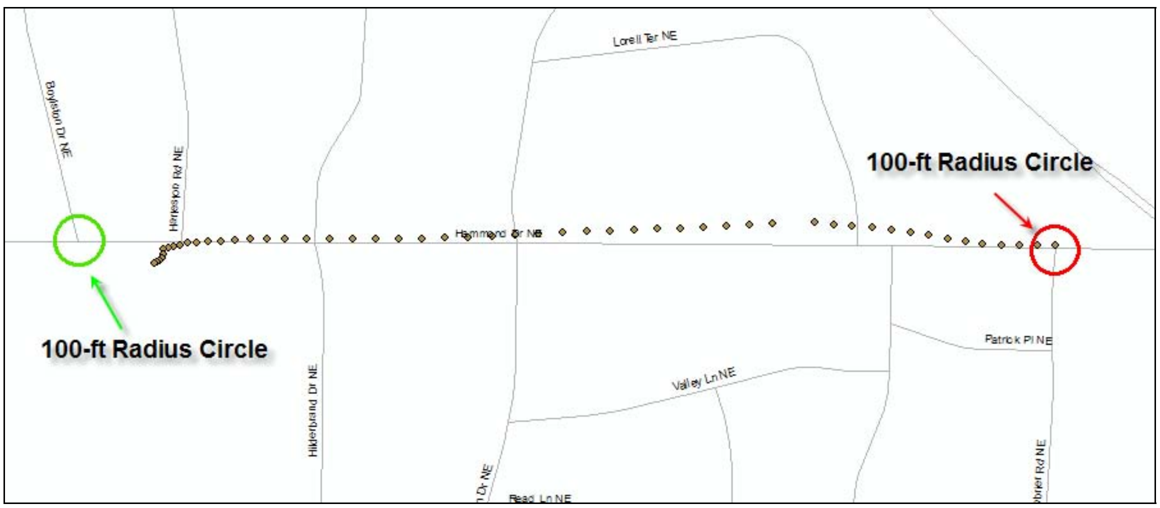


Figure 20. Example of an Incomplete Trip on Hammond Drive

Figure 21 shows the combined application of Step 5 and Step 6 in the processing of a complicated trip data file found on Hammond Drive. First, Step 5 separates the trip into three sub-trips, identified by the time gaps of 29 seconds and 385 seconds. Note that two of the sub-trips are westbound while one sub-trip is eastbound. Next, Step 6 evaluates the starting and ending locations of each sub-trip. As a result, only the eastbound trip satisfies the endpoint criteria. The application order of these trips is critical to their operation. If Step 5 did not initially separate the single initial trip into three trips, then Step 6 would not have detected and removed the partial westbound travel and carried forward the complete eastbound travel.

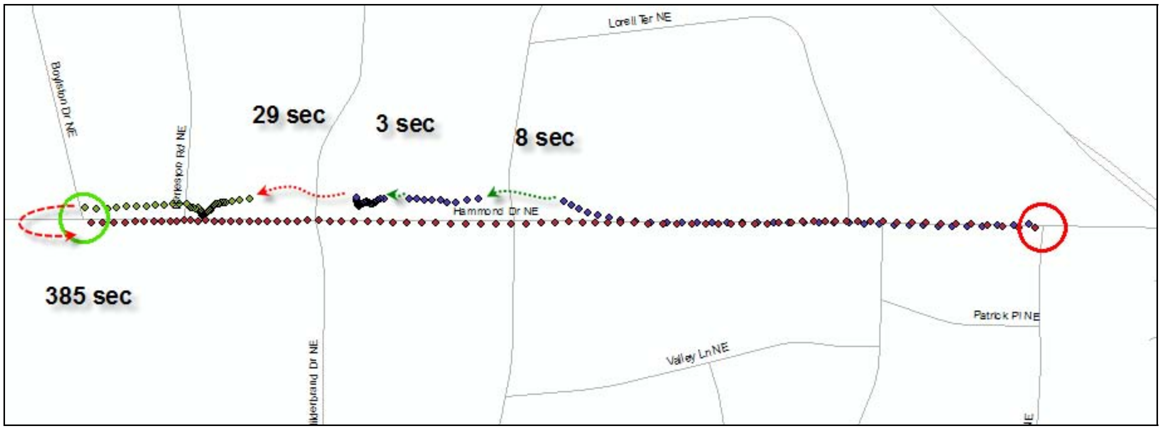


Figure 21. Example Application of Steps 5 and 6

4.1.7 Step 7 -- Define Direction of Travel

At this stage in the data filtering process each corridor file potentially includes two directions of travel, northbound and southbound or eastbound and westbound. In the upcoming statistical modeling, the direction of travel for each trip is required. Thus, step 7 determines the trip direction for each trip on each corridor. In this step the west/south corridor endpoint is defined as endpoint A and the east/north endpoint is defined as endpoint B. The research team manually determined the default orientation of each corridor as 1) south-to-north or 2) west-to-east and the endpoints A and B. An automated process is utilized to compare the timestamps at which a vehicle passes points A and B during a trip. If the vehicle passed the corridor's endpoint A before the endpoint B, the trip is either northbound or eastbound. Otherwise if the vehicle passed endpoint B before endpoint A the trip direction is determined to be southbound or westbound, depending on the corridor's orientation.

This step also added a data attribute to each instrumented vehicle data point labeled SDIST. SDIST is the distance in feet from the starting point of the corridor to the respective instrumented vehicle data point location. The SDIST for any data point is determined as $SDIST_{i-1}$ plus the distance between data point (i-1) and (i), where $SDIST_{i-1}$ is the distance from the corridor starting point to the GPS location of instrumented vehicle at point (i-1). The distance between the two consecutive instrumented vehicle points (i-1) and (i) may be calculated based on speed data or coordinate data. For the speed-derived distance, the distance is calculated from the average of the speed (in ft/sec) at point (i-1) and point (i) multiplied by the time difference between those two points, usually a 1-second period. The coordinate data method determines the distance between two consecutive points using the latitude and longitude information. In the calculation of $SDIST_{i,}$ the distance between data point (i-1) and (i) is based on the average of these two approaches.

4.1.8 Step 8 -- Convert GMT to Local Time and Remove Nighttime Trips

There is a high likelihood that lighting conditions may impact a driver's selected free-flow speed. For the statistical modeling stage, the desire is to consider daytime travel only, eliminating the confounding influence of lighting conditions. A trip is considered to be a nighttime trip in this study if the trip is made before sunrise or after sunset. Since the sunrise and sunset times vary significantly throughout the year, the research team calculated sunrise and sunset times specific to each trip. The sunrise and sunset times were calculated using an automated script process. A sun altitude of -0.833 degrees is chosen in the determination of sunrise/sunset as it is the position where the upper edge of the disk of the sun touches the earth's horizon, accounting for atmospheric refraction. The research team also adjusted the calculated sunrise and sunset times by adding a 30-minute buffer to the sunrise time and subtracting a 30-minute buffer from the sunset time. Approximately 23 percent of the remaining trips were detected as nighttime trips and removed by this filter.

As part of the separation of daytime and nighttime trips it is also necessary to adjust the timestamp recorded by the instrumented vehicles. The GPS timestamps were recorded based on Greenwich Mean Time (GMT), thus it is necessary to convert the GMT time to Eastern Standard Time (i.e. the time zone of Atlanta) prior to applying the sunrise/set times determined in the first part of this step.

4.1.9 Step 9 – Remove Trips Under Inclement Weather Conditions

Inclement weather may influence a driver's speed. This step removes trips that likely occurred during rain conditions. Snow/ice conditions were not observed during the study period. The determination of potential inclement weather during a trip is based on the hourly precipitation data from several weather stations in Metro Atlanta. These weather stations are located at the Fulton County Airport, Dekalb-Peachtree Airport, and Hartsfield Atlanta Airp ort (see Figure 22). A trip is removed if measurable rainfall is recorded at the two closest stations during the 2-hour time window before the trip. This rule removed approximately 20 percent of the trips remaining after Step 8. While a portion of these removed trips likely did not experience inclement weather, the research team chose to implement a conservative rule, trading an increased likelihood of eliminating non-inclement weather trips for decreasing the likelihood of not eliminating inclement weather trips.

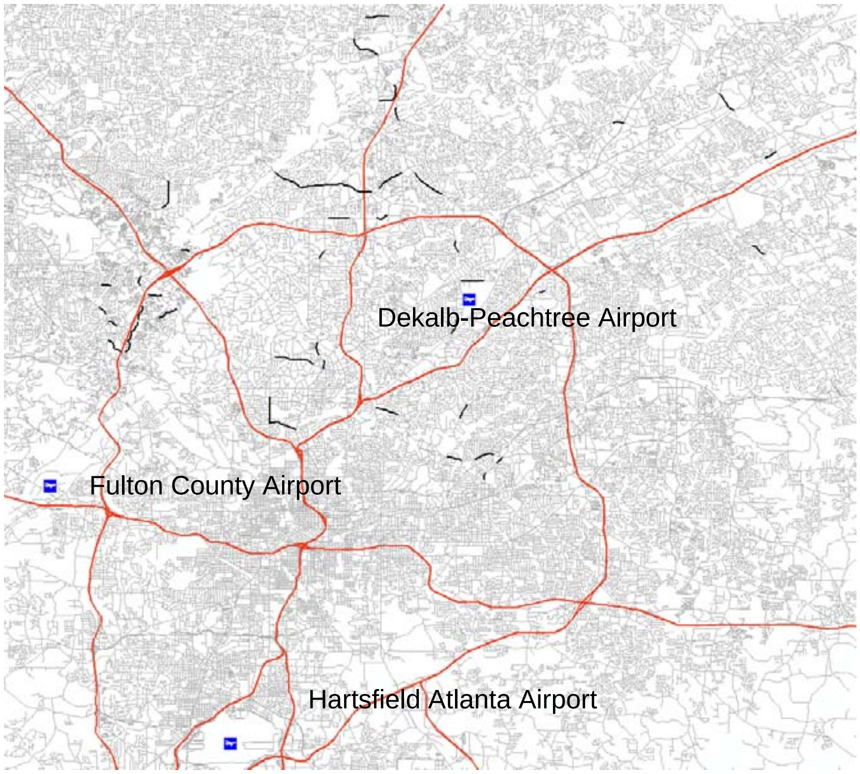


Figure 22. Locations of the 3 Weather Stations

4.1.10 Step 10 -- Remove Potentially Non-Free-flow Trips

The instrumented vehicle data does not provide a direct measure of the operational conditions under which a trip is taken, i.e., free-flow or non-free-flow. Here free flow speed is defined as the desired speed of the driver or speed selected by the driver given the roadway design. Under non-free-flow conditions a driver selects their speed in response to the interaction with other vehicles. Thus, in Step 10 the research team applies a series of developed heuristic filters utilizing the characteristics of the GPS trajectory data to help identify and remove trips that were likely non-free-flow trips. As a first step in developing these filters, the research team constructed a Graphic User Interface (GUI) application called the GPS Speed Profile Viewer. This application plots the speed profiles – the plot between distance (feet) from the corridor starting point (X-axis) and the vehicle speed in mph (Y-axis) – for all trips, or trips during a user selectable time period, that occurred on a corridor. Figure 23 depicts the speed profile of westbound trips on Corridor No. 20, Hammond Drive, between Perimeter Center Parkway and Peachtree Dunwoody Road.

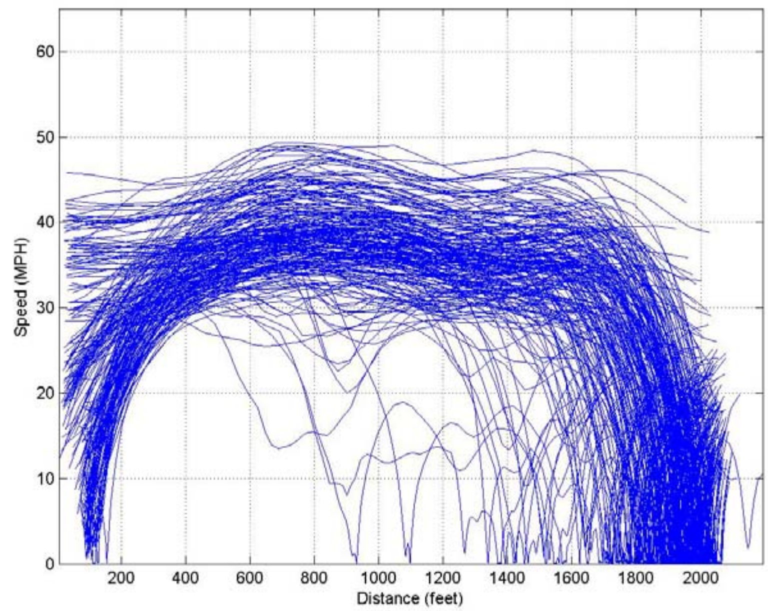


Figure 23. Example Speed Plot using the Speed Profile Viewer

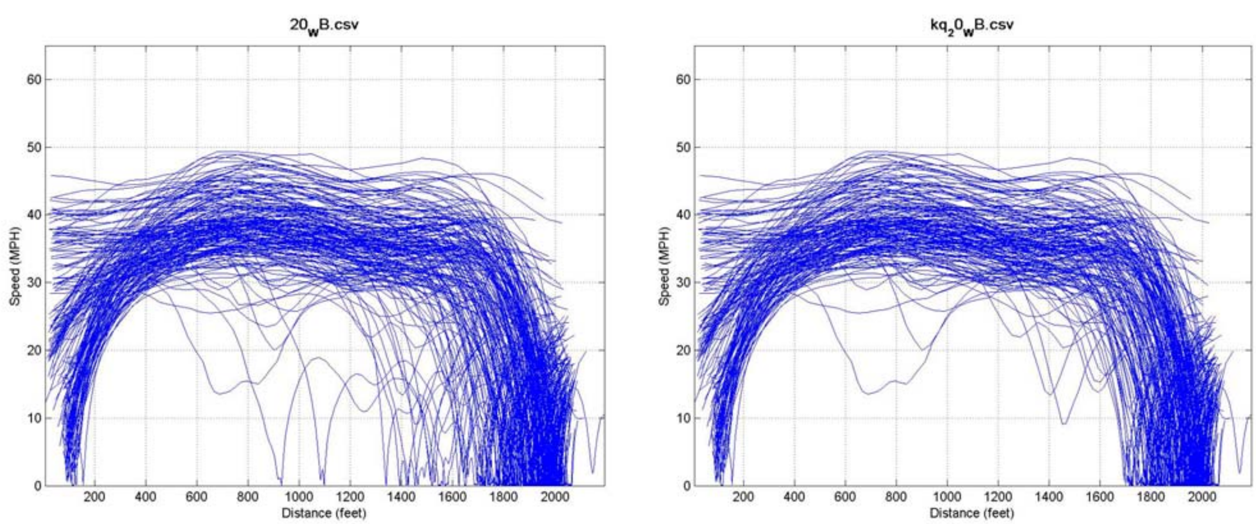
Figure 23 illustrates that a number of vehicles may have stopped or significantly slowed in the corridor mid-section during their trips. From the graph, it is clearly seen that these stopped and slowed vehicle trips are not in free-flow operation. Each of the filters developed to identify and remove these trips is summarized in the following sections.

An alternative initially considered by the research team was a time-of-day filter to remove all peak period traffic and define free-flow as non-peak period traffic. However, upon inspection of the candidate corridors the research team identified several irregular peak hour periods in commercial and warehouse districts and commonly accepted non-peak hours clearly exhibiting non-free-flow trip characteristics. Thus, time-of-travel based filters did not adequately remove non-free-flow trips or necessarily retain all free-flow trips. To overcome the peak time based filter drawbacks, the research team developed a combination of filters based on trip characteristics, as described in following sections. This approach successfully removed the peak and non-peak hour trips that did not exhibit free-flow behavior, in essence enabling the use of variable peak hours with respect to the individual corridors.

A. Deceleration Queue Filter

As seen in Figure 23, vehicles may enter a queue at the downstream (deceleration) end of a trip. When the stopping location of the vehicle indicates a significant queue length the vehicle should not be assumed as free-flowing on the upstream portion of the corridor, as a lengthy queue indicates likely congested or driver constrained conditions. For this study a queue extending at least 15 vehicles was determined to represent a significant queue (to be discussed in the following paragraph). A queue length of 300-feet implies a 15 vehicles queue, assuming each vehicle occupies approximately 20-feet. A queue buffer zone extending 400-feet upstream of the center of the trip end intersection helped identify vehicles that were likely within at least 300-feet of the intersection. As the "destination" trip end coordinate occurs at the center of the intersection, the additional 100-feet (400' – 300' = 100') accommodated the intersection width (center to stop-bar dimension) and provided some additional leeway to account for GPS distance fluctuations.

The deceleration queue filter removes vehicles that stop between the mid-point of the corridor and the beginning of the 15-vehicle queue region. The research team selected the 15-vehicle (400-feet) value following a pattern recognition and sensitivity analysis for each intersection. Initially the research team investigated a separate queue value for each functional classification; however, the 400-feet value conservatively identified queued vehicles for all locations and subsequent free-flow filters capture other irregular trips. Figure 24 illustrates the effect of the Deceleration Queue Filter. Trips that experienced speeds lower than 5 mph between the midpoint (1000 feet from the starting of the corridor) and 400 feet from the downstream intersection (2100-400 = 1700 feet) were excluded from further analysis. Approximately 6 percent of the trips were removed by this filter.



(a) (b)

Figure 24. Trip Speeds (a) Before and (b) After Applying Deceleration Queue Filter

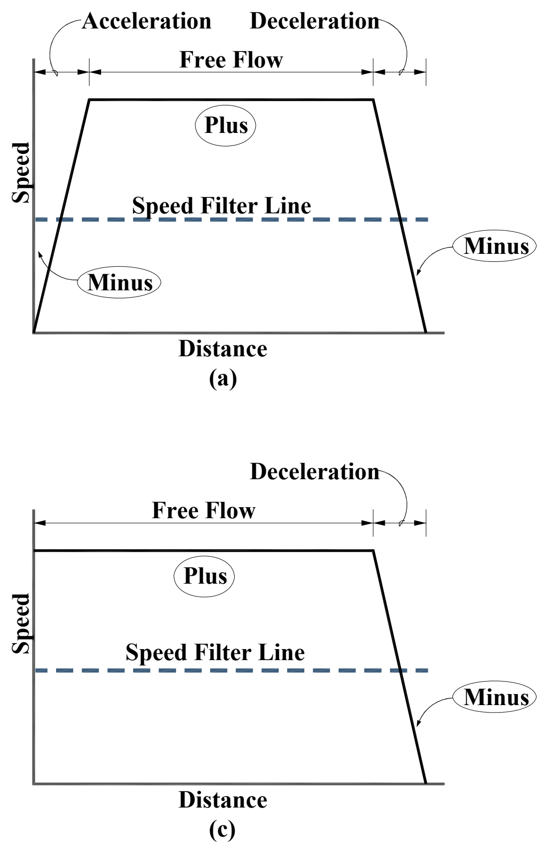


Figure 25. Four Speed Patterns Defining Potential Free-Flow Speed Trips

B. Ten-MPH Filter

Figure 25 provides a simplified presentation of the four potential free-flow speed profile trip patterns. Figure 26 depicts a simplified presentation of four potential "non-free-flow" trip patterns. The patterns depicted in Figure 26 may occur, for example, when a study vehicle is trailing a vehicle that reduces its speed to execute a turn mid-block. (Recall that if the instrumented vehicle itself turned off the road mid-block the trip is filtered out in Steps 5 and 6.) To remove trips that are clearly not free-flow due to this phenomenon, the research team used a 10 mph filter that identified trips that experienced speeds less than 10 mph outside the acceleration or deceleration zones.

Through visual inspection, identifying a trip that violates this rule is a relatively simple matter. However, due to the large number of trips and the desire to test the sensitivity of overall trip loss to the filter cutoff value, the research team developed an efficient automated implementation of the rule. This was accomplished through the use of a pattern recognition approach where a negative sign represented speeds less than a designated filter speed (in this case 10-mph) and a positive sign identified speeds greater than the designated filter speed.

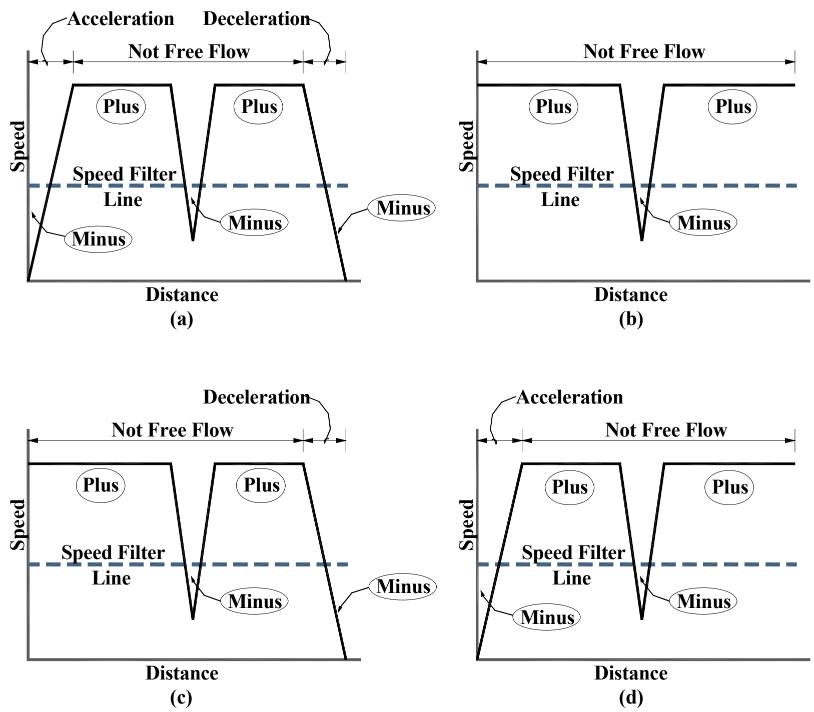


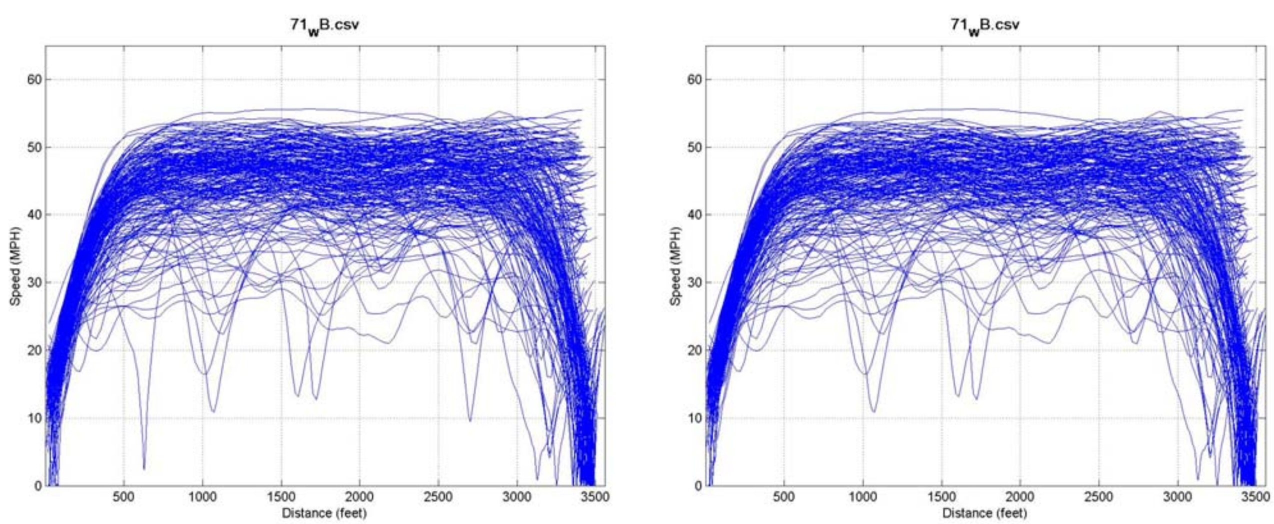
Figure 26. Four Speed Patterns Defining Potential Non-Free-Flow Speed Trips

The first step in the pattern recognition checked the pattern sign at the corridor mid-point. For a trip to be useful in the statistical modeling stage, the research team assumed that the vehicle must be traveling at free-flow speed by the corridor mid-point. Any trip with a negative pattern sign (i.e., speed less than 10 mph) at the mid-point could be safely assumed to not be traveling at free-flow speed and was removed from further analysis.

Next, the pattern recognition algorithm considered speed data in the area starting from the upstream intersection to 400 feet before the end of the corridor, previously defined as a queuing area. When a vehicle entered the corridor after being stopped (i.e., the vehicle was stopped at a red light or stop sign), the free-flow pattern would consist of a negative (or minus) sign followed by a positive (plus) sign, indicating the vehicle accelerated to a speed greater than 10 mph. See Figure 25a (vehicle stops at downstream intersection) and Figure 25d (vehicle does not stop at downstream intersection). If the same trip had an additional change from positive to negative to positive -- representing vehicle deceleration to a speed below 10 mph and then acceleration to a speed above 10 mph -- the trip was identified as a non-free-flow trip and was removed from the free-flow data set. This pattern recognition procedure eliminated trips that conform to Figure 26a and Figure 26d.

The final step in the pattern detection evaluated trips that began in the positive zone then remained positive throughout the trip or moved to the negative zone. That is, the upstream intersection was either not stop-controlled or the signal was green and the vehicle entered the corridor at greater than 10 mph and then may or may not have stopped at the corridor end point. Again, the pattern recognition algorithm considered speed data in the area starting from the upstream intersection to 400 feet before the end of the corridor. If any of these trips entered the positive zone after entering a negative zone, they were identified as non-free-flow trips and were then removed from the free-flow data set. This pattern recognition procedure eliminated trips that conform to Figure 26(b) and Figure 26(c).

At the conclusion of the 10 mph filter, the only remaining trips with speed data below 10 mph are limited to trips where the 10 mph speed must have occurred while accelerating at the trip start or decelerating at the trip end. This filter does not require a strict definition of the length of the acceleration and deceleration zones, other than a general assumption that acceleration is complete by the corridor midpoint and deceleration does not begin until after the midpoint. Approximately 8% of the available trips were detected and removed by this filter. Figure 27 depicts a speed profile before and after the 10 mph filter.



(a) (b)

Figure 27. Speeds (a) Before and (b) After Applying 10 mph Filter

C. Mid-Point Free-Flow Speed Determination

The research team performed a sensitivity analysis to determine an approximate minimum value for free-flow speed conditions. It was assumed that by the corridor mid-point a vehicle should be able to achieve free-flow speed. This analysis investigated several potential guidelines to identify trips not at free-flow speed at the corridor mid point:

1. Speed Limit Minus 10 mph;
2. 70% of Mean Speed of Trips Unique to Each Driver at the Corridor Midpoint;
3. 75% of Mean Speed of Trips Unique to Each Driver at the Corridor Midpoint;
4. 70% of Speed Limit;
5. 75 % of Speed Limit; and
6. Lower value of Option 2 (70% of Mean Speed) or Option 4 (70% of Speed Limit).

Though all trips are depicted in the speed profile plots, at several sites many of the trips were unique to one driver. As a result the analysis used the average speed per driver to estimate the mean speed and standard deviation for Options 2, 3, and 6.

Table 10 shows the sample sensitivity results for one corridor and the results of Options 1, 2, 4, and 6. The 75% thresholds were also subjected to a similar sensitivity analysis. As a result of this evaluation, the research team selected Option 6 and identified any trips below this value at the corridor mid-point as non-free-flow.

Table 10. Free-Flow Speed Filter Sensitivity Analysis

	Corridor ID	63_N	63_S	01_E	01_W	82_N	82_S	69_E	69_W
Attributes	Speed Limit	30	30	35	35	40	40	45	45
	Functional Class	17	17	16	16	16	16	19	19
	No. of Trips	86	42	318	302	90	118	74	116
	No. of Drivers	11	10	41	45	25	26	12	12
	Mean Speed	35.95	35.27	37.71	37.85	41.79	43.09	51.06	54.31
Criteria	Speed Limit-10	20	20	25	25	30	30	35	35
	70% Mean Speed	25.17	24.69	26.39	26.50	29.26	30.16	35.74	38.02
	70% Speed Limit	21	21	24.5	24.5	28	28	31.5	31.5
	75% Speed Limit	22.5	22.5	26.25	26.25	30	30	33.75	33.75
	Min of 70% Speeds	21	21	24.5	24.5	28	28	31.5	31.5
Filter Results	Speed Limit-10	76	40	276	280	84	108	70	109
	% Loss	11.63%	4.76%	13.21%	7.28%	6.67%	8.47%	5.41%	6.03%
	% Remaining	88.37%	95.25%	86.79%	92.72%	93.33%	94.59%	94.59%	93.97%
	70% Mean Speed	70	40	277	275	84	108	69	112
	% Loss	18.60%	4.76%	12.89%	8.94%	6.67%	8.47%	6.76%	3.45%
	% Remaining	81.40%	95.24%	87.11%	91.06%	93.33%	91.53%	93.24%	96.55%
	70% Speed Limit	74	40	275	277	83	108	71	115
	% Loss	13.95%	4.76%	13.52%	8.28%	7.78%	8.47%	4.05%	0.86%
	% Remaining	86.05%	95.24%	86.48%	91.72%	92.22%	91.53%	95.95%	99.14%
	Min of 70% Speeds	74	40	275	277	83	108	71	115
	% Loss	13.95%	4.76%	13.52%	8.28%	7.78%	8.47%	4.05%	0.86%
	% Remaining	86.05%	95.24%	86.48%	91.72%	92.22%	91.53%	95.95%	99.14%

D. Lower Bound Free-Flow Speed Filter

The data processing applied the threshold identified in Section C (lower of 70% of the mean speed or 70% of the speed limit) to each corridor (excluding the acceleration and deceleration regions) using the same pattern recognition process as described in Section B. Approximately 9 percent of the trips remaining after the Section B filter were detected and removed by this Section D filter. Figure 28 demonstrates the results for one corridor location using the lower bound free-flow speed filter.

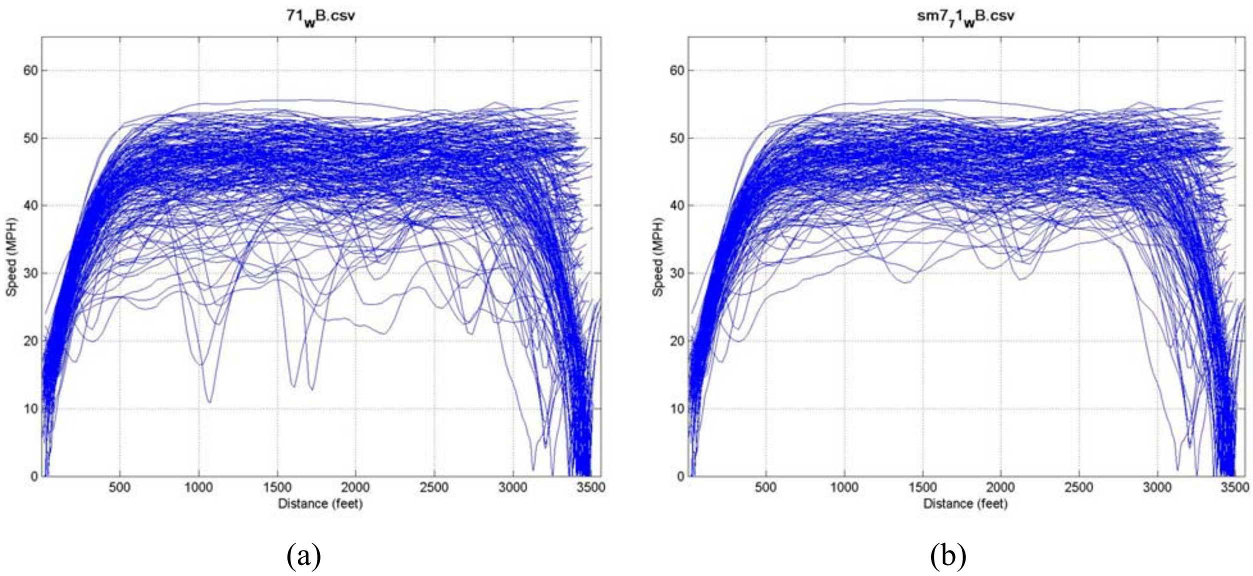


Figure 28. Speeds (a) Before and (b) After Lower Bound Free-Flow Speed Filter

4.1.11 Step 11 -- Removing data points in the acceleration and deceleration zones

To obtain free-flow speed conditions, the research team further determined the zones containing acceleration or deceleration effects from the traffic control at the two ends of corridors. This step trims the instrumented vehicle trajectories, removing the data points within the acceleration and deceleration zones. To implement this filter it is necessary to determine the deceleration and acceleration zone distances.

A. Deceleration zone length determination

Definitions:

- The deceleration zone extends from the beginning point of deceleration activity (as defined below) to the downstream intersection.
- Ninety percent of the vehicles are assumed to begin to decelerate due to traffic control within the deceleration zone (e.g., stop sign, traffic signal).
- A vehicle is considered to have begun decelerating at the point nearest the downstream intersection where its measured deceleration is greater than 1 mph/sec. Utilizing a 1 mph/sec deceleration cutoff allows for fluctuation in driving behavior during free-flow conditions as well as possible GPS receiver error.
- The deceleration zone is assumed to begin after the corridor midpoint.

Algorithm:

Step A1: Identify the corridor midpoint.

Step A2: For an individual vehicle trip identify the location the vehicle speed first drops below 10 mph downstream of the corridor midpoint, see Figure 29.

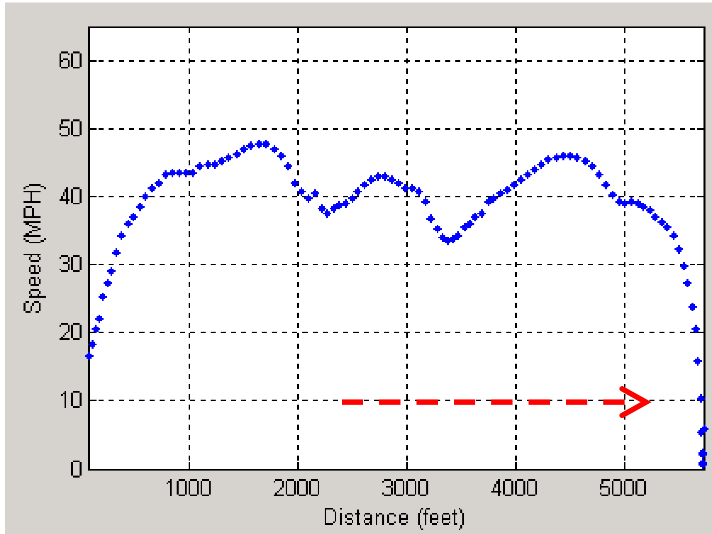


Figure 29. Identify Location Speed First Drops Below 10 mph

Step A3: From the location identified in Step A2 (define as data point i) check each upstream data point sequentially (i.e., data points i-1, i-2, i-3 and so on) and identify the closest data point to point i with a deceleration less than 1 mph/sec, see Figure 30. Locate the Closest Deceleration Less Than 1 mph/sec This point is identified as the deceleration starting location due to the downstream traffic control for the subject tri p, see Figure 31.

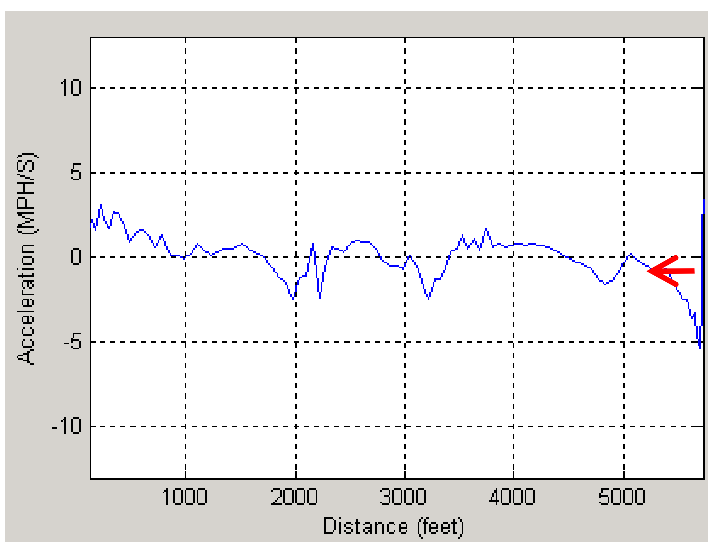


Figure 30. Locate the Closest Deceleration Less Than 1 mph/sec

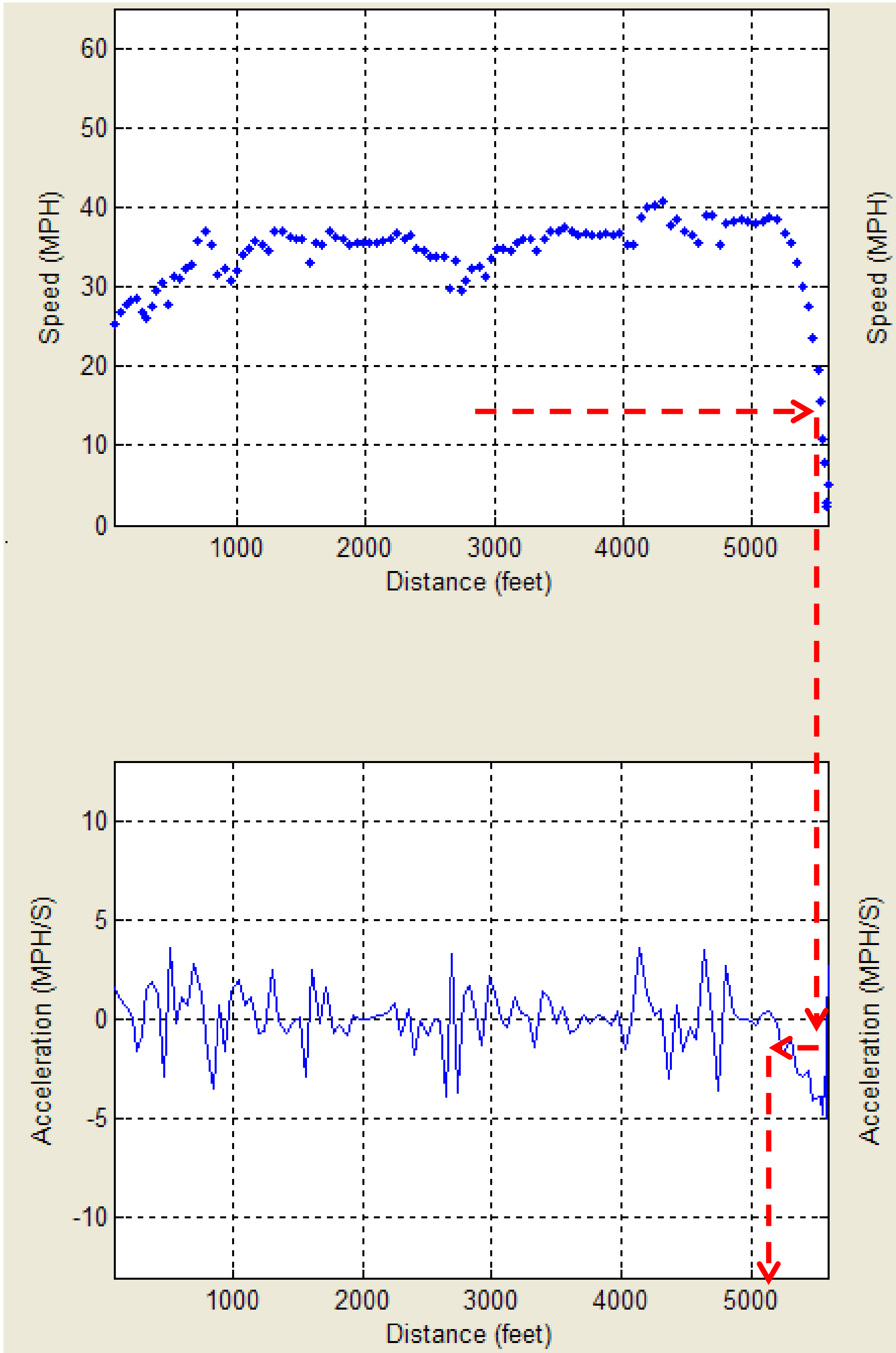


Figure 31. Determine Individual Trip Deceleration Start Location

Step A4: Repeat Steps A2 and A3 for each trip on the corridor and create a list containing the deceleration zone starting locations for each trip.

Step A5: Sort the list of starting locations with furthest upstream point listed first. Finally, identify the starting location of the deceleration zone for a corridor as the 90th percentile upstream location from the list of trip deceleration starting locations. See Figure 32.

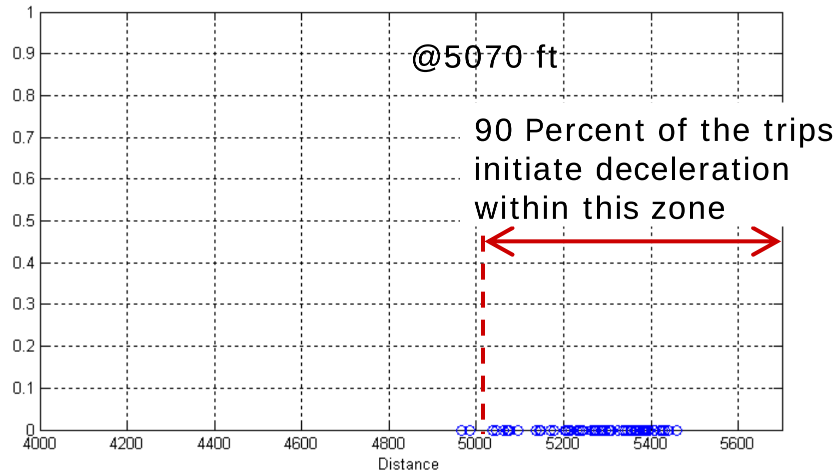


Figure 32. Example Plots of Deceleration Points for a Corridor's Trips

B. Acceleration zone determination

Definitions:

- The acceleration zone extends from the start of the corridor to the end point of the acceleration activity (as defined below).
- Ninety percent of the vehicles are assumed to have ceased acceleration due to traffic control at the corridor boundary and reached their desired cruising speed by the end of the acceleration zone. Speeds within the acceleration zone are influenced by traffic control at the upstream intersection, thus, vehicles are not considered to be operating under free-flow conditions.
- A vehicle is considered to have reached its cruising speed when the acceleration rate first drops below 1 mph/sec after the vehicle has reached a predetermined minimum speed (discussed below in Step B2). As with the deceleration zone determination, the 1 mph/sec acceleration cutoff allows for fluctuation in driving behavior during free-flow conditions as well as possible GPS receiver error.

Algorithm:

Step B1: Identify the corridor midpoint.

Step B2: For an individual vehicle trip identify the speed data point closest to the corridor midpoint where the vehicle speed is less than the lower bound speed line. For this effort the lower bound speed line is defined as the minimum of the speed limit minus 10 mph or 25 mph. For example, if the speed limit is 30 mph, the lower bound speed line is 20 mph, if the speed limit is 45 mph, the lower bound speed line is 25 mph (see Figure 33).

Aside: the lower bound speed lines between the acceleration and deceleration zones are based on different criteria as a result of the different traffic characteristics in each zone. In the deceleration zone vehicles are likely to slow to speeds below 10 mph within the corridor boundary. However, vehicles often begin their acceleration from a location upstream of the corridor boundary, e.g., several cars back in a queue at a signalized intersection or starting from the stop bar on the upstream side of the corridor boundary intersection. Therefore, many vehicles undergoing "start-up" acceleration may have already obtained a speed greater than 10 mph prior to entering the corridor.

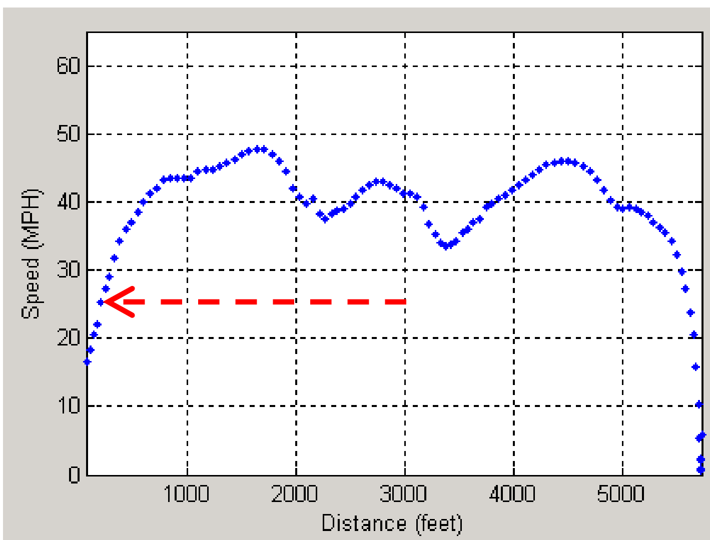


Figure 33. Determine the Speed Data Point Closest to the Corridor Midpoint Below the Lower Bound Speed Threshold

(Corridor ID 35 NB, lower bound line = 25 mph)

Step B3: From the location identified in Step B2 (define as data point i) check each downstream data point sequentially (i.e., data point i+1, i+2, i+3...) and identify the closest data point to point i with an acceleration rate below 1 mph. See Figure 34.

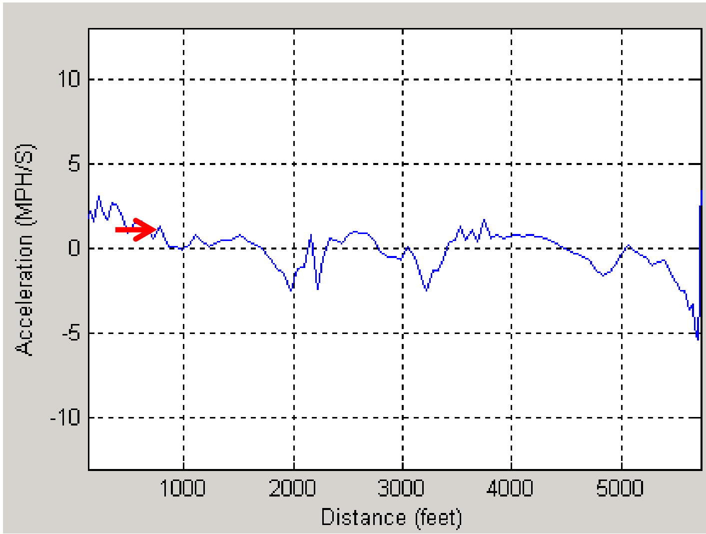
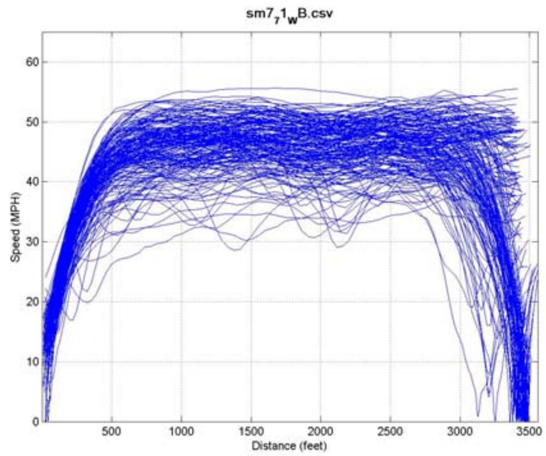


Figure 34. Locate Acceleration Rate Below 1 mph

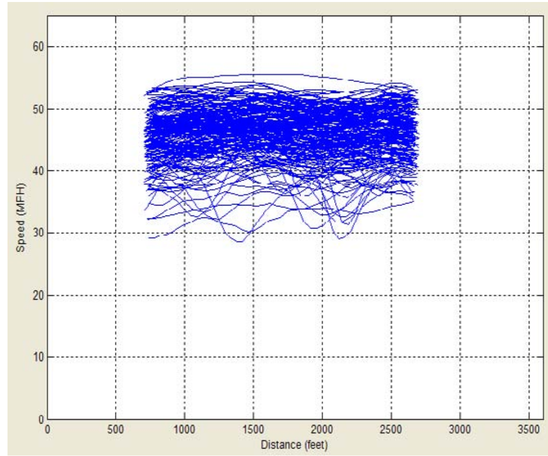
Step B4: Repeat Steps B2 and B3 for each trip on the corridor and create a list containing the acceleration zone ending locations for each trip.

Step B5: Sort the list of ending point locations with furthest upstream point listed first. Finally, identify the ending location of the acceleration zone for a corridor as the 90th percentile downstream location from the list of trip acceleration ending point locations.

Once the acceleration and deceleration zones are determined for a corridor, all trip data points within those zones are removed. Figure 35. Speeds (a) Before and (b) After Acceleration/Deceleration Filter compares speed profiles before and after the identification and removal of speed values located in the acceleration and deceleration zones.



(a)

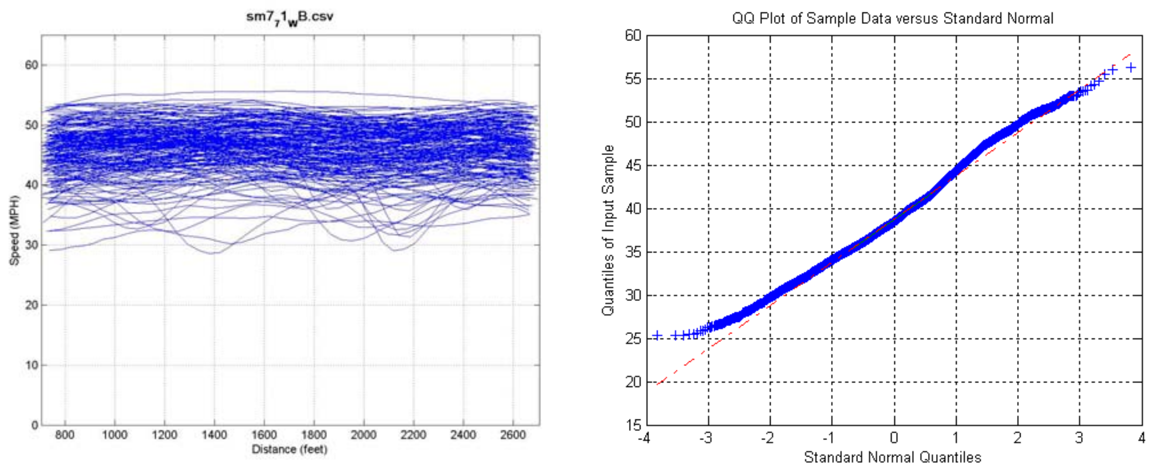


(b)

Figure 35. Speeds (a) Before and (b) After Acceleration/Deceleration Filter

4.1.12 Step 12 -- Remove highly deviated trips

After applying the procedures developed in Steps 1 through 11, the research team observed that a small portion of the remaining trips had high speed variations likely unrelated to corridor design. This variation is likely the result of traffic friction related to other vehicles on the roadway, implying non-free-flow conditions and that such trips should not be included in the analysis. Therefore, the research team developed a lower bound speed criteria to remove trips with high speed deviations relative to other trips on the corridor. Quantile-Quantile plots (Q-Q plots) were utilized to graphically compare the speed data distribution to a normal distribution. Based on the Q-Q plots, the majority of the corridors were characterized by a similar pattern of the speed data, where speed data began to deviate from normality at approximately minus two standard deviations from the mean. The Q-Q plot depicted in Figure 36b shows this speed data deviation from normality beginning at approximately the two standard deviations lower threshold. Figure 37 demonstrates the results following the application of this filter. Approximately 11 percent of the remaining trips were detected as highly deviated trips and removed by this filter.



(a) Speed profile (b) Q-Q Plot

Figure 36. Vehicle Trajectory Speeds and Quantile-Quantile Plot of WB Corridor No. 21

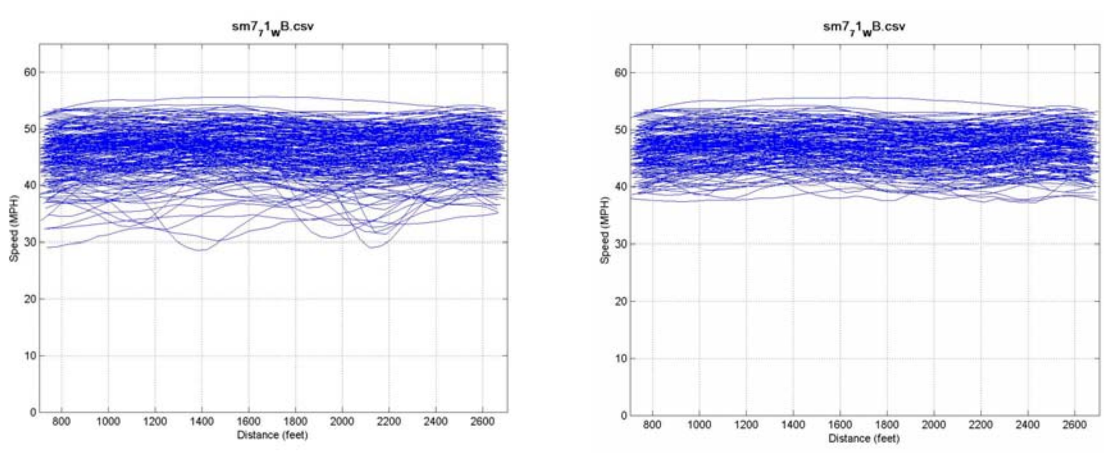


Figure 37. Speeds (a) Before and (b) After Applying the Highly Deviated Trips Filter

4.1.13 Step 13 -- Check Quality of GPS signal

The quality of GPS data in an urban environment can vary based on topography and the built infrastructure. The criteria for number of satellites (SAT) and Position Dilution of Precision (PDOP) is based on the acceptable data accuracy, data availability, and other characteristics of the GPS data utilized for this study. PDOP is an indicator of the reliability of the GPS data. In this study, acceptable quality GPS data is defined as data with a minimum SAT of 4 and PDOP value between 1 and 8. Additionally, the minimum percentage of acceptable quality data for each trip was set to 80%, meaning that if more than 80 percent of data points from one trip passed the GPS signal criteria, this trip was included. Note that 22 percent of the trips remaining after the previous step are removed by this filter. The majority of these removed data points are due to malfunctioning GPS antennas or installation issues, resulting in the data being invalid for GPS based analysis. As such, this data should not even be included in initial data analysis. Future versions of this data processing procedure will remove these trips as part of a data pre-screening, eliminating these points from even first level analysis (i.e., corridor selection, corridor ranking, etc.). The future objective of this step will be limited to capturing data quality issue where a vehicle is normally collecting acceptable data and then loses GPS lock do to operating downtown or other adverse conditions.

Table 11 Summary Statistics for Speed Observations and Number of Drivers, summarizes the number of speed observations and number of drivers for each corridor. Over all corridors the number of speed observation per driver ranged from 3 to 96 and the number of drivers ranged from 3 to 71.

Table 11 Summary Statistics for Speed Observations and Number of Drivers

COR	# of veh	Range obs. per veh	Avg obs. per veh	Total trips	COR	# of veh	Range obs. per veh	Avg obs. per veh	Total trips	COR	# of veh	Range obs. per veh	Avg obs. per veh	Total trips
00_NB	19	27 to 44	37	94	28_EB	37	14 to 21	18	118	71_WB	11	25 to 35	30	164
00_SB	15	31 to 47	41	45	28_WB	26	17 to 27	21	72	72_NB	14	30 to 49	39	43
01_EB	31	18 to 30	23	138	29_EB	35	11 to 17	14	180	72_SB	10	21 to 35	28	31
01_WB	32	21 to 37	30	132	29_WB	36	9 to 14	11	210	73_EB	16	15 to 36	30	61
02_NB	42	17 to 29	23	153	30_NB	23	23 to 44	36	242	73_WB	22	22 to 41	33	103
02_SB	48	15 to 26	21	188	30_SB	18	31 to 46	38	122	74_EB	9	63 to 96	80	15
03_EB	44	12 to 24	19	343	31_EB	27	6 to 11	8	77	74_WB	7	63 to 93	82	15
03_WB	54	14 to 25	20	483	31_WB	22	7 to 11	9	64	78_WB	8	27 to 43	38	8
04_EB	42	18 to 35	29	221	32_NB	19	16 to 28	23	144	79_WB	3	20 to 27	23	20
04_WB	46	23 to 40	32	188	32_SB	21	20 to 32	26	142	80_EB	7	30 to 55	42	44
05_SB	30	3 to 6	5	61	33_NB	17	21 to 37	31	38	80_WB	9	28 to 53	41	49
07_NB	27	4 to 8	6	149	33_SB	22	20 to 35	28	54	81_NB	10	35 to 62	51	41
07_SB	24	5 to 10	8	191	34_EB	21	9 to 16	13	43	81_SB	14	36 to 62	50	47
08_EB	26	5 to 10	8	409	34_WB	32	8 to 13	10	95	82_NB	21	14 to 23	19	73
08_WB	25	4 to 8	6	237	35_NB	43	51 to 93	73	118	82_SB	18	10 to 17	14	82
10_NB	46	9 to 15	12	199	35_SB	22	53 to 83	68	129	83_WB	12	17 to 22	19	16
10_SB	41	9 to 16	12	219	36_EB	26	37 to 60	50	163	84_EB	15	28 to 43	37	83
12_NB	32	37 to 68	56	286	36_WB	38	32 to 59	47	171	84_WB	22	29 to 52	45	122
14_EB	49	14 to 25	19	150	38_NB	31	22 to 45	35	115	85_EB	17	25 to 43	35	83
14_WB	46	14 to 28	21	206	38_SB	16	21 to 41	33	61	85_WB	13	25 to 45	35	154
15_EB	33	37 to 63	48	71	39_EB	24	38 to 64	50	46	86_NB	32	17 to 29	24	85
15_WB	18	37 to 53	45	39	39_WB	17	37 to 55	46	22	86_SB	43	16 to 28	22	139
16_NB	20	6 to 11	9	99	40_EB	21	33 to 51	41	67	87_NB	10	43 to 63	54	130
16_SB	26	7 to 14	11	99	40_WB	20	31 to 48	38	49	87_SB	11	41 to 61	55	89
17_NB	30	12 to 21	17	109	41_NB	32	16 to 28	22	101	89_NB	18	25 to 47	36	142
17_SB	30	15 to 26	21	221	41_SB	26	11 to 21	15	74	89_SB	23	24 to 42	33	214
18_NB	69	8 to 15	11	338	42_NB	34	39 to 63	56	170	90_NB	23	31 to 46	38	65
18_SB	71	7 to 15	11	363	42_SB	29	37 to 62	53	89	90_SB	20	35 to 59	48	62
19_EB	26	36 to 59	50	58	51_NB	24	32 to 47	40	54	92_NB	17	38 to 65	53	71
19_WB	18	33 to 52	43	60	51_SB	23	31 to 48	42	56	92_SB	12	30 to 45	38	42
20_EB	48	14 to 23	19	228	52_NB	20	12 to 17	14	44	93_NB	7	41 to 58	46	23
20_WB	39	11 to 18	15	137	52_SB	28	13 to 20	17	52	93_SB	8	41 to 57	47	32
21_EB	31	27 to 46	36	227	55_NB	14	32 to 50	40	45	94_NB	17	26 to 41	34	82
21_WB	30	28 to 48	38	135	55_SB	13	32 to 47	42	54	94_SB	21	24 to 46	36	160
22_EB	21	64 to 98	76	62	58_EB	7	47 to 66	58	48	95_EB	23	22 to 29	25	32
22_WB	27	44 to 74	59	112	58_WB	6	49 to 65	56	35	95_WB	19	20 to 28	23	57
23_NB	24	42 to 74	58	63	59_WB	10	15 to 23	20	25	96_NB	5	41 to 55	45	11
23_SB	33	47 to 74	62	128	63_NB	11	27 to 50	40	64	97_EB	7	37 to 49	42	17
24_NB	32	15 to 26	21	127	67_NB	9	24 to 41	33	32	97_WB	8	37 to 50	44	11
24_SB	17	18 to 28	22	42	67_SB	7	26 to 39	32	30	98_EB	22	23 to 37	30	61
25_NB	45	10 to 17	14	156	69_EB	8	29 to 39	34	60	98_WB	26	21 to 37	29	162
25_SB	54	10 to 17	14	235	69_WB	6	25 to 38	31	93	99_EB	15	39 to 52	48	19
26_EB	40	21 to 38	31	160	71_EB	11	29 to 46	39	33	99_WB	17	39 to 54	48	21
26_WB	26	26 to 43	34	132										

4.1.14 Step 14 -- Determining statistics of drivers' speed

Several statistics such as 95th percentile, 85th percentile, median, 15th percentile, 5th percentile, and mean speed among drivers were calculated every 100 feet for later use in the modeling portion of this project. Each percentile speed is calculated using the driver speed data point located nearest the selected 100 ft. interval. The selection of 100 ft intervals for modeling was due to data management constraints (continuous speed evaluation at 1 Hz intervals was not considered feasible while 100 ft intervals maintained detailed information and offered a manageable format). As shown in the legend for Figure 38, 95pct refers to 95th percentile speed, 05pct refers to 5th percentile speed, 9505p refers to the difference between the 95th and 5th percentile speeds, 85pct refers to 85 percentile speed, 15pct refers to 15th percentile speed, 8515p refers to the difference between the 85th percentile and 15th percentile speeds, mean refers to the mean speed, and median refers to the median speed at a given location.

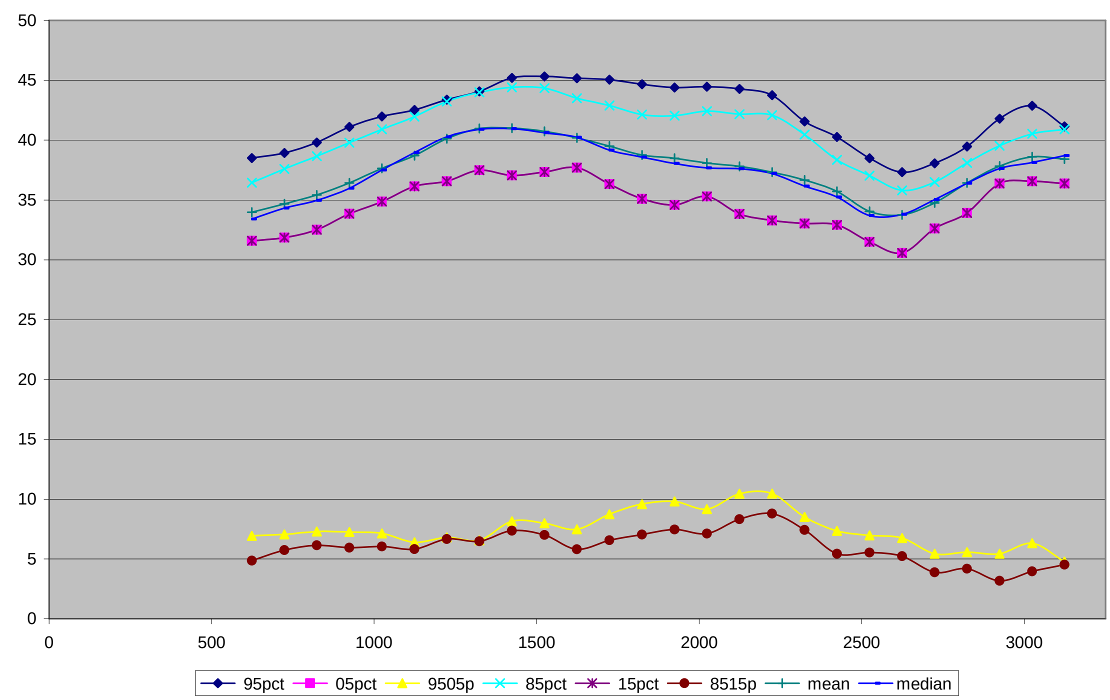


Figure 38. Statistics of Speed Profiles at one Location

4.2 Summary

Following this data processing stage, the number of candidate corridors with adequate speed data dropped to 72 locations. Approximately 66 percent of the total observed trips were removed during the Step 1 to Step 14 process due to their potential non-free-flow patterns or poor data quality. The remaining 137 directional road links located on the 72 sites are included in the statistical analysis. There are a total of 15,158 trips, made by 408 drivers, within the final selected corridors. The data includes 406,398 second-by-second instrumented vehicle data points, equivalent to 113 hours of travel. Appendix C includes a summary of the various data processing stages and their direct impact on candidate corridor speed data. Chapters five and six will next present the operating speed data analysis and operating speed models.

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5 OPERATING SPEED DATA ANALYSIS

5.1 Introduction

The driver selected speeds (desired speeds) under free-flow conditions are, in part, the reflection of the impact of the road environment on a driver's speed choice. Previous studies consider the maximum speeds along a tangent and the minimum speeds along a horizontal curve as the desired speeds since tangents do not have any geometric constraints on drivers. However, these studies often assumed that drivers reach their maximum speeds at the middle point of tangents and reach their lowest speeds at the middle point of horizontal curves because traditional data collection methods (radar gun, detector) could only measure speeds at a few specific pre-selected locations along the roadway.

This chapter will discuss the potential fault in these common assumptions of studies relying on spot speed measurements and the method used in this research to determine the driver selected speeds on tangents and horizontal curves given vehicle trajectory data. This discussion includes speed variation component analysis, data aggregation into geometry categories, and the final data arrangement.

5.2 Driver Selected Speeds on Tangents and Horizontal Curves

With the second-by-second speed profile in this study, the research team found that this assumption is not always realistic for modeling operating speeds, especially on urban streets. Drivers reach their maximum speeds at different locations along the tangents. In fact, the same driver may reach his or her maximum speeds at different locations along the same tangent for different trips. This study also found that drivers reach their minimum speeds at different locations, not just at the middle point of the horizontal curve. To simplify data reduction, the research team compiled the operating speed data for every 100-foot interval.

The speed profiles collected in this study indicate that drivers vary their cruising speeds along the corridor. Therefore, this analysis uses a variety of speed percentile statistics, including the 85^{th} and 95^{th} percentile, to estimate the driver selected speed for a given trip. This relationship is schematically depicted in Figure 39. Though the research team measured the maximum observed speed at each location (for example, several 100-foot sequential locations along a tangent), this maximum value was rarely sustained so percentile speed values appear to more accurately reflect continuous driving behavior.

The minimum segment length criterion helped assure that observed drivers reached their desired speeds on selected corridors under free-flow conditions. However, locations with horizontal curvature are more complicated because the lengths of tangent preceding the curves vary and may have a direct influence on speed within the curve. Figure 40 depicts an example of an isolated horizontal curve and associated approach tangent.

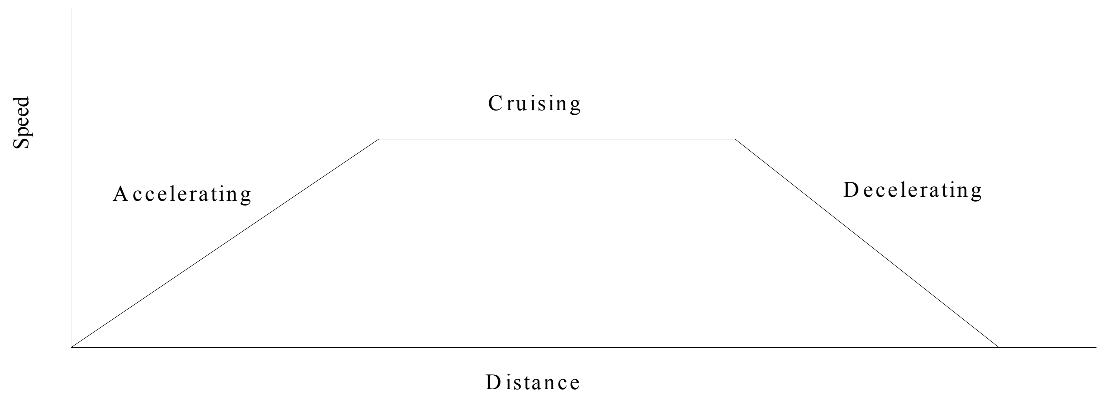


Figure 39. Speed Profile along Tangent

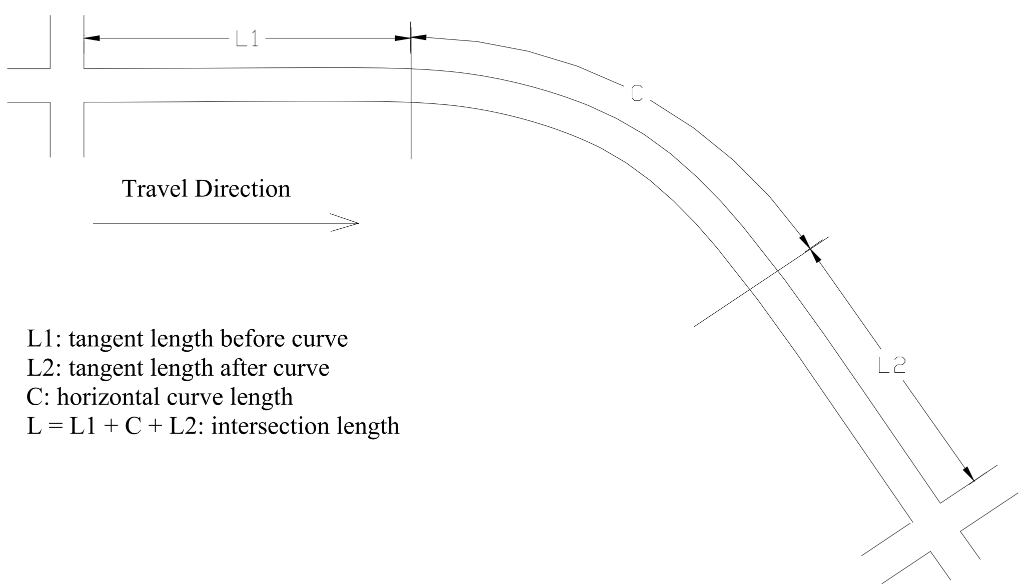


Figure 40. Horizontal Curve between Two Tangents

For a horizontal curve between two tangents, two situations can occur. If the length of tangent before the horizontal curve (L1 in Figure 40) is long enough so that drivers reach their preferred speeds in the tangent region, drivers will generally decelerate when they start traveling along the curve. In this case, this report defines cruising speeds as the speeds at which vehicles are traveling along the horizontal curve as represented by percentile statistics.

Most of previous studies used minimum speeds (at the midpoint of curve) as the driver selected speeds. However, the speed profiles collected in this study indicated that after drivers reached their minimum speeds, they tended to adjust (increase) their speeds when they were still traveling along the curves. This speed choice is consistent with the research team's assumptions for the tangent model. Figure 41 shows the speed profile under an isolated horizontal curve condition.

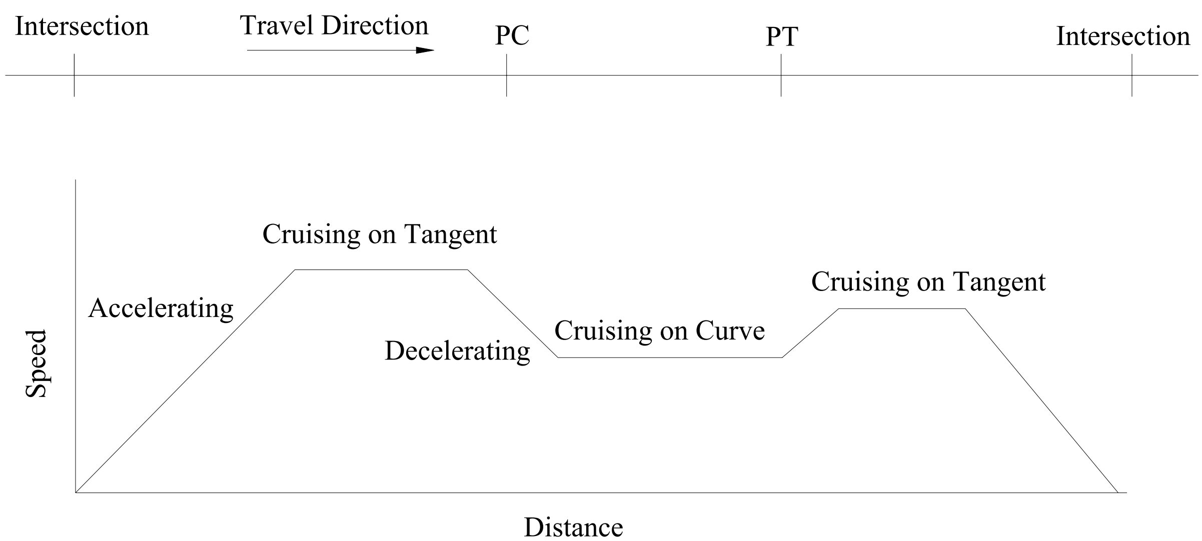


Figure 41. Speed Profile along Horizontal Curves with Long Leading Tangents

If the length of tangent before the horizontal curve is not long enough to permit drivers to achieve their preferred free-flow speed before they approach the horizontal curve, drivers continue to increase their speed along the curve. In this case, this report defines cruising speeds as the speeds at which vehicles are traveling along the horizontal curve after they reach their maximum (acceleration) speeds. Therefore, this study uses the percentile speed statistics along the curve to estimate drivers' desired speed under all curve conditions. Figure 42 depicts a schematic of the speed profile under this "short approach tangent" scenario. The data filtering process (described in Chapter 4 of this report) removed the portion of the trip on the curve where the vehicle continued to accelerate. As a result, free-flow speed analysis excluded this acceleration (or companion deceleration) phenomenon.

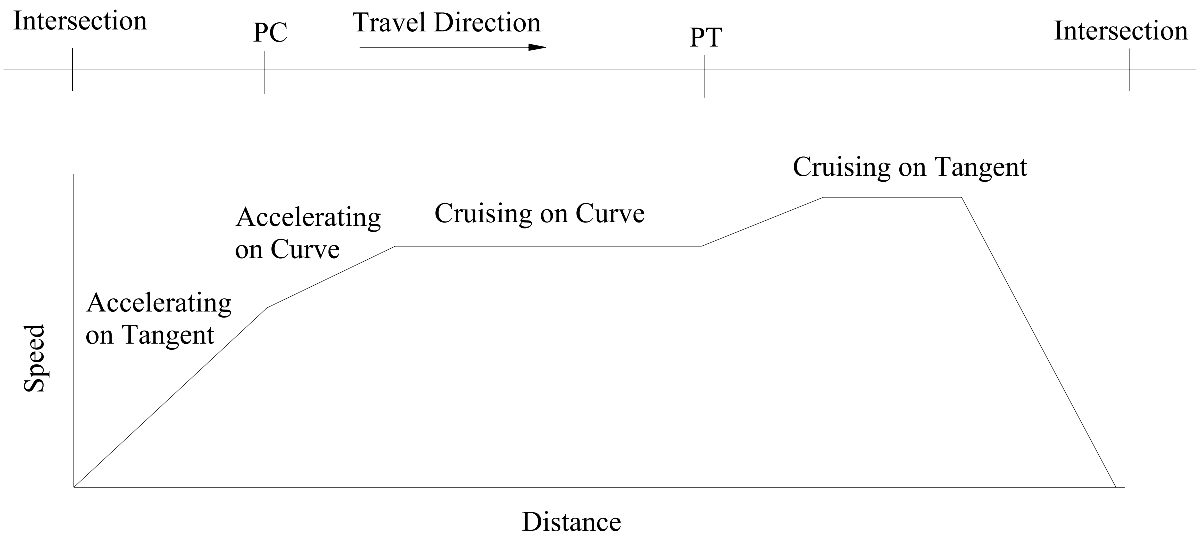


Figure 42. Speed Profile along Horizontal Curves with Short Leading Tangents

5.3 Speed Variation Components Analysis

The research team assumed that observed speed variations may be due to several potential sources including driver/vehicle characteristics, road environments, and other unknown or unobservable factors.

Most of previous operating speed models attempt to explain speed variation solely based on road feature variations. In those models, the dependent variable is operating speed while the independent variables are road features. Based on the historic spot speed data collection approach, their aggregated speed model generally represents the driving population at each site. Therefore, the variation of the driver and vehicle characteristics were removed in aggregation. The actual source of model error includes both road characteristics and unknown features as shown in equation 1.

$$\sigma^2_{speed} = \sigma^2_{road} + \sigma^2_{unknown} \qquad \text{(Eqn. 1)}$$

In this study, since the driver and vehicle information is available for each speed data point and multiple trips from the same drivers are available, the research team was able to identify the variation caused by driver and vehicle. As a result, the models developed using in-vehicle data can include the influences of drivers and vehicles into model development. In this study, the source of speed variation includes road features, driver/vehicle characteristics, and other unknown factors, as shown in equation 2.

$$\sigma^2_{speed} = \sigma^2_{driver} + \sigma^2_{road} + \sigma^2_{unknown} \quad \text{(Eqn. 2)}$$

Where:

σ^2_{speed} : speed variation,
σ^2_{driver} : driver/vehicle characteristics variation
σ^2_{road} : road feature variation, and
$\sigma^2_{unknown}$: unknown variation

5.4 Speed Data Aggregation

For each trip along a tangent or horizontal curve corridor, the research team compiled statistics that included the mean speed, 5^{th} and 15^{th} percentile speed, 85^{th} and 95^{th} percentile speed, maximum speed, and the minimum free-flow speeds. This study uses the 85^{th} (V85) and 95^{th} (V95) percentile speed to estimate the upper bounds of driver selected speeds (desired speeds) along tangents. The research team similarly used the 5^{th} (V5) and 15^{th} (V15) percentile speeds to estimate the lower bounds of driver selected speeds (desired speeds) along tangents.

As speed changes occurred in the vicinity of the start or end of horizontal curves, the research team parsed the free-flow data (for analysis purposes) into three horizontal geometry categories:

- T1 is the tangent segment that is not located within 200’ of the beginning or ending of a horizontal curve;

- T2 is the speed transition zone located on a tangent segment within 200’ of the beginning or ending of a horizontal curve; and

- HZ is the horizontal curve segments or interest.

The research team developed speed models for the stable T1 and HZ conditions. The T2 regions were generally characterized by constantly changing speed and were therefore removed from the model development procedure for this project.

5.5 Study Data Layout

Table 12 presents the layout of the dataset developed for this analysis. In this dataset, each subject (driver) had different observations (trips). The road feature variables (x_{ijk}) are the same if the observations (trips) occurred at the same site.

Table 12. Longitudinal Data Layout

Subject (i)	**Observation (j)**	**Response**	**Covariates (k)**		
1	1	y_{11}	x_{111}	$\cdots$	x_{11p}
1	2	y_{12}	x_{121}	$\cdots$	x_{12p}
.	.	.	.	$\cdots$	.
1	n_1	y_{1n_1}	$x_{1n_1 1}$	$\cdots$	$x_{1n_1 p}$
.	.	.	.	.	.
.	.	.	.	.	.
.	.	.	.	.	.
N	1	y_{N1}	x_{N11}		x_{N1p}
N	2	y_{N1}	x_{N21}		x_{N2p}
.	.	.	.		.
N	n_N	y_{Nn_N}	$x_{Nn_N 1}$		$x_{Nn_N p}$

In which

i = 1, 2, …, N subjects (drivers)
j = 1, 2, …, n_i observations (trips) for subject i,
k = 1, 2, …, p road feature variables ,
y_{ij} = response (aggregated speed statistic) for subject i on observation j, and
x_{ijk} = road feature variable k for observation j from subject i.

This study initially modeled 2683 trips at tangent locations and 2049 trips at horizontal curves (for base model development purposes). For model validation, 1090 trips occurred at tangent locations and 718 trips were located at horizontal curve locations. In total, this analysis included a combined 6,540 trips for tangent and curve locations. All free-flow trips occurred during daytime conditions and within a period of one year.

6 OPERATING SPEED MODELS

6.1 Regression Techniques

Linear regression is a technique commonly used to describe a statistical relationship between a dependent variable and one or more explanatory or independent variables. The simple linear regression model has the following general form:

$$\mathbf{y} = \mathbf{X\beta} + \varepsilon \qquad \text{(Eqn. 3)}$$

Where:

$\mathbf{y}$ is the dependent variable vector,
$\mathbf{X}$ is independent variable model matrix,
$\boldsymbol{\beta}$ are the regression parameters vector, and
$\boldsymbol{\varepsilon}$ is the random error term vector [$\boldsymbol{\varepsilon} \sim N(0, \sigma^2 \mathbf{I}_n)$]r

Ordinary linear regression assumes the error terms are not correlated (independent). That is, the outcome of one observation has no effect on the error term of any other observations. This assumption results in response variables that are then not assumed to be correlated (68).

An analysis of variance partitions the total sum of squares (SSTO) into the Regression Sum of Squares (SSR) and Error Sum of Squares (SSE) as follows:

$$SSTO = SSR + SSE \qquad \text{(Eqn.4)}$$

Where:
SSTO = total sum of squared deviation from the mean,
SSR = deviation of the fitted regression value from the mean, and
SSE = deviation of the fitted regression value from the observed value.

The coefficient of determination (R^2) represents the proportion of total variation explained by the predictor variables, as showed in Equation 5. The larger the R^2, the larger proportion of the total variation is explained by the predictor variables.

$$R^2 = SSR/SSTO = 1 - SSE/SSTO \qquad \text{(Eqn. 5)}$$

Many previous studies have employed this statistical approach to predict drivers' speed choices based on physical conditions such as roadway geometry and roadside features. The 85^{th} percentile speed is the general statistic used to describe operating speed when assessing the influence of the road environment on speed selection.

However, normal linear regression methods are not appropriate for this modeling effort because speed data from the same driver at different sites are likely to be correlated. The dependent variables (y_i) are not independent to each other. Therefore, the independence assumption of normal linear regression is violated. This study uses a linear mixed effects (fixed-effects and random-effects) model, which is an extension of the ordinary linear regression model. Linear mixed effects models add another random variable to reflect the influence from each individual subject so that a model will permit within-subject correlations and accounts for the influence of both fixed and random-effects in explaining the response variable (speed). The specific components of the linear mixed effects model include:

- Fixed effects: factor levels in the sample are all levels to which reference will be made (e.g., street environment features); and

- Random effects: factor levels represent a random sample from the population (e.g. drivers).

The linear mixed effects model is represented by Equation 6 [69].

$$\mathbf{y}_i = \mathbf{X}_i\boldsymbol{\beta} + \mathbf{Z}_i\mathbf{b}_i + \boldsymbol{\varepsilon}_i \qquad \text{(Eqn. 6)}$$

Where:

$\mathbf{y}_i$ is the response vector for response for subject i,
$\mathbf{X}_i$ is the fixed effects model matrix for subject i,
$\mathbf{Z}_i$ is the random effects model matrix for subject i,
$\mathbf{b}_i$ is the vector of random effects coefficients ($\mathbf{b_i \sim N(0,\psi)}$),
$\boldsymbol{\beta}$ is the vector of fixed effects coefficients,
$\boldsymbol{\varepsilon}_i$ is the vector of random error term ($\boldsymbol{\varepsilon}_i \sim N(0, \sigma^2\mathbf{I}_n)$), and
Ψ is the covariance matrix for the random effects.

The fixed effects are applicable if researchers are only interested in treatments observed in the study. The random effects are applicable if treatments are a random sample from a larger population of treatments, and researchers are interested in all treatment levels in the population. Since this study is interested in the entire driver population, drivers were modeled as random factors.

The models developed for this research project are based on the fact that the selected drivers were randomly drawn from the Atlanta metro area population at large (as a part of the Commute Atlanta research project). The extension of these speed models to other regions would then require the assumption that drivers in the Atlanta region are similar to drivers in other locations. The driver (subject) variable is a random effect and, in this way, the research team was able to incorporate the sampling variability and make inferences about the driver population from which the subjects were selected.

6.1.1 Random Intercept Mixed Effects Model

The random intercept mixed effects model is a simple mixed effect model with the following form:

$$y_{ij} = \beta_{0i} + \beta_1 x_{1j} + \beta_2 x_{2j} + \ldots + \beta_p x_{pj} + \varepsilon_{ij} \qquad \text{(Eqn. 7)}$$

Where:

y_{ij} is the response (speed) of subject (driver) i at site j,
β_{0i} is the intercept of subject I ($\beta_{0i} = \beta_0 + v_{0i}$),
β_0 is the mean speed across the population,
v_{0i} is a random variable that represents the deviation from the mean speed for subject I ($v_{0i} \sim N(0, \sigma^2_v)$) or the influence of driver/vehicle characteristics on his/her speeds,
β_i is the coefficient for road feature variable i,
x_{ij} is road features variable,
ε_{ij} is the random error for subject i at site j ($\varepsilon_{ii} \sim N(0, \sigma^2)$),
σ^2 is within subject variance, and
σ^2_v is between subject variance.

This model indicates that the speed of driver i at site j is influenced by road features, driver characteristics, and vehicle characteristics. Each driver's initial speed (intercept) is determined by the population mean speed β_0, plus a unique contribution from that driver v_{0i}. Therefore, each driver has his or her own distinct initial speeds. The population intercepts and slope parameters (β_i) represent the overall trend while the subject parameter (v_{0i}) represents the deviation of each subject from the population trend. This model assumes that the influence of road features is the same for all drivers (the same coefficient β_i for all drivers). Figure 43 represents this model graphically with only one independent variable (lane width). In this figure, *Driver j* is driving more aggressively than *Driver i*.

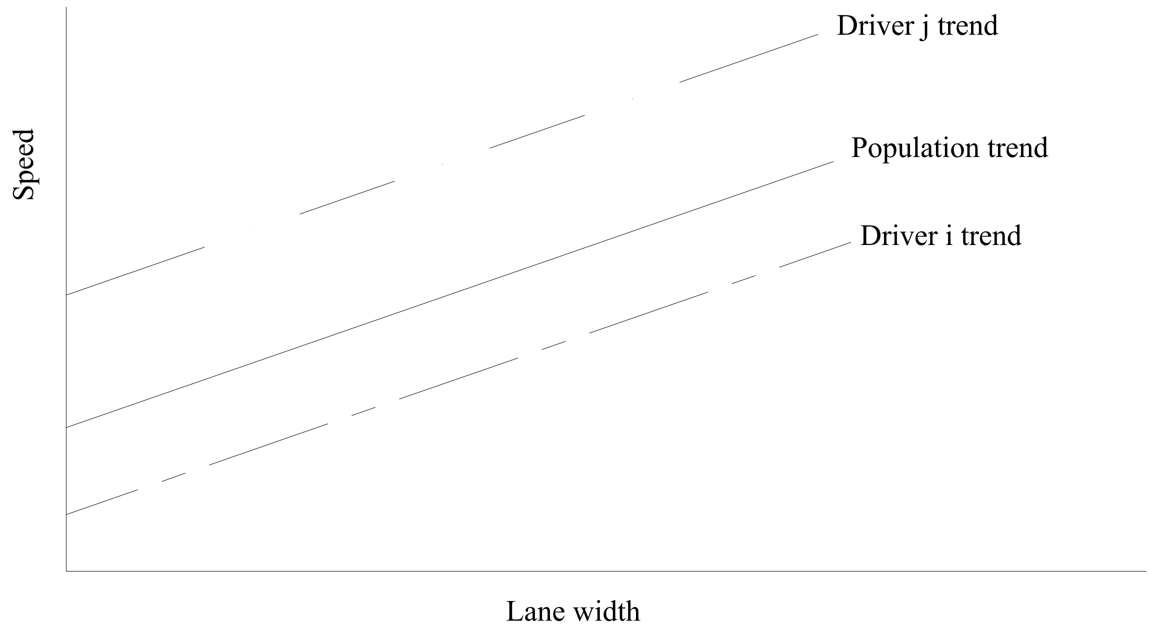


Figure 43. Random-Intercept Mixed Effects Model

The between-subject variance σ^2_v measures the variability of speeds from different drivers at the same site. The greater variability observed for the different drivers' mean speeds at the same site, the greater the σ^2_v. If all drivers traveled at the same speeds at the same site, the between-subject variance (σ^2_v) will be zero. The within-subject variance σ^2 measures the variability of speeds from the same drivers. The greater speed variability observed from different trips from the same driver, the greater the σ^2.

The random intercept mixed effects model is represented as a linear regression model with a random intercept. In this model, researchers are interested in estimating the coefficient of fixed effects (β_i) and testing hypothesis about the variance of random effects (σ^2_v).

The intra-class correlation (ICC) is a proportion representing the unexplained variance that is attributed to an individual subject. If ICC is near zero, differences in the mean speeds among different drivers for the same site conditions are not significant. On the other hand, if ICC is large, much of the total variance is caused by the differences among different drivers.

$$\mathrm{ICC} = \frac{\sigma_v^2}{\sigma_v^2 + \sigma^2} \qquad \text{(Eqn. 8)}$$

6.1.2 Model Estimation

Maximum likelihood (ML) estimation is used in linear mixed effects models for estimating parameters. In the ordinary least squares regression, the objective in fitting a model is to estimate the parameters that minimize the sum of squared errors of predictions. In maximum likelihood, the objective in fitting a model is to estimate the parameters that make the observed data (y_{ij}) most likely to have occurred, in other words, maximize the likelihood (L) of observing the sample values. Generally, it is easier to work with the log of the likelihood function (log-likelihood). The maximum value of L can be derived by finding the point at which log-likelihood has a slope of zero [70].

Assuming a normal regression model represented by $y_i \sim N(\beta_0 + \beta_1 x_i, \sigma^2)$, the probability density function in Equation 9 represents the likelihood (probability) of y_i given the mean ($\beta_0 + \beta_1 x_i$) and variance (σ^2).

$$p(y_i|\beta_0, \beta_1, \sigma^2) = \frac{1}{\sqrt{2\pi\sigma^2}}\exp\left\{-\frac{1}{2\sigma^2}\left(y_i - (\beta_0 + \beta_1 x_i)\right)^2\right\} \qquad \text{(Eqn. 9)}$$

The likelihood is equal to

$$L(\beta_0, \beta_1, \sigma^2) = \left(\frac{1}{\sqrt{2\pi\sigma^2}}\right)^n \exp\left\{-\frac{1}{2\sigma^2}\sum_{i=1}^{n}\left(y_i - (\beta_0 + \beta_1 x_i)\right)^2\right\} \qquad \text{(Eqn. 10)}$$

The log-likelihood is equal to

$$\begin{aligned}\mathrm{LogL}(\beta_0, \beta_1, \sigma^2) &= -\frac{n}{2}\log 2\pi - \frac{n}{2}\log\sigma^2 - \frac{1}{2\sigma^2}\sum_{i=1}^{n}\left(y_i - (\beta_0 + \beta_1 x_i)\right)^2 \quad \text{(Eqn. 11)}\\ &= -\frac{n}{2}\log 2\pi - \frac{n}{2}\log\sigma^2 - \frac{1}{2\sigma^2}SSE\end{aligned}$$

In this case, minimizing SSE is equivalent to maximizing the log-likelihood.

$$\hat{\beta}_0 = \overline{Y} - \hat{\beta}_1(x) \qquad \text{(Eqn. 12)}$$

$$\hat{\beta}_1 = \frac{\sum_{i=1}^{n} Y_i(x_i - \overline{x})}{\sum_{i=1}^{n}\left(x_i - \overline{x}\right)^2} \qquad \text{(Eqn. 13)}$$

$$\hat{\sigma}^2 = \frac{1}{n}\sum_{i=1}^{n}\left(Y_i - \left(\hat{\beta}_0 + \hat{\beta}_1 x_i\right)\right)^2 \qquad \text{(Eqn. 14)}$$

A model with large log-likelihood is preferred over one with a small log-likelihood. However, a model with more parameters normally has a larger log-likelihood than a model with fewer parameters. Therefore, The Akaike Information Criterion (AIC) [71] and Bayesian Information Criterion (BIC) [72] are used to compare models with the correction of the number of parameters. Normally, smaller AIC and BIC values indicate better models.

$$AIC = - 2 \text{ log-likelihood} + 2n \qquad \text{(Eqn. 15)}$$

$$BIC = - 2 \text{ log-likelihood} + n \log(N) \qquad \text{(Eqn. 16)}$$

Where
n=number of covariance parameters, and
N=number of observations.

Restricted maximum likelihood estimation (REML) has the same merits as ML but has the advantage of taking into account the loss of degrees of freedom involved in estimating the fixed effects [68]. For example, the REML estimator of the error variance in the simple balanced one-way ANOVA model is SSE/(n – p), where SSE is the within-group sum of squares, n is the sample size and p is the number of the fixed effect parameters. In contrast, the ML estimator is SSE/n.

6.2 Operating Speed Candidate Variables

In this study, the predicting dependent variable is the driver selected speed at candidate corridors. The research team compiled a large number of candidate site variables including roadside objects, access density, cross-section features, grade, and land use. Table 13 presents several of the candidate (independent) variables evaluated for this study. Variables that require additional explanation are summarized in the following sections.

6.2.1 Roadside Environment Rating

For the purposes of this study the data collection team ranked the roadside environment as one of four pictorial categories. Category one represents a roadside condition relatively free of fixed objects while category four represents a roadside condition characterized by dense roadside conditions, non-traversable slope, or roadside barrier protection.

Table 13. Description of Independent Variables

Variables	***Description***
RR	roadside environment rating (see discussion in text)
Dwy*	driveway density (number of driveways per mile)
Int**	Intersection density (number of intersections per mile)
Grade	Refer to Table 14
u/d	-1: negative vertical grade where $g < -4\%$ 0: $-4\% \leq g \leq +4\%$ 1: positive vertical grade where $g > \pm 4\%$
Lanewidth	lane width (ft) where lanes wider than 12' are treated as having widths of 12'
Lanenum	number of lanes in one direction of travel
Sd	sight distance 0: <100' 1: 100' to 150' 2: 150' to 200' 3: 200' to 280' 4: 280' to 360' 5: 360' to 460' 6: > 460'
Sw	0: no sidewalk 1: sidewalk
Parking	0: no on-street parking 1: on-street parking
Median	0: no median 1: median (raised or TWLT)
median_width	width of median (ft)
Curb	0: no raised curb 1: raised curb present
Landuse	0: residential, church, school, country club, golf course, forest, undeveloped, or a combination 1: commercial, industrial, office, apartments, shopping, hospital, museum, municipal building, mixed use (commercial & residential) 2: others including greater than 20% of commercial, residential, and undeveloped property
Radius	horizontal curve radius (ft) up to 1700' (curves with radii greater than 1700' are treated as tangents)
Curvedir	direction of horizontal curvature where: 0: horizontal curve to the left 1: horizontal curve to the right
speed limit	posted speed limit

Notes:

* The Pearson correlation between intersections and driveways was quite low (value of 0.16 where 1.0 represents perfect correlation and 0.0 represents no correlation), so these two items have been treated as separate variables.

** In the current analysis cross intersections and T-intersection are treated as a single variable as the existing data set includes no traffic control on the mainline and each direction of traffic flow is being modeled separately.

Roadside objects considered in this rating code included:

- Trees and bushes,
- Utility and light poles,
- Guardrail,
- Street sign, Speed limit signs (temporary signs such as Work Zone sign excluded),
- Fences or concrete walls,
- Fire hydrants,
- Mailboxes, and
- Bus stops or bus shelters.

The roadside environment rating provides a rating for the field of view from the driver's perspective. As the road segments are selected to be uniform the driver's perceived field of view is reasonably consistent throughout the segment. Thus, the data collection team developed the site rating based on the "overall" environment condition of the corridor. For example, if a single street sign was located three feet laterally from the travel lane while all other objects were located ten feet laterally from the travel lane, the effect from the single sign was considered to be insignificant for drivers. Figure 45 demonstrates the selection process for the appropriate roadside rating (RR) value at a particular location. Figure 45 depicts a typical roadside condition (shown from a driver's perspective) that would be assigned a value of RR=1. Similarly, Figure 46, Figure 47, and Figure 48 represent RR values of 2, 3, and 4, respectively.

6.2.2 Horizontal Curve Radius as a Variable

The study sites included a variety of horizontal curve radii ranging from a sharp curve (short radius) that could be expected to dramatically influence a driver's selected speed up to a very gentle curve (long radius) that would probably have very little influence on speed choice. To determine the threshold for inclusion of a radius value in the speed models, the research team assessed the relationship between radius and model sensitivity by testing various maximum radii in the model development. Ultimately, the research team determined that curves with a radius value greater than 1700 feet did not have a direct impact on the model speeds. Similarly, as radii increased their influence on speed decreased. Figure 44 graphically depicts this observation for the horizontal curves with one lane of travel in each direction. The trend lines represent the 30, 35, and 45 mph posted speed limits.

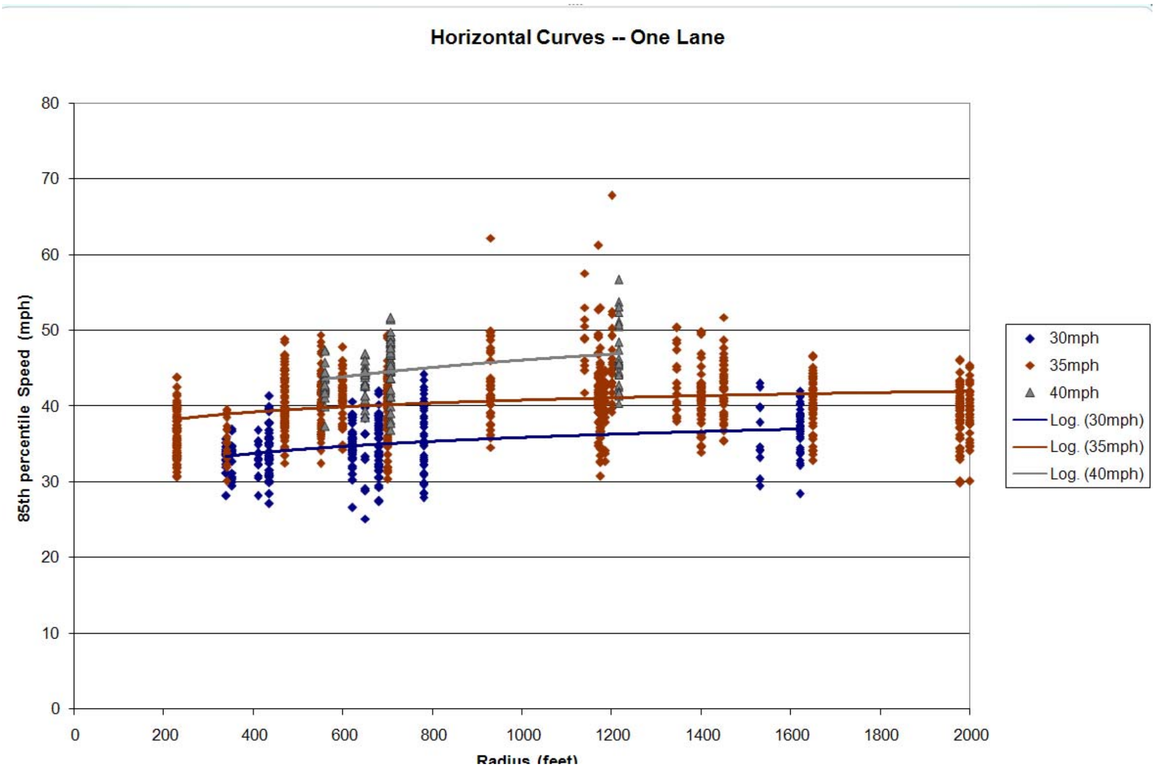


Figure 44. Speed vs. Radius for Various Speed Limit Configurations

6.2.3 Vertical Grade as a Variable

The research team encountered a challenge when attempting to include vertical geometry into the analysis models. During field data investigations, the data collection team used a smart level to periodically determine vertical geometry conditions; however, this method provided only “spot grades” and may have not adequately captured all of the various vertical geometric fluctuations. In general, if the field collection team determined that the length of the corridor was level (defined a ranging from -2% up to +2%), they did not capture specific vertical geometry. For locations where vertical conditions extended beyond these conventionally recognized level terrain conditions, the data collection team attempted to collect ample data to approximate the vertical geometry. This approach can be challenging when performing a task using engineering judgment applied to field conditions, but as-built plans were not available for the candidate corridors. Figure 50 depicts a sample vertical profile acquired during field data collection and a companion speed profile (that includes acceleration zones). This specific site did not have any horizontal curvature and speed variations appeared to correspond directly to the extreme vertical grades. Ultimately, the research team divided the collected vertical geometry data into seven general terrain/grade categories depicted in Table 14. These categories were determined based on a series of classification statistical strata analyses. In some cases, a corridor was sub-divided to accommodate extreme changes in grade as those depicted in Figure 50.

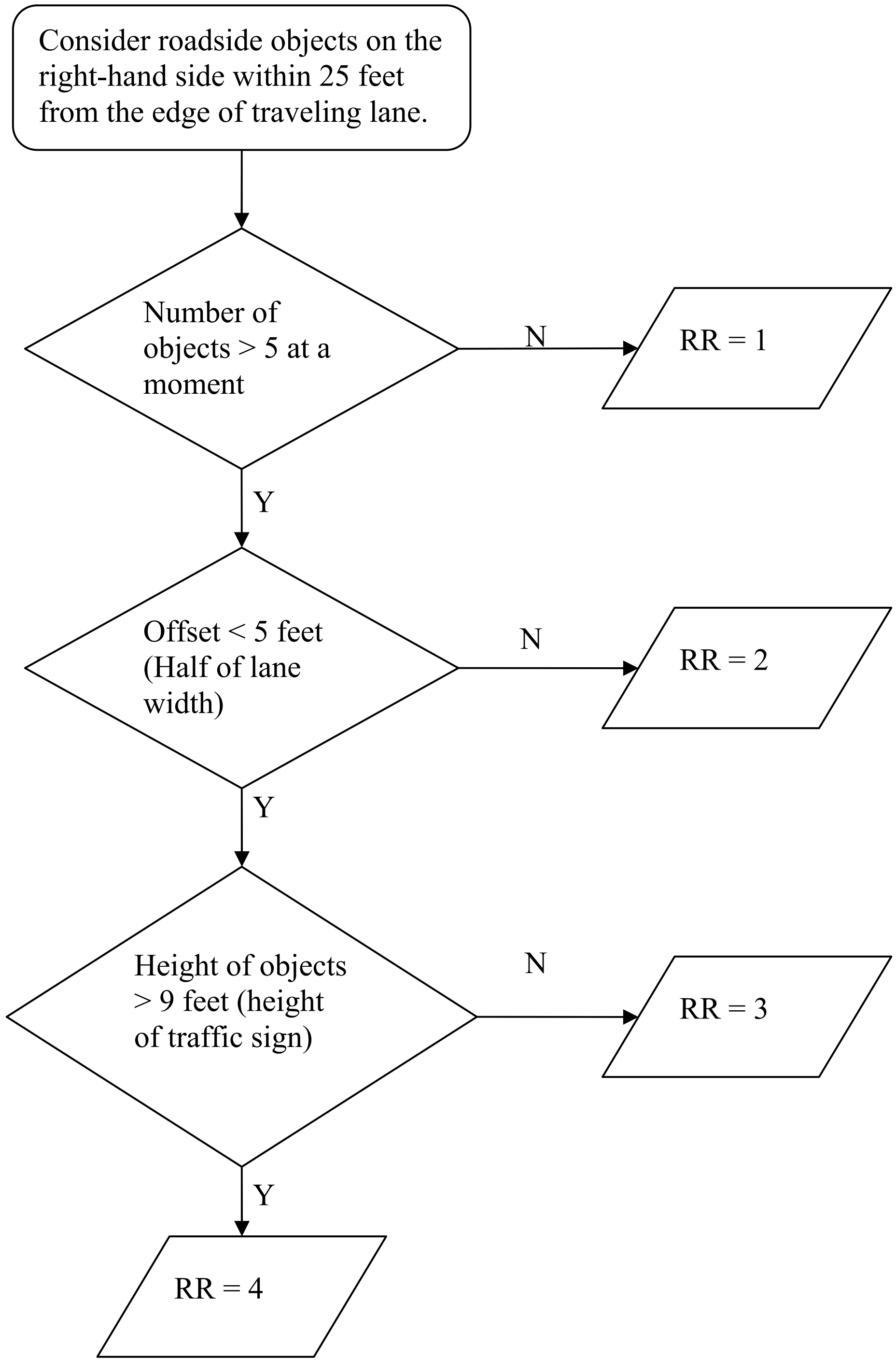


Figure 45. Flow Chart for the Roadside Rating Process

Roadside Environment Rating = 1

[Within 25 feet offset from the edge of traveling lane, roadside objects are few, i.e., less than 5 objects per snapshot.]

Figure 46. Roadside Environment Rating = 1

Roadside Environment Rating = 2

[Higher density of roadside objects than category 1, i.e. more than 5 objects at a moment. Average offset is larger than 5 feet.]

Figure 47. Roadside Environment Rating = 2

Roadside Environment Rating = 3

[Average offset is less than 5 feet. Small roadside objects, i.e. heights are less than 9 feet. Small objects include mailboxes, hydrants, guardrails, low fences, small bushes, etc.]

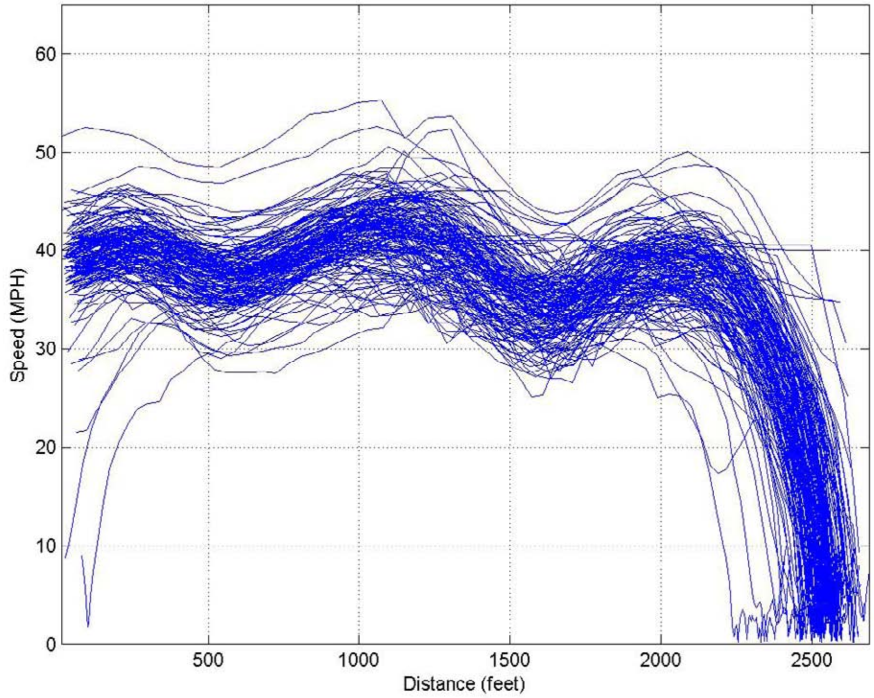
Figure 48. Roadside Environment Rating = 3

Roadside Environment Rating = 4

[Average offset is less than 5 feet. Tall objects are within 5 feet offset. Tall objects are such as utility poles, traffic signs, tall fence, and trees.]

Figure 49. Roadside Environment Rating = 4

Speed Profile



Vertical Profile

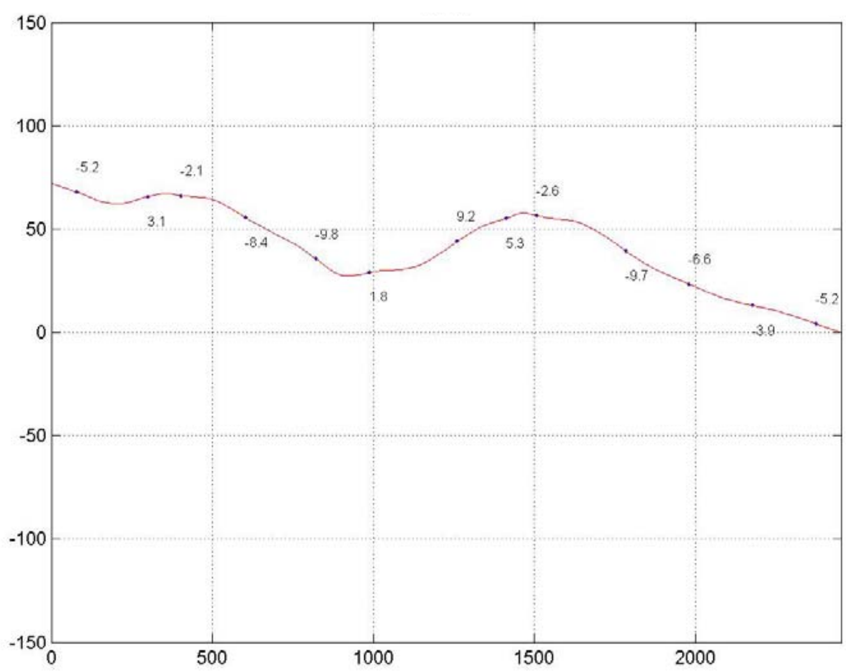


Figure 50. Example Vertical Profile and Companion Speed Profile

Table 14. Summary of Grade Variable Conditions

Terrain Indication	Description	grade variable	u/d variable
RD	Rolling / down ($g < -4\%$)	1	- 1
MD	Moderate / down ($-4\% \leq g < -2\%$)	0	0
L	Level Terrain ($-2\% \leq g \leq +2\%$)	0	0
M	Moderate / varying grade	0	0
MU	Moderate / up ($+2\% < g \leq +4\%$)	0	0
RU	Rolling / up ($g > +4\%$)	0	1
R	Rolling / varying grade	1	0

6.2.4 Land Use as a Variable

The candidate sites included a variety of land use configurations. Actual land use may not directly affect speed, but the resulting built environment may have an influence on a driver's speed choice. Though a variable for driveway density was included in the analysis, land use as represented in this modeling effort was only moderately correlated to the driveway density variable as land use included much more that driveway location. It is reasonable to assume, however, that the combination of a commercial land use and driveway density may imply heavier driveway access than that of residential land use and driveways (implying more minor, lighter volume driveways). Initially, the research team developed seven land use categories; however, during model analysis the research team used classification strata to determine if the various land use categories influenced speed behavior in significantly different ways. This land use analysis resulted in a division of land use into the three final land use categories as shown in Table 15.

Table 15. Land Use Variable Options

Land Use (LU) Description	Initial LU Code	Final LU Code
Single family homes, church, school, country club, golf course	1	0
Commercial, industrial, office, apartment, shopping, hospital, museum, municipal building	2	1
Forest, parallel railroad track, undeveloped land	3	0
Greater than 20% of categories 1 and 2	12	1
Greater than 20% of categories 1 and 3	13	0
Greater than 20% of categories 2 and 3	23	1
Greater than 20% of categories 1, 2, and 3	123	2

6.3 Model Development

Before developing statistical models for operating speeds, the research team separated the data into two sets. The sites available after completion of the speed filtering processes described in Chapter 4 were randomly separated into a model development data set (70 percent of the sites available) and a validation set (30 percent of the remaining sites). Initial model development included only the model development data sets. Following development of preliminary models, the research team then used the models resulting from this effort to estimate free-flow speeds at the validation sites. Upon comparing the results estimated using these models to actual speeds at the validation set site, the speed estimates were significantly different at several of the sites. Due to the random selection procedure used for separating the data the research team noted that land use in the validation set tended to be more residential than land use in the model development set. Though there were several variables that could influence the performance of the models, this land type is likely the source of the different results for both data sets. The models presented in this chapter, therefore, include the merged data from both the model development and the validation data sets. When using these models to estimate free-flow speeds for a jurisdiction, it would be useful to perform a field validation for local conditions to identify their fit prior to wide scale deployment.

Based on the statistical methodology previously summarized, the research team members developed random intercept mixed effects models for each independent variable to test the significance of each variable at the 95% significance level. Discussion in this section will refer to the 85% percentile analysis; however, similar analysis and companion results for other test thresholds are presented in tabular format later in this chapter. In addition, based on statistically significant variables, the model development required the separation of tangent models from horizontal curve models. Similarly, roads with one lane of travel in each direction resulted in different significant variables than those for two lanes in each direction. As a result, the model development effort was ultimately divided into the four following speed model categories:

- T1One: Tangent Road with one-lane per direction of travel;
- T1Two: Tangent Road with two-lanes per direction of travel;
- HZOne: Horizontal Curve with one-lane per direction of travel; and
- HZTwo: Horizontal Curve with two-lanes per direction of travel.

In addition, the research team performed what they referred to as a “logic analysis” by evaluating each variable to determine if its influence on the overall model was logical. Often a variable with very small influence on the overall result may require substantial data collection. The research team also evaluated each variable to determine if its inclusion in the final models was practical and justifiable. For example, due to the extensive effort to collect the sight distance variable, the report includes models with and without this unique variable. The analysis considered speed limit as a candidate variable; however, this item was strongly correlated to several of the other variables and so was ultimately excluded from continued model development.

The following detailed description refers to development of the 85th percentile model for the tangent road with one-lane per direction of travel. It should be noted that the research team withheld a subset of data for validation testing. The model information depicted in the following is the final model information following both preliminary and validation analysis.

Table 16 lists the coefficients and p-values for each independent variable for the initial tangent model (prior to the logic analysis) with one-lane per direction of travel referred to as T1One. The results in Table 16 also indicate the representative coefficients for the T1One model.

Table 16. Coefficients and P-Values for T1One Model

Variable	**Coefficient**	***P-value***
(Intercept)	45.5264	*<.0001*
Grade	-1.0266	*0.0001*
u/d	-0.9836	*<.0001*
Sd	0.6572	*<.0001*
Curb	-1.5665	*<.0001*
RR	-1.6976	*<.0001*
Landuse	-1.6208	*<.0001*
Int	-0.4539	*<.0001*
AIC	12212.3	
BIC	12219.9	
logLik	-6104.15	
σ_v	5.7179	
σ	16.8195	
ICC	*0.25*	

The resulting 85th percentile model that includes the variable "sight distance" can be written as:

$$V85 = 45.53 - [1.03 \times grade] - [0.98 \times u/d] + [0.66 \times sd] - [1.57 \times curb] - [1.70 \times RR] - [1.62 \times landuse] - [0.45 \times int] \quad \text{(Eqn. 17)}$$

Due the time consuming nature of field measuring the sight distance (in a manner consistent with procedures commonly defined for the measurement of intersection sight distance), the research team also developed an 85th percentile model that excluded sight distance as a candidate variable. Table 17 depicts this modified model and equation 18 depicts the model in equation form. The sub-caption "No SD" in equation 18 refers to the fact that this model is the 85th percentile model for the T1One model that excludes sight distance (thus SD) as a variable.

The resulting 85th percentile model that includes the variable "sight distance" can be written as:

$$V85_{\text{No SD}} = 49.97 - [1.44 \times grade] - [0.82 \times u/d] - [1.39 \times curb] - [2.37 \times RR] - [1.44 \times landuse] - [0.41 \times int] \quad \text{(Eqn. 18)}$$

Table 17. Coefficients and P-Values for T1One Model without Sight Distance Variable

Variable	**Coefficient**	***P-value***
(Intercept)	49.9665	*<.0001*
Grade	-0.9595	*0.0023*
u/d	-1.2590	*0.0066*
Curb	-1.3446	*<.0001*
RR	-2.3790	*<.0001*
Landuse	-1.4418	*<.0001*
Int	-0.4143	*<.0001*
AIC	12276.2	
BIC	12283.8	
logLik	-6136.1	
σ_v	6.1972	
σ	17.2829	
ICC	*0.26*	

The resulting models for the tangent section with one lane in each direction of travel (T1One) are characterized by the following observations:

- If available, the sight distance (based on field measurements) has an important contribution to the model. This sight distance variable is correlated to the roadside rating with a Pearson Correlation Coefficient of -0.53. There were no other strong correlations between sight distance and the remaining variables (including grade).

- Drivers tend to select lower operating speeds with the increase of the roadside object densities or the decrease of the roadside object offsets as represented by an increasing roadside rating value.

- Drivers tend to travel at lower speeds with the increase of intersection density.

- Drivers tended to travel at slightly lower speeds at locations with raised curb.

- As land use density increased, adjacent operating speeds decreased.

- The vertical grade characterized by extreme slopes influenced operating speed. At locations with steep downhill grades, operating speeds increased. At locations with varying grades (up and down and moderate slopes), speeds were not affected. At locations with steep uphill grades, operating speeds decreased.

- The tangent with one lane in each direction of travel configuration included only a small number of medians. As a result, the influence of median treatments for this road configuration was not statistically significant.

An important step in evaluating model fit is to verify model development assumptions based on the inspection of diagnostic plots of residuals. Figure 51 includes a Pearson residual scatter plot, histogram plot, and quantile plot. For residuals to represent a random distribution, the scatter plot should not depict trends or extreme outliers. The histogram plot should represent a normal distribution as shown. Finally, the quantile plot should closely resemble a 45-degree linear representation as shown. The diagnostic plots, therefore, support the initial model assumptions.

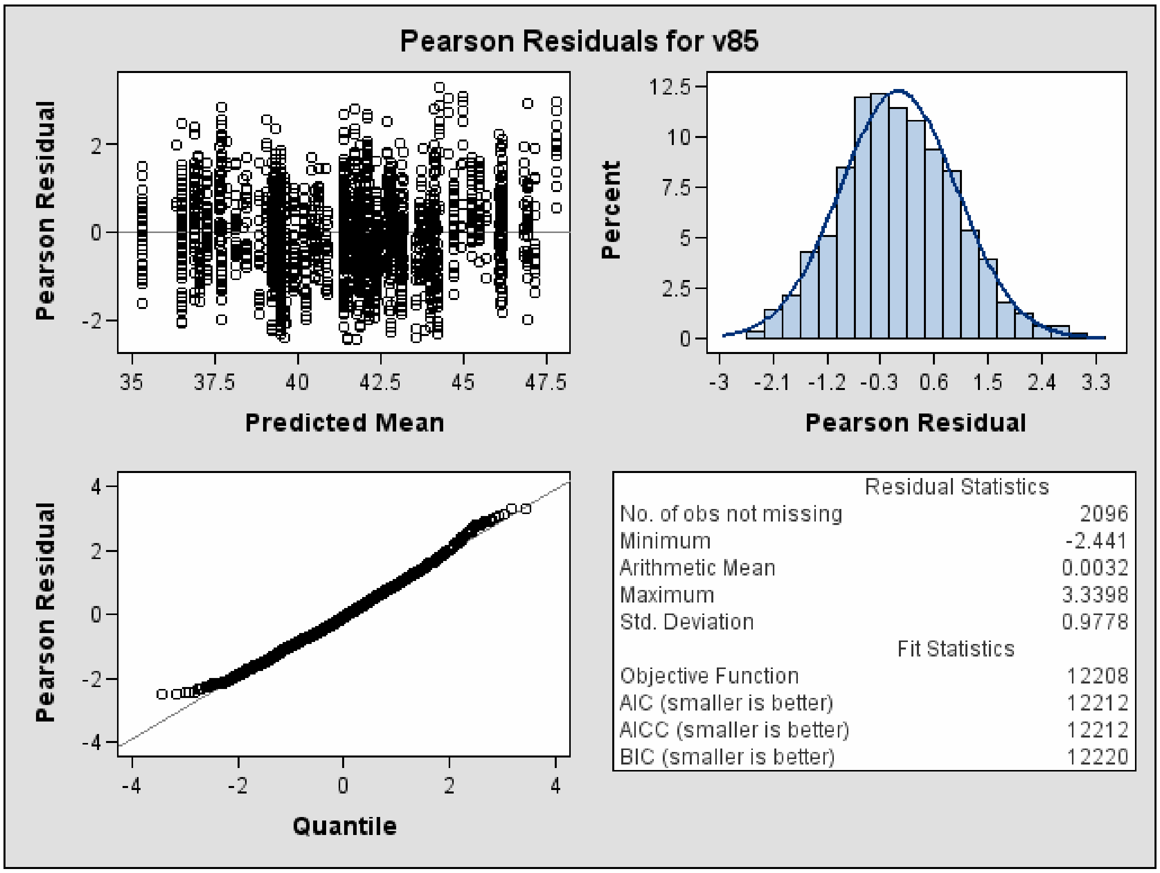


Figure 51. Residual Diagnostic Plots for the T1One Model

The research team evaluated various interactions between independent variables in the model development, but did not identify any significant improvements to the model using this approach. Therefore, the final models presented in this report do not include any interaction between variables.

The ICC values of 0.25 and 0.26 for the T1One models with and without the sight distance variable indicates that 25 to 26 percent of the unexplained variance of speeds was due to the characteristics of different drivers or vehicles.

The research team applied a model development procedure similar to that described for the T1One model to the tangent, two-lanes per direction of travel (T1Two), the horizontal curve with one-lane per direction of travel (HZOne), and the horizontal curve with two-lanes per direction of travel (HZTwo). Table 18 summarizes the 85th percentile model for all four roadway configurations.

Table 18. 85th Percentile Full Models for the Four Road Configurations

Full Tangent Models	
One-lane per direction of travel (T1One)	*Two-lanes per direction of travel (T1Two)*
Includes Sight Distance: V85 = 45.53 – [1.03 x grade] - [0.98 x u/d] + [0.66 x sd] - [1.57 x curb] - [1.70 x RR] - [1.62 x landuse] - [0.45 x int]	Includes Sight Distance: V85 = 40.30 + [4.93 x grade] - [1.19 x u/d] + [0.84 x median] +[0.95 x sd] - [0.69 x curb] - [0.03 x dwy] - [0.14 x int]
Excludes Sight Distance: $V85_{No\ SD}$ = 49.97 – [1.44 x grade] - [0.82 x u/d] - [1.39 x curb] - [2.37 x RR] - [1.44 x landuse] - [0.41 x int]	Excludes Sight Distance: $V85_{No\ SD}$ = 41.27 + [5.38 x grade] - [1.08 x u/d] + [0.37 x lanewidth] - [1.01 x curb] - [0.03 x dwy]
Full Horizontal Curve Models	
One-lane per direction of travel (HZOne)	*Two-lanes per direction of travel (HZTwo)*
Includes Sight Distance: V85 = 40.78 + [0.94 x grade] - [0.71 x u/d] + [0.67 x sd] - [1.45 x curb] - [1.17 x RR] -[1.48 x sw] - [1.04 x landuse] - [0.02 x dwy] - [0.24 x int] + [0.00234 x radius]	Includes Sight Distance: V85 = 23.53 - [1.16 x u/d] + [1.04 x lanewidth] + [0.71 x sd] + [4.33 x curb] - [0.84 x sw] - [0.14 x dwy] - [0.34 x int] + [0.0058 x radius] - [1.59 x curvedir]
Excludes Sight Distance: $V85_{No\ SD}$ = 43.70 - [0.66 x u/d] - [2.15 x curb] - [1.69 x RR] - [1.82 x sw] - [0.13 x int] + [0.003 x radius]	Excludes Sight Distance: $V85_{No\ SD}$ = 26.27 + [0.69 x lanewidth] + [2.65 x median] + [3.92 x curb] + [1.59 x RR] - [0.14 x dwy] - [0.28 x int] + [0.0049 x radius] - [1.53 x curvedir]

Upon inspection of the full tangent and horizontal curve models, a few variables appear unstable such as the curb variable. This appears to occur specifically for the two-lane configurations. This odd “curb” variable influence may be due to the nature of the data collected. For roads with one-lane in each direction of travel, the research team could confidently conclude that the observed speed was associated with the lane immediately adjacent to the curb. For roads with two-lanes in each direction of travel, the driver could position his or her vehicle in either lane (thereby resulting in an unstable curb variable). This phenomenon provides suspect model results if applied on average across both lanes. As a result, the research team developed reduced models that excluded unstable candidate variables due to this vehicle positioning question. Similarly, during model validation the "grade" variable and the "u/d" variable were determined to be too strongly correlated for both variables to be retained in the final models. As a result, the more stable "u/d" variable remains in the final reduced models. Table 19 presents the reduced final models for the 85th percentile condition.

Table 19. 85th Percentile Reduced Models for the Four Road Configurations

Reduced Tangent Models	
One-lane per direction of travel (T1One)	*Two-lanes per direction of travel (T1Two)*
Includes Sight Distance: $V85 = 45.10 - [0.96 \times u/d] + [0.71 \times sd]$ $- [1.37 \times curb] - [1.57 \times RR]$ $- [1.44 \times landuse] - [0.02 \times dwy]$ $- [0.47 \times int]$	Includes Sight Distance: $V85 = 41.62 - [0.79 \times u/d]$ $+[1.40 \times sd] - [1.18 \times RR]$ $- [0.21 \times int]$
Excludes Sight Distance: $V85_{No\ SD} = 49.85 - [0.77 \times u/d]$ $- [1.10 \times curb] - [2.31 \times RR]$ $- [1.16 \times landuse] - [0.02 \times dwy]$ $- [0.43 \times int]$	Excludes Sight Distance: $V85_{No\ SD} = 39.07 + [0.85 \times lanewidth]$ $- [1.05 \times median] - [1.01 \times RR]$ $- [0.13 \times int]$
Reduced Horizontal Curve Models	
One-lane per direction of travel (HZOne)	*Two-lanes per direction of travel (HZTwo)*
Includes Sight Distance: $V85 = 40.56 + [0.69 \times sd]$ $- [2.01 \times curb] - [0.96 \times RR]$ $- [0.71 \times landuse] - [0.03 \times dwy]$ $- [0.35 \times int] + [0.0024 \times radius]$	Includes Sight Distance: *Sight Distance No Longer Significant for Reduced Model Format (without curb or sidewalk variables)*
Excludes Sight Distance: $V85_{No\ SD} = 43.76 - [2.20 \times curb]$ $- [1.49 \times RR] - [0.02 \times dwy]$ $- [0.31 \times int] + [0.003 \times radius]$	Excludes Sight Distance: $V85_{No\ SD} = 38.45 + [2.61 \times median]$ $- [0.09 \times dwy] + [0.0059 \times radius]$ $- [1.39 \times curvedir]$

The final reduced 85th percentile models as shown in Table 19 exhibit the following characteristics:

- Roadside features such as the roadside rating and adjacent land use more directly influence the operating speed on two-way, two-lane roads than they do when there are two travel lanes in each direction.

- The density of intersections and driveways minimally affect speed choices.

- For the observed sites, extreme vertical grade more directly influenced speed choices for roads on tangent than it did at locations with horizontal curvature. This observation, however, may be due to the site locations with extreme grade and merits further analysis of the influence of vertical grade on speed choice.

- In general, raised curb, more dense roadside and land use conditions, and more frequent driveway and intersection density contribute to reduced operating speeds.

- Flatter curves (larger radii) and improved available sight distance contributed to increased operating speeds.

- The influence of a median varies. This observation may be due to a smaller median sample size.

The 85th percentile speed represents a large percentage of the driving population; however, use of this value exclusively for speed estimation implies that approximately 15-percent of drivers who exceeded this speed can be disregarded. It is logical that a full understanding of free-flow operating speeds on low-speed urban streets should encompass the various speed thresholds expected for the facility. Table 20, therefore, includes models for the 5th, 15th, 50th, 85th, and 95th percentile speeds as well as mean speed for tangent sections with one lane of travel in each direction. Similarly, Table 21, Table 22, and Table 23 each include similar models for the T1Two, HZone, and HZTwo road configurations.

Table 20. Summary of Final Tangent Models with One Travel Lane per Direction

T1One Models with Sight Distance (sd)								
Model Definition	Intercept	u/d	sd	curb	RR	landuse	dwy	int
5th Percentile	43.68	-0.83	+0.45	-0.76	-1.54	-1.74	-0.04	-0.43
15th Percentile	43.96	-0.85	+0.48	-0.89	-1.54	-1.68	-0.03	-0.44
50th Percentile (Median)	44.57	-0.87	+0.59	-1.13	-1.54	-1.59	-0.03	-0.46
85th Percentile	45.10	-0.96	+0.71	-1.37	-1.57	-1.44	-0.02	-0.47
95th Percentile	45.28	-0.98	+0.76	-1.52	-1.55	-1.36	-0.02	-0.48
Mean	44.54	-0.89	+0.60	-1.13	-1.55	-1.56	-0.03	-0.46

T1One Models that exclude Sight Distance (sd)							
Model Definition	Intercept	u/d	Curb	RR	Landus e	dwy	int
5th Percentile	46.70	-0.70	-0.59	-2.01	-1.56	-0.04	-0.40
15th Percentile	47.21	-0.72	-0.71	-2.05	-1.48	-0.04	-0.41
50th Percentile (Median)	48.52	-0.71	-0.91	-2.16	-1.34	-0.03	-0.43
85th Percentile	49.85	-0.77	-1.10	-2.31	-1.16	-0.02	-0.43
95th Percentile	50.36	-0.78	-1.23	-2.34	-1.06	-0.02	-0.44
Mean	48.54	-0.73	-0.90	-2.17	-1.32	-0.03	-0.42

Note: Shaded variables not significant at the 95% level, but still pass the 90% test.

Table 21. Summary of Final Tangent Models with Two Travel Lanes per Direction

T1Two Models with Sight Distance (sd)					
Model Definition	Intercept	u/d	sd	RR	Int
5th Percentile	39.89	-0.96	+1.32	-1.30	-0.31
15th Percentile	40.06	-0.96	+1.35	-1.27	-0.29
50th Percentile (Median)	40.75	-0.85	+1.37	-1.20	-0.24
85th Percentile	41.62	-0.79	+1.40	-1.18	-0.21
95th Percentile	41.87	-0.83	+1.43	-1.19	-0.19
Mean	40.81	-0.89	+1.37	-1.21	-0.24

T1Two Models that exclude Sight Distance (sd)					
Model Definition	Intercept	lanewidth	median	RR	Int
5th Percentile	34.86	+1.01	-0.89	-1.07	-0.22
15th Percentile	35.59	+0.98	-0.97	-1.06	-0.20
50th Percentile (Median)	37.23	+0.91	-1.01	-1.01	-0.15
85th Percentile	39.07	+0.85	-1.05	-1.01	-0.13
95th Percentile	39.50	+0.84	-1.08	-1.02	-0.10
Mean	37.20	+0.92	-1.01	-1.02	-0.16

Note: The shaded variable is not significant at the 95% level, but still passes the 90% test.

Table 22. Summary of Horizontal Curve Models with One Travel Lane per Direction

HZOne Models with Sight Distance (sd)								
Model Definition	Intercept	sd	curb	RR	landuse	dwy	int	Radius
5th Percentile	38.65	+0.48	-1.86	-0.86	-0.60	-0.02	-0.32	+0.0021
15th Percentile	38.99	+0.51	-1.87	-0.89	-0.62	-0.02	-0.31	+0.0021
50th Percentile (Median)	39.66	+0.59	-1.94	-0.89	-0.63	-0.03	-0.33	+0.0023
85th Percentile	40.56	+0.69	-2.01	-0.96	-0.71	-0.03	-0.35	+0.0024
95th Percentile	40.85	+0.73	-2.05	-0.97	-0.69	-0.04	-0.36	+0.0025
Mean	38.74	+0.60	-1.94	-0.92	-0.65	-0.03	-0.33	+0.0023

HZOne Models that exclude Sight Distance (sd)						
Model Definition	Intercept	curb	RR	dwy	int	Radius
5th Percentile	40.66	-2.10	-1.26	0	-0.28	+0.0026
15th Percentile	41.15	-2.14	-1.33	0	-0.27	+0.0026
50th Percentile (Median)	42.24	-2.26	-1.43	0	-0.28	+0.0028
85th Percentile	43.76	-2.20	-1.49	-0.02	-0.31	+0.0030
95th Percentile	44.28	-2.25	-1.53	-0.02	-0.32	+0.0031
Mean	42.35	-2.27	-1.46	0	-0.29	+0.0029

Note: Shaded variables not significant at the 95% level, but still pass the 90% test.

Table 23. Summary of Horizontal Curve Models with Two Travel Lanes per Direction

HZTwo Models that exclude Sight Distance (sd)					
Model Definition	Intercept	median	dwy	radius	Curvedir
5th Percentile	36.74	+2.00	-0.13	+0.0060	-1.50
15th Percentile	36.97	+2.15	-0.12	+0.0059	-1.44
50th Percentile (Median)	37.72	+2.33	-0.11	+0.0058	-1.33
85th Percentile	38.45	+2.61	-0.09	+0.0059	-1.39
95th Percentile	38.66	+2.72	-0.09	+0.0060	-1.43
Mean	37.71	+2.36	-0.11	+0.0059	-1.41

The final models for the various speed categories and road configurations can be used as a means for estimating expected speeds for a given facility. The following section provides an example of how a transportation professional may use these models to evaluate expected speed performance on a candidate facility.

6.4 Application Example of the Operating Speed Model

The application of operating speed models to the design or evaluation process may help engineers better understand the expected performance of a facility. The following example demonstrates this use:

Example Problem:

Assume a road has a tangent section with one travel lane in each direction. The road is further characterized as follows:

- lane width: 3.6 m (12 ft)
- roadside rating: 2
- driveway density: 30 driveways per mile
- Intersection density: 3 per mile
- curb type: raised curb present
- sidewalk: yes
- on-street parking: no
- land use: residential
- field measured sight distance: not available
- vertical grade = +6%

Question #1: What is the expected 85th percentile speed for this facility?

Solution #1: Using the models provided in Table 19 and the variable definitions in Table 13, the 85th percentile speed can be computed as follows:

$V85_{No\ SD}$ = 49.85 - [0.77 x u/d] - [1.10 x curb] - [2.31 x RR] - [1.16 x landuse] - [0.02 x dwy] - [0.43 x int]

Where:

u/d = + 1	landuse = 0
curb = 1	dwy = 30
RR = 2	int = 3

$V85_{No\ SD}$ = 49.85 - 0.77 -1.10 -4.62 -0 – 0.60 – 1.29 = 41.47 mph

Question #2: If the road described in Question #1 includes a horizontal curve with a radius of 600' and the road curves to the right, what is the expected 85th percentile speed for this facility?

Solution #2: The 85th percentile speed can be computed as follows:

$V85_{No\ SD}$ = 43.76 - [2.20 x curb] - [1.49 x RR] - [0.02 x dwy] - [0.31 x int] + [0.003 x radius]

Where: Variables are same from before, except now R=600'

$V85_{No\ SD}$ = 43.76 - 2.20 - 2.98 - 0.60 – 0.93 +1.8 = 38.85 mph

<u>Question #3</u>: For the tangent portion of the road as described in Question #1, list the expected 5^{th}, 15^{th}, 50^{th}, 85^{th}, and 95^{th} percentile speeds. What would be the expected mean?

<u>Solution #3:</u> Using the equations depicted in Table 20. Summary of Final Tangent Models with One Travel Lane per Direction, the various speeds would be as follows:

V5 = 38.99 mph;
V15 = 40.36 mph;
V50 = 40.39 mph;
V85 = 41.47 mph (calculated previously);
V95 = 41.75 mph; and
Mean = 40.41 mph.
The expected speeds range from approximately 39 to 42 mph with a mean around 40.4 mph.

6.5 Summary of Model development

The use of mixed models for estimating speed conditions from equipped-vehicle data collection methodologies addresses the violation of independence required for many statistical methods. The use of one robust speed model is not practical for evaluating operating speeds for free-flow conditions at low-speed urban street locations since roadside features have a stronger effect on two-lane, two-way roads than on their four-lane counterparts. Most of the variables performed in an intuitive manner. For example, better sight distance corresponded to higher operating speeds.

The models may benefit from improved data regarding the vertical grade as well as a larger median sample size. In general, the resulting models provide additional insight into driver selected speeds at urban locations. Future urban street speed model development should benefit from the information contained in this report as it will enable researchers to target specific variable sensitivities.

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7 SUMMARY OF FINDINGS

The operating speed a driver selects may be due to influences by prevailing traffic, traffic control devices, or the road environment. In an effort to determine the road environment influence, free-flow speed evaluations for corridors remote to stop-control traffic control devices are studied. This report reviews existing speed models in the published literature, historic statistical procedures for speed model development, common methods used to evaluate driver's perception of the road environment, and possible factors that may influence a driver's speed choice.

With the dramatic advancements in global positioning system technology, it is now feasible to evaluate transportation facilities using probe vehicles. For this study, the research team used data for one year (2004) where drivers in the Atlanta, Georgia region freely drove their personal vehicles equipped with data collection equipment. The equipment and data collection process were part of the Commute Atlanta project and provided to this project as a courtesy. Speed data for free-flow conditions, however, is not straightforward since there is no clear way to determine if a vehicle is operating under free-flow conditions. As a result, the research team for this project developed an extensive free-flow speed filter process. The procedures involved in this data mining effort are described in detail in Chapter 4. Considerable road characteristic data complemented the speed data resulting in a robust data set ideal for the evaluation of urban speed conditions.

The ultimate objective of this effort was the development of operating speed models for the various low-speed urban streets. Ninety-two corridors were initially selected for detailed analysis with 72 corridors ultimately included in the final analysis. Corridors were selected such that the corridor speed limit was 45 mph or less, corridors were distributed throughout the Metro Atlanta area, and the corridors were distributed among the low speed urban street functional classes. The research effort sought to achieve a balance between the number of drivers and number of trips. Finally, mainline stop control traffic devices may be found only on the corridor boundaries. Several unique configurations such as reversible lanes were excluded from the study due to small sample size.

Ultimately, the research team developed four road type models. For a tangent location, they developed speed models for two-way, two-lane roads and models for two-way, four-lane roads. Similar models were also developed for horizontal curve locations where the radius was 1700’ or less. Chapter 6 includes a summary of the various speed models. The models uncovered several roadway characteristics:

- Roadside features such as the roadside rating and adjacent land use more directly influence the operating speed on two-way, two-lane roads than they do when there are two travel lanes in each direction.

- The density of intersections and driveways minimally affect speed choices.

- For the observed sites, extreme vertical grade more directly influences speed choices for roads on tangent than at locations with horizontal curvature. This observation, however, may be due to the site locations with extreme grade and merits further analysis of the influence of vertical grade on speed choice.

- In general, raised curb, more dense roadside and land use conditions, and more frequent driveway and intersection density contribute to reduced operating speeds.

- Flatter curves (larger radii) and improved available sight distance contribute to higher operating speeds.

- The influence of a median varies. This observation may be due to a smaller median sample size.

A final logic analysis considered variable stability and application as well as changing correlations during the calibration procedures. The final result was a set of speed curves for the 5th, 15th, 50th, 85th, and 95th percentile speed at each of the four road configurations. In addition, a model for estimating the mean speed was also developed. Finally, Chapter 6 includes an example problem to depict the ease for using the various speed models. One variable, field-measured sight distance, was determined to be a costly variable. As a result, this report includes models with and without the sight distance variable, as appropriate.

The research summarized in this report will be useful to researchers in the future as they try to further define the complex urban environment and the variables that influence speed choices in this environment. There were several clear findings in this analysis, but also the report helped identify a few future research needs such as increased evaluation of sites with medians and extreme vertical grades. The models contained in this report were simplified so that they can be easily used by agencies to evaluate expected speed conditions for their facilities. For example, the hard-to-measure sight distance variable is included in the final models where significant, but a second set of models that exclude this variable are also present to enhance potential use by practitioners in direct application of these models. It would, however, be wise for the jurisdiction to field validate the models to determine if the drivers in their jurisdictions have similar speed choices as those for the study sites (Atlanta, Georgia drivers).

8 REFERENCES

1. Mustyn, B. J., and D. Sheppard. *A National Survey of Driver's Attitudes and Knowledge About Speed Limits*, SR 548, Transport and Road Research Laboratory, Crowthorne, England, 1980.

2. European Transport Safety Council. *Reducing Traffic Injuries Resulting from Excess and Inappropriate Speed*, Brussels, 1995.

3. American Association of State Highway and Transportation Officials. *A Policy on Geometric Design of Highways and Streets*, Washington, D.C., 1994.

4. Highway Capacity Manual. Transportation Research Council, Washington, D.C., 2000.

5. Oppenlander, J. C. *Variables Influencing Spot-Speed Characteristics—Review of the Literature*, Special Report 89, HRB, National Research Council, Washington, D.C., 1966.

6. Ericsson, E. "Variability in Urban Driving Patterns," *Transportation Research Part D, Vol. 5*, 2000.

7. Gattis, J. L., A. Watts. "Urban Street Speed Related to Width and Functional Class," *Journal of Transportation Engineering*, Vol. 125, No. 3, 1999.

8. Kanellaidis, G. "Factors Affecting Drivers' Choice of Speed on Roadway Curves," *Journal of Safety Research*, Vol. 26, No. 1, 1995.

9. Ottesen, J. L., and R. A. Krammes. "Speed-Profile Model for a Design-Consistency Evaluation Procedure in the United States," *Transportation Research Record 1701*, TRB, National Research Council, Washington, D. C., 2000.

10. Kanellaidis, G., J. Golias, and S. Efstathiadis. "Drivers Speed Behaviour on Rural Road Curves," *Traffic Engineering & Control*, July/August, 1990.

11. Warren, D. L. Chapter 17: "Speed Zoning and Control," *Synthesis of Safety Research Related to Traffic Control and Roadway Elements Vol. 2*, Report No. FHWA-TS-82-233, Federal Highway Administration, Washington, DC, 1982.

12. Tignor, S. C., and D. Warren. "Driver Speed Behavior on U.S. Streets and Highways," Compendium of Technical Papers, Institute of Transportation Engineers, Washington, DC, 1990.

13. Rowan, N. J., and C. J. Keese. *A Study of Factors Influencing Traffic Speed*, Bulletin 341, HRB, National Research Council, Washington, D. C., 1962.

14. Cooper, D., P. Jordan, and J. Young. *The Effect on Traffic Speeds of Resurfacing a Road*, SR 571, Transport and Road Research Laboratory, Crowthorne, England, 1980.

15. Parker, M. R., Jr. *Effects of Raising and Lowering Speed Limits on Selected Roadway Sections*, Report No. FHWA-RD-92-084, Federal Highway Administration, Washington, DC, January 1997.

16. Tenkink, E. "The effect of lead vehicles on speed choice under restricted sight distances." In A. G. Gale, I. D. Brown, C. M. & S. Taylor (Eds.), *Vision in Vehicles – III* (pp. 391-397). New York: Elseview Science Publishing Company, 1991.

17. American Associate of State Highway and Transportation Officials. *Roadside Design Guide*, Wasington, D.C., 1996.

18. Polus A., M. Livneh, and J. Craus. "Effect of Traffic and Geometric Measures on Highway Average Running Speed," *Transportation Research Record 960*, TRB, National Research Council, Washington, D. C., 1984.

19. Federal Highway Administration. *Manual on Uniform Traffic Control Devices (MUTCD), Millennium Edition*, Washington, D.C., 2000.

20. Garber, N. J., and R. Gadiraju. "Factors Affecting Speed Variance and Its Influence on Accidents," *Transportation Research Record 1213*, TRB, National Research Council, Washington, D. C., 1989.

21. Leish, J. E., and J. P. Leish. "New Concepts in Design Speed Application." *Transportation Research Record 631*, TRB, National Research Council, Washington, D. C., 1977.

22. Fitzpatrick, K., P. Carlson, M. Brewer, and M. Woolridge. "Design Factors That Affect Driver Speed on Suburban Streets," 80th TRB Annual Meeting, 2001. Washington, D.C.

23. Zwahlen, H. T. "Advisory Speed Signs and Curve Signs and Their Effect on Driver Eye Scanning and Driver Performance," *Transportation Research Record 1111*, TRB, National Research Council, Washington, D. C., 1987.

24. Várhelyi, A. "Drivers' Speed Behaviour at A Zebra Crossing: A Case Study," *Accident Analysis and Prevention*, Vol. 30, No. 6, 1998.

25. Ewing, R. *Traffic Calming State of the Practice*. By ITE for the Federal Highway Adminstration, Washington, D.C., 1999.

26. Amour, M. "The Effect of Police Presence on Urban Driving Speeds," *ITE Journal*, February, 1986.

27. Walter, C. E. "Suburban Residential Traffic Calming," ITE 1994 Compendium of Technical Papers.

28. Comte, S. L., and A. H. Jamson. "Traditional and Innovative Speed-Reducing Measures for Curves: An Investigation of Driver Behavior Using a Driving Simulator," *Safety Science, Vol. 36*, 2000.

29. Barbosa, H. M., M. R. Tight and A. D. May. "A Model of Speed Profile for Traffic Calmed Roads," *Transportation Research Part A, Vol. 34*, 2000.

30. Beaubien, R. F. "Controlling Speeds on Residential Streets," *ITE Journal*, 1989.

31. Liang, W. L., M. Kyte, F. Kitchener, and P. Shannon. "Effect of Environmental Factors on Driver Speed: A Case Study," *Transportation Research Record 1635*, TRB, National Research Council, Washington, D. C., 1998.

32. Lamm, R., J. C. Hayward, and J. G. Cargin. "Comparison of Different Procedures for Evaluating Speed Consistency," *Transportation Research Record 1100*, TRB, National Research Council, Washington, D.C., 1986.

33. Scallen, S., and J. Carmody. *Investigating The Effects of Roadway Design on Driver Behavior: Application for Minnesota Highway Design*, Minnesota Department of Transportation, February, 1999.

34. Transportation Research Board National Research Council. *Managing speed: Review of current practice for setting and enforcing speed limits.* Washington, D.C.: National Academy Press, 1998.

35. Kang, K. "Ordered Probit Model of the Speed Selection Behavior: Results Based on a Korean Micro Data," Transportation, Traffic Safety and Health – Human Behavior, Fourth International Conference, Tokyo, Japan, 1998.

36. Poe, C. M., J. P. Tarris, and J. M. Mason. *Relationship of Operating Speeds to Roadway Geometric Design Speeds*, Pennsylvania Transportation Institute, December, 1996.

37. Hassan, V., and S. Easa. "Visual Perception of Horizontal Curves in Three-Dimensional Combined Alignments," 79th TRB Annual Meeting, 2000. Washington, D.C.

38. Gibreel, G. M., S. M. Easa, and I. A. El-Dimeery. "Prediction of Operating Speed on Three Dimensional Highway Applications," *ASCE Journal of Transportation Engineering, Vol. 127, No. 1*, 2001.

39. Haglund, M., and L. Åberg. "Speed choice in relation to speed limit and influences from other drivers**,"** *Transportation Research Part F: Traffic Psychology and Behaviour, Vol. 3, Issue 1*, 2000.

40. Elslande, P. V., and L. Faucher-Alberton. "When expectancies become certaintied: A potential adverse effect of experience." In Rothengatter, T., and E. C. Vaya (Eds)., *Traffic Transport and Psychology: Theory and Application* (pp. 147-153). New York: Pergamon, 1997.

41. Smiley, A. "Driver Speed Estimation: What Road Designers Should Know," Presented at: Transportation Research Boad, 78th Annual Meeting Workshop on Role of Geometric Design & Human Factors in Setting Speed, January 11, 1999.

42. Bartmann, A., W. Spijkers, and M. Hess. "Street environment, driving speed and field of vision." In A. G. Gale, I. D. Brown, C. M. & S. Taylor (Eds.), *Vision in Vehicles – III* (pp. 281-389). New York: Elseview Science Publishing Company, 1991.

43. The National Highway Traffic Safety Administration, *National Survey of Speeding and Other Unsafe Driving Actions*, 1998.

44. Lamm, R., E. M. Choueiri, J. C. Hayward, and A. Paluri. "Possible Design Procedure to Promote Design Consistency in Highway Geometric Design on Two-Lane Rural Roads," *Transportation Research Record 1195*, TRB, National Research Council, Washington, D.C., 1988.

45. Passetti, K. A., and D. B. Fambro. "Operating Speeds on Curves with and without Spiral Transitions," *Transportation Research Record 1658*, TRB, National Research Council, Washington, D.C., 1999.

46. Glennon, J. C., T. R. Neuman, and J. E. Leisch. *Safety and Operational Considerations for Design of Rural Highway Curves*, Report No. FHWA-RD-86/035. Federal Highway Administration, Washington DC, August 1983.

47. Andjus, V., and M. Maletin. "Speeds of Cars on Horizontal Curves," *Transportation Research Record 1612*, TRB, National Research Council, Washington, D. C., 1998.

48. Islam, M. N., and P. N. Seneviratne. "Evaluation of Design Consistency of Two-Lane Rural Highways," *ITE Journal*, *Vol. 64, No. 2,* 1997.

49. Fitzpatrick, K., L. Elefteriadon, D. W. Harwood, J. M. Collins, J. McFaddin, I. B. Anderson, R. A. Krammes, N. Irizarry, K. D. Parma, K. M. Bauer, and K. Passetti. *Speed Prediction for Two Lane Rural Highways*, Report No. FHWA-RD-99-171, Federal Highway Administration, August, 2000.

50. McLean, J. "An Alternative to the Design Speed Concept for Low Speed Alignment Design," *Transportation Research Record 702*, TRB, National Research Council, Washington, D.C., 1979.

51. Lamm, R., E. M. Choueiri, and T. Mailaender. "Comparison of Operating Speeds on Dry and Wet Pavements of Two-Lane Rural Highways," *Transportation Research Record 1280*, TRB, National Research Council, Washington, D.C., 1990.

52. Andueza, P. J. "Mathematical Models of Vehicular Speed on Mountain Roads," *Transportation Research Record 1701*, TRB, National Research Council, Washington, D. C., 2000.

53. Donnell, E. T., Y. Ni, M. Adolini, and L. Elefteriadou. "Speed Prediction Models For Trucks on Two-Lane Rural Highways", 80th TRB Annual Meeting, 2001. Washington, D.C.

54. Liapis, E. D., B. Psarianos, and E. Kasappi. "Speed Behavior Analysis at Curved Ramp Sections of Minor Interchanges," 80th TRB Annual Meeting, 2001. Washington, D.C.

55. Polus A., K. Fitzpatrick, and D. B. Fambro. "Predicting Operating Speeds on Tangent Sections of Two-lane Rural Highways," *Transportation Research Record 1737*, TRB, National Research Council, Washington, D.D., 2000.

56. Jessen, D. R., K. S. Schurr, P. T. McCoy, G. Pesti, and R. R. Huff. "Operating Speed Prediction on Crest Curves of Rural Two-Lane Highways in Nebraska," 80th TRB Annual Meeting, 2001. Washington, D.C.

57. Poe, C. M., J. P. Tarris, J. M. Mason. "Influence of Access and Land Use on Vehicle Operating Speeds Along Low-Speed Urban Streets," *Conference Proceedings for the Second National Access Management Conference*, Vail, Colorado, 1996.

58. Fitzpatrick, K., C. B. Shamburger, R. A. Krammes, and D. B. Fambro. "Operating Speed on Suburban Arterial Curves," *Transportation Research Record 1579*, TRB, National Research Council, Washington, D.C., 1997.

59. Bonneson, J. A. "Side Friction and Speed as Controls for Horizontal Curve Design," *Journal of Transportation Engineering, Vol. 125, No. 6*, 1999.

60. Poe, C. M., & J. M. Mason. "Analyzing Influence of Geometric Design on Operating Speeds along Low-Speed Urban Streets: Mixed-Model Approach," *Transportation Research Record 1737*, TRB, National Research Council, Washington, D.C., 2000.

61. Klee, H., C. Bauer, E. Radwan, and H. Al-deek. "Preliminary Validation of Driving Simulator Based on Forward Speed," *Transportation Research Record 1689*, TRB, National Research Council, Washington, D.C., 1999.

62. Lockwood, C. "Virtual Traffic Claming," *Traffic Engineering & Control, Vol. 39, No. 1*, 1998.

63. Zakowska, L. "Road Curve Evaluation Based on Road View Perception Study," *Transportation Research Record 1689*, TRB, National Research Council, Washington, D.C., 1999.

64. Fambro, D. B., R. J. Koppa, D. L. Picha, and K. Fitzpatrick. "Driver Braking Performance in Stopping Sight Distance Situations," *Transportation Research Record 1701*, TRB, National Research Council, Washington, D.C., 2000.

65. Guensler, R., M. Thornton, M. O. Rodgers, K. Dixon, J. Pearson, W. Bachman, J. Leonard, and J. Daniel. "Evaluation of Ramp Metering Impacts on Air Quality: The Atlanta I-75 Case Study," Georgia Institute of Technology, 2000.

66. Wang, J., K. Dixon, H. Li, and J. Ogle., "Normal Acceleration Behavior of Starting Passenger Vehicles at All-Way Stop-Controlled Intersections," Transportation Research Record, (currently at publisher, TRB Paper No. 04-4838), 2004.

67. Wang, J., K. Dixon, H. Li, and J. Ogle. "Normal Deceleration Behavior of Passenger Vehicles at Stop-Sign-Controlled Intersections Based on In-Vehicle GPS Data," Transportation Research Record; (accepted for publication, TRB Paper No. 05-1711), 2005.

68. Verbeke, G., and G. Molenberghs. Linear Mixed Models in Practice, A SAS-Oriented Approach, Springer, 1997.

69. Verbeke, G., and G. Molenberghs. Linear Mixed Models for Longitudinal Data, Springer, 2000.

70. Harrell, F.E. Regression Modeling Strategies, With Applications to Linear Models, Logistic Regression, and Survival Analysis, Springer Series in Statistics, Springer, 2001.

71. Sakamoto, Y., Ishiguro, M., and Kitagawa G. Akaike Information Criterion Statistics. D. Reidel Publishing Company, 1986.

72. Schwarz, G. Estimating the Dimension of a Model, Annals of Statistics, v6, 1978.

73. Schurr, K.S., McCoy P.T., Pesti, G., and Huff, R. Relationship between Design, Operating, and Posted Speeds on Horizontal Curves on Rural Two-Lane Highways in Nebraska. Presented at the 81th TRB Annual Meeting, Washington, D.C., 2002.

APPENDIX A. ACRONYM DEFINITIONS

Table 24. Acronyms Used in Report Text

Acronym	Definition
2-D	Two-Dimensional
3-D	Three-Dimensional
AASHTO	American Association of State Highway and Transportation Officials
AIC	Akaike Information Criteria
BIC	Bayesian Information Criteria
df	Degrees of Freedom
DOD	Department of Defense
DST	Daylight Savings Time
FHWA	Federal Highway Administration
GDOT	Georgia Department of Transportation
GIS	Geographical Information System
GMT	Greenwich Mean Time
GPS	Global Positioning System
GUI	Graphic User Interface
HCM	Highway Capacity Manual
ICC	Intra-class Correlation+96
L	Likelihood
LID	Link ID
LME	Linear Mixed Effects
ML	Maximum likelihood
MLE	Mixed linear effects
MUTCD	Manual of Uniform Traffic Control Devices
NHTSA	National Highway Traffic Safety Administration
OLR	Ordinary Linear Regression
PDOT	Position Dilution of Precision
Q-Q Plot	Quantile-Quantile Plot
R^2	Coefficient of determination
RC File	Road Characteristic File (for the State of Georgia)
RCLINK	10-digit GDOT route identification number
REML	Restricted Maximum Likelihood Estimation
RR	Roadside Rating Variable
SAT	Number of Satellites
SSE	Error Sum of Squares
SSR	Regression Sum of Squares
SSTO	Total Sum of Squares
TRL	Transport Research Laboratory
TWLT	Two-way left-turn lane
UCF	University of Central Florida

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APPENDIX B. SUMMARY OF EXISTING MODELS

Table 25. Existing Operating Speed Models for Rural Conditions

Speed Prediction Model	Location	R^2
Lamm et al. (44) Lamm et al. (44) V85 = 93.85 – 1.82DC	Two-lane rural highway curves, passenger cars, grades < 5%, 0 < degree of curvature	0.79
McLean (50) $V85 = 53.8 + 0.464V_F - 3.26(1/R)*10^3 + 8.5(1/R)^2*10^4$	Two-lane rural highway curves, passenger cars	0.92
Passetti and Fambro (45) Passetti and Fambro (45) V85 = 103.9 – 3030.5(1/R)	Two-lane rural highway curves, passenger cars	0.68
Kanellaidis et al. (10) Kanellaidis et al. (10) $V85 = 129.88 - 623.1/(1/R)^{0.5}$	Two-lane rural highway curves, passenger cars	0.78
Glennon et al. (46) Glennon et al. (46) V85 = 150.08 – 4.14DC	High-speed rural alignments, passenger cars, grades < 5%	0.84
Ottesen and Krammes (9) V85 = 102.44 – 1.57DC + 0.012L – 0.01DC*L V85* = 41.62 – 1.29DC + 0.0049L – 0.12DC*L + 0.95Va	Two-lane rural highway curves, passenger cars, grades < 5%, 3 < degree of curvature < 12. *Model is useful only if approach tangent speeds are actually measured.	0.81 0.90
Andjus and Maletin (47) V85 = 16.92 lnR – 14.49	Two-lane rural road curves, passenger cars, grades < 4%	0.98
Islam and Seneviratne (48) $V85^{(1)} = 95.41 - 1.48*DC - 0.012*DC^2$ $V85^{(2)} = 103.03 - 2.41*DC - 0.029*DC^2$ $V85^{(3)} = 96.11 - 1.07*DC$	Two-lane rural highways, passenger cars (1) beginning of curve (2) middle of curve (3) end of the curve	0.99 0.98 0.90
Andueza (52) $V85^{(1)} = 98.25 - 2795/R2 - 894/R1 + 7.486D + 9308L1$ $V85^{(2)} = 100.69 - 3032/R1 + 27819L1$	Two-lane rural highways, passenger cars (1) horizontal curves (2) tangents	0.84 0.79
Jessen et al. (56) $V85^{(1)} = 73.9 + 0.400Vp - 0.124GAPT - 0.00143T_{ADT}$ $V85^{(2)} = 83.1 + 0.307Vp - 0.00141T_{ADT}$	Two-lane rural highways, passenger cars (1) crest vertical curve with limited stopping sight distance (2) crest vertical curve with non-limited stopping sight distance	0.55 0.42
Fitzpatrick et al. (49) $V85^{(1)} = 102.10 - 3077.13/R$ $V85^{(2)} = 105.98 - 3709.90/R$ $V85^{(3)} = 104.82 - 3574.51/R$ $V85^{(4)} = 96.61 - 2752.19/R$ $V85^{(5)} = 105.32 - 3438.19/R$ $V85^{(6)} = 103.24 - 3576.51/R$ $V85^{(7)}$ = assumed desired speed $V85^{(8)}$ = assumed desired speed $V85^{(9)} = 105.08 - 149.69/K$	Two-lane rural highway, passenger cars (1) horiz. curve, –9% < grade < –4%, (2) horiz. curve, –4% < grade < 0, (3) horiz. curve, 0 < grade < 4%, (4) horiz. curve, 4% < grade < 9%, (5) horiz. curve with sag vertical curve (6) horiz. curve combined with limited sight distance crest vertical curve, (7) sag vertical curve on horizontal tangent, (8) vertical crest curve with unlimited sight distance on horizontal tangent, (9) vertical crest curve with limited sight distance on horizontal tangent	0.58 0.76 0.76 0.53 0.92 0.74 0.80

Speed Prediction Model	Location	R^2
Gibreel and Easa (38) $V85^{(1)} = 91.81 + 0.010R + 0.468\sqrt{Lv} - 0.006G_1^3 - 0.878\,ln(A) - 0.826\,ln(L_0)$ $V85^{(2)} = 47.96 + 7.217\,ln(R) + 1.534\,ln(\sqrt{Lv}) - 0.258G_1 - 0.653A - 0.008\,L_0 + 0.020\,exp(E,)$ $V85^{(3)} = 76.42 + 0.023R + 2.300 \times 10^{-4}K^2 - 0.008\,exp(A) - 1.230 \times 10^{-4}\,L_0^2 + 0.062\,exp(E),$ $V85^{(4)} = 82.78 + 0.011R + 2.067\,ln(K) - 0.361G_2 - 1.091 \times 10^{-4}\,L_0^2 + 0.036\,exp(E),$ $V85^{(5)} = 109.45 - 1.257G_2 - 1.586\,ln(L_0),$	Two-lane rural highway, passenger cars (1) Point 1 was set out at about 60-80 m on the approach tangent before the beginning of the spiral curve. (2) Point 2 was the end of spiral curve and the beginning of horizontal curve in the direction of travel (SC). (3) Point 3 was the midpoint of horizontal curve (MC). (4) Point 4 was the end of horizontal curve and the beginning of spiral curve in the direction of travel (CS). (5) Point 5 was set out at about 60–80 m on the departure tangent after the end of the spiral curve.	0.98 0.98 0.94 0.95 0.79
Polus et al. (55) $V85^{(1)} = 101.11 - 3420/GMs$ $V85^{(2)} = 105.00 - 28.107/e^{(0.00108*GML)}$ $V85^{(3)} = 97.73 + 0.00067*GM$ $V85^{(4)} = 105.00 - 22.953/\,e^{(0.00012*GML)}$	Two-lane rural highway tangents, passenger cars (1) R1 and R2 ≤ 250 m and TL ≤ 150 m. (2) R1 and R2 < 250 m and TL between 150 and 1000 m. (3) R1 and R2 > 250 m and TL between 150 and 1000 m. (4) TL > 1000 m.	0.55 0.74 0.20 0.84
Liapis et al. (54) $V85^{(1)} = -0.360839DC - 3.683548E + 75.161$ $V85^{(2)} = -0.472675DC - 3.795879E + 85.186$	Two-lane rural roads, passenger cars (1) off-ramps (2) on-ramps	0.75 0.73

Speed Prediction Model	Location	R^2
Donnell et al. (53)	Two-lane rural highway, trucks	
$V85^{(1)}$ = 51.5 + 0.137R - 0.779 GAPT + 0.0127 L1 - 0.000119 (L1 * R)	$^{(1)}$ 200 meters prior to horizontal curve	0.62
$V85^{(2)}$ = 54.9 + 0.123 R - 1.07 GAPT + 0.0078 L1 - 0.000103 (L1 * R)	$^{(2)}$ 150 meters prior to horizontal curve	0.63
$V85^{(3)}$ = 56.1 + 0.117 R - 1.15 GAPT + 0.0060 L1 - 0.000097 (L1 * R)	$^{(3)}$ 100 meters prior to horizontal curve	0.61
$V85^{(4)}$ = 78.7 + 0.0347 R - 1.30 GAPT + 0.0226 L1	$^{(4)}$ 50 meters prior to horizontal curve	0.55
$V85^{(5)}$ = 78.4 + 0.0140 R - 1.40 GDEP – 0.00724 L2	$^{(5)}$ Beginning of horizontal curve (PC)	0.56
$V85^{(6)}$ = 75.8 + 0.0176 R - 1.41 GDEP – 0.0086 L2	$^{(6)}$ QP	0.60
$V85^{(7)}$ = 75.1 + 0.0176 R - 1.48 GDEP – 0.00836 L2	$^{(7)}$ Middle of horizontal curve (MC)	0.60
$V85^{(8)}$ = 74.7 + 0.0176 R - 1.59 GDEP – 0.00814 L2	8.1.1.1.1.1 $^{(8)}$ 3QP	0.61
$V85^{(9)}$ = 74.5 + 0.0176 R - 1.69 GDEP – 0.00810 L2	8.1.1.1.1.2 $^{(9)}$ End of horizontal	0.61
$V85^{(10)}$ = 82.8 - 2.00 GDEP – 0.00925L2	curve (PT)	0.56
$V85^{(11)}$ = 83.1 - 2.08 GDEP – 0.00934L2	8.1.1.1.1.3 $^{(10)}$ 50 meter beyond	0.58
$V85^{(12)}$ = 83.6 - 2.29 GDEP – 0.00919L2	horizontal curve	0.60
$V85^{(13)}$ = 84.1 - 2.34 GDEP – 0.00944L2	8.1.1.1.1.4 (PT50)	0.61
	8.1.1.1.1.5 $^{(11)}$ 100 meter beyond horizontal curve	
	8.1.1.1.1.6 (PT100)	
	8.1.1.1.1.7 $^{(12)}$ 150 meter beyond horizontal curve	
	8.1.1.1.1.8 (PT150)	
	8.1.1.1.1.9 $^{(13)}$ 200 meter beyond horizontal curve	
	8.1.1.1.1.10 (PT200)	

Where:
V85 = 85th percentile speed (km/h)
Va = 85th percentile speed on approach tangent (km/h)
Vp = posted speed (km/h)
V_F = Desired speed of the 85th percentile (km/h)
R = horizontal curve radius (m)
DC = degree of curve (degree/30 m)
L = length of curve (m)
L1 = tangent length before the curve (m)
L2 = tangent length after the curve (m)
R1 = radius of the previous curve (m),
R2 = radius of the following curve (m)
S = minimum sight distance for the curve (m)
GAPT = grade of approach tangent
GDEP = grade of departure tangent
T_{ADT} = ADT
K = rate of vertical curvature
E = superelevation rate
A = algebraic difference in grades
G_1 and G_2 = first and second grades in the direction of travel in percent
L_0 = horizontal distance between point of vertical intersection and point of horizontal intersection (m)
TL = tangent length (m)
GMs = (R1 + R2)/2 (m)
GML = (TL* $(R1*R2)^{0.5}$)/100 (m^2)

Table 26. Existing Operating Speed Models for Urban Conditions

Speed Prediction Model	Location	R^2
Fitzpatrick et al. (58) $V85^{(1)} = 56.34 + 0.808R^{0.5} + 9.34/AD$ $V85^{(2)} = 39.51 + 0.556\ (IDS)$	Urban roadways, passenger cars, pickup trucks, and vans (1) suburban arterial horizontal curves, (2) suburban arterial vertical curves	0.72 0.56
Bonneson (59) $V85 = 63.5R(-B + \sqrt{B^2 + \frac{4c}{127R}}) \le Va$ $c = E/100 + 0.256 + (B - 0.0022)Va$ $B = 0.0133 - 0.0074I_{TR}$	Urban roadways, horizontal curves, passenger cars, urban low speed, high speed roadways rural low speed, high speed roadways turning roadways, -8.4% < grade < 8.0%	0.96
Poe et al. (36) Speed = $\beta_0 + \beta_1$(Alignment) + β_2(Cross Section) + β_3(Roadside) + β_4(Traffic Control)	Low speed urban streets, passenger cars, pick up, single-unit truck	
Where: V85 = 85th percentile speed (km/h) Va = 85th percentile speed on approach tangent (km/h) R = horizontal curve radius (m) AD = approach density (approaches per km) IDS = inferred design speed (km/h) E = superelevation rate I_{TR} = indicator variable for turning roadway (= 1.0 if Va > V85; 0.0 otherwise)		

APPENDIX C. SUMMARY OF DATA PROCESSING RESULTS

The detailed data processing procedures outlined in Chapter 4 of this report identify methods used by the research team to identify suitable speed data for free-flow day-time assessments. These target speed conditions provide the best indication of speed information that is a result of the road environment rather than extraneous issues such as traffic congestion, inclement weather, or night time lighting. The following summary outlines the results of this process and references summary tables that demonstrate the influence of a particular data filter.

1. After the GPS data were grouped by corridor location, data processing Step 4 sorted the data and removed duplicates from each file. Table 27 shows the data points before and after removing duplicate data. Every corridor had less than one percent of duplicate data except for corridor number 79 which had approximately 4 percent.

2. Data processing Step 5 applied the 10-second gap rule to separate sub-trips within a trip if it has time. The results of this filter are shown in Table 28.

3. After separating sub-trips (trips that have time gaps larger than 10 seconds), data processing Step 6 filtered out incomplete trips. Table 29 shows the percent loss as the ratio of the number of the removed trips to the total trips before applying this data processing step.

4. After checking for complete trips, data processing Step 7 separated trips on the same corridor by direction of travel. Table 30 shows the number of points distributed to each travel direction. The percent distribution was calculated from the number of points in one direction divided by the total points from both directions of the road. No trips were removed as a result of this data processing step.

5. Data processing Step 8 filtered out trips that occurred during night time hours. The night time filter began 30 minutes before the calculated sunset and extended to 30 minutes after the calculated sunrise time of the next day. Table 31 demonstrates the various trips removed as a result of this night time trip filter.

6. Data processing Step 9 filtered out trips made during increment weather. Table 32 depicts the number of trips affected by this filter.

7. Data processing Step 10A removed non-free-flow trips associated with the deceleration queue. Table 33 shows a summary of trips affected by this data processing step.

8. Data processing Step 10B removed non-free-flow trips using the 4-pattern free-flow configuration and a 10 mph speed filter line. Table 34 shows the results of this data processing filter.

9. Data processing Step 10D removed potentially non-free-flow data by checking the 4-pattern free-flow against the minimum value between 70 percent of speed limit and 70 percent of average driver speed at the mid point. Table 35 demonstrates the results following application of this filter.

10. Data processing Step 12 removed highly deviated trips that were potentially a result of traffic induced fluctuations. Table 36 shows the resulting number of trips following application of this filter.

11. Data processing Step 13 evaluated the GPS signal quality and removed trips that contained questionable signals. Table 37 demonstrates the influence of this GPS signal quality filter.

Table 27. Data Points Before and After Removing Duplicated Data

COR	BEFORE	AFTER	DRIVERS	TRIPS	PERCENT LOSS
01	125008	125000	80	1297	0.01%
02	147662	147653	99	1631	0.01%
03	242331	242315	85	2321	0.01%
04	134266	134250	84	1573	0.01%
05	27463	27457	74	887	0.02%
06	66611	66603	96	1423	0.01%
07	44923	44919	65	1246	0.01%
08	69273	69263	62	2409	0.01%
09	123157	123147	93	1726	0.01%
10	146090	146047	87	1975	0.03%
12	138297	138249	72	1884	0.03%
14	82563	82558	110	1135	0.01%
15	58957	58953	108	650	0.01%
16	57817	57811	113	1274	0.01%
17	86916	86902	87	1516	0.02%
18	67367	67353	135	1627	0.02%
19	59628	59620	64	765	0.01%
20	85482	85477	104	1051	0.01%
21	1231604	1231465	216	7900	0.01%
22	177239	177222	71	2016	0.01%
23	154603	154576	95	1310	0.02%
24	39602	39598	58	824	0.01%
25	69405	69399	95	850	0.01%
26	59259	59254	64	850	0.01%
28	34416	34412	64	515	0.01%
29	70337	70316	66	1045	0.03%
30	135660	135660	47	1235	0.00%
31	97993	97991	74	1663	0.00%
32	69376	69282	60	1038	0.14%
33	41132	41130	59	512	0.00%
34	46961	46959	100	735	0.00%
35	179245	179224	109	1554	0.01%
36	126542	126541	68	1278	0.00%
37	15385	15383	66	682	0.01%
38	68441	68435	74	1073	0.01%
39	93685	93598	96	866	0.09%
40	79669	79650	60	945	0.02%
41	64675	64665	78	890	0.02%
42	97234	97227	93	1492	0.01%
51	26960	26960	51	349	0.00%
52	21743	21739	53	415	0.02%
53	8577	8577	23	108	0.00%
54	6652	6650	20	149	0.03%
55	18938	18938	21	238	0.00%
56	9700	9700	16	125	0.00%
57	9410	9384	43	230	0.28%
58	47367	47363	31	292	0.01%
59	3490	3490	17	78	0.00%
60	32577	32576	15	229	0.00%
61	3105	3104	11	33	0.03%

COR	BEFORE	AFTER	DRIVERS	TRIPS	PERCENT LOSS
62	**15655**	**15655**	**17**	**381**	0.00%
63	**19325**	**19324**	**19**	**296**	0.01%
64	**4995**	**4993**	**23**	**64**	0.04%
65	**5570**	**5570**	**14**	**77**	0.00%
66	**3580**	**3576**	**16**	**116**	0.11%
67	**27487**	**27486**	**29**	**452**	0.00%
68	**41431**	**41429**	**24**	**567**	0.00%
69	**28036**	**28036**	**18**	**326**	0.00%
70	**40574**	**40559**	**12**	**453**	0.04%
71	**44927**	**44924**	**47**	**603**	0.01%
72	**26668**	**26668**	**42**	**456**	0.00%
73	**142650**	**142649**	**60**	**1464**	0.00%
74	**28515**	**28514**	**42**	**627**	0.00%
76	**1607**	**1607**	**11**	**50**	0.00%
77	**137105**	**66**	**10**	**1512**	0.02%
78	**11471**	**87**	**31**	**166**	0.02%
79	**36588**	**88**	**22**	**452**	3.93%
80	**28129**	**54**	**17**	**225**	0.01%
81	**19330**	**80**	**27**	**263**	0.00%
82	**35029**	**143**	**47**	**626**	0.01%
83	**29630**	**218**	**74**	**710**	0.01%
84	**115139**	**143**	**32**	**898**	0.02%
85	**47676**	**93**	**31**	**548**	0.00%
86	**49442**	**269**	**90**	**751**	0.25%
87	**87101**	**112**	**28**	**907**	0.00%
88	**15195**	**57**	**27**	**249**	0.01%
89	**90978**	**164**	**47**	**1077**	0.02%
90	**41619**	**169**	**44**	**352**	0.00%
91	**17518**	**68**	**14**	**279**	0.00%
92	**55907**	**146**	**33**	**683**	0.00%
93	**11335**	**90**	**27**	**208**	0.01%
94	**80806**	**149**	**34**	**1239**	0.02%
95	**18061**	**147**	**56**	**292**	0.02%
96	**13504**	**67**	**23**	**187**	0.01%
97	**30445**	**90**	**31**	**321**	0.02%
98	**41696**	**180**	**51**	**824**	0.00%
99	**16675**	**103**	**44**	**190**	0.00%
100	40813	40811	48	655	0.00%

Table 28. Comparison Between Number of Trips and Sub-Trips

COR	POINTS	DRIVERS	TRIPS	SUBTRIP	PERCENT TRIPS INCREASED
01	64060	72	1202	1251	4.08%
02	147545	99	1631	1703	4.41%
03	170253	85	2228	2388	7.18%
04	134238	84	1573	1695	7.76%
05	25121	74	887	956	7.78%
06	62153	96	1411	1525	8.08%
07	44855	65	1246	1296	4.01%
08	69228	62	2409	2530	5.02%
09	59203	91	1542	1603	3.96%
10	127388	75	1832	2355	28.55%
12	138103	72	1884	2285	21.28%
14	55676	109	1124	1157	2.94%
15	58945	108	650	686	5.54%
16	57805	113	1274	1356	6.44%
17	86791	87	1516	1601	5.61%
18	67321	135	1627	1743	7.13%
19	59570	64	765	813	6.27%
20	84672	104	1051	1115	6.09%
21	222774	113	3567	3841	7.68%
22	148906	70	1845	1960	6.23%
23	154519	95	1310	1380	5.34%
24	39574	58	824	847	2.79%
25	40843	89	795	841	5.79%
26	59243	64	850	929	9.29%
28	20393	64	513	521	1.56%
29	70285	66	1045	1111	6.32%
30	135632	47	1235	1271	2.91%
31	97985	74	1663	1975	18.76%
32	69270	60	1038	1075	3.56%
33	33778	58	509	527	3.54%
34	46957	100	735	764	3.95%
35	177510	109	1554	1675	7.79%
36	126481	68	1278	1308	2.35%
37	15299	66	682	715	4.84%
38	68352	74	1073	1145	6.71%
39	41049	81	448	502	12.05%
40	79637	60	945	1099	16.30%
41	64625	78	890	949	6.63%
42	57400	79	1075	1124	4.56%
51	19169	49	345	353	2.32%
52	20232	53	413	460	11.38%
53	5557	19	100	107	7.00%
54	721	20	97	100	3.09%
55	18938	21	238	254	6.72%
56	6640	16	120	122	1.67%
57	1959	6	37	39	5.41%
58	28208	27	281	300	6.76%
59	3490	17	78	82	5.13%
60	32008	15	229	247	7.86%
61	3104	11	33	45	36.36%

COR	POINTS	DRIVERS	TRIPS	SUBTRIP	PERCENT TRIPS INCREASED
62	15655	17	381	401	5.25%
63	19318	19	296	322	8.78%
64	4993	23	64	70	9.38%
65	5570	14	77	87	12.99%
66	3573	16	116	129	11.21%
67	24286	29	452	467	3.32%
68	18562	14	206	226	9.71%
69	11278	15	319	325	1.88%
70	40485	12	453	716	58.06%
71	44903	47	603	635	5.31%
72	26665	42	456	475	4.17%
73	65554	58	1437	1476	2.71%
74	26936	26	280	311	11.07%
76	841	11	50	54	8.00%
77	80510	10	1496	1837	22.79%
78	11469	31	166	203	22.29%
79	35016	22	452	548	21.24%
80	18016	17	225	244	8.44%
81	15882	27	263	283	7.60%
82	25005	47	612	661	8.01%
83	6804	41	274	314	14.60%
84	52190	32	720	756	5.00%
85	47676	31	548	557	1.64%
86	45006	89	749	801	6.94%
87	41246	21	681	687	0.88%
88	14712	25	245	256	4.49%
89	90924	47	1077	1168	8.45%
90	24443	42	327	348	6.42%
91	17499	14	279	287	2.87%
92	51475	32	679	700	3.09%
93	11321	27	208	213	2.40%
94	79815	34	1239	1332	7.51%
95	18057	56	292	307	5.14%
96	13503	23	187	202	8.02%
97	26585	31	321	339	5.61%
98	41628	51	824	883	7.16%
99	12893	44	189	197	4.23%
100	34953	48	652	683	4.75%

Table 29. Comparison of GPS Before and After Removing Incomplete Trips

COR	BEFORE		AFTER		PERCENT LOSS
	POINTS	NO. OF SUBTRIPS	POINTS	NO. OF SUBTRIPS	
1	64060	1251	60979	1145	7.15%
2	147545	1703	121868	1252	23.61%
3	170253	2388	143526	1898	16.61%
4	134238	1695	85036	914	43.10%
5	25121	956	19732	691	27.17%
6	62153	1525	31429	742	48.90%
7	44855	1296	28851	756	40.05%
8	69228	2530	51316	1274	49.81%
9	59203	1603	48059	1194	24.45%
10	127388	2355	68247	1135	41.27%
12	138103	2285	76013	765	59.45%
14	55676	1157	47543	871	23.75%
15	58945	686	46182	428	34.77%
16	57805	1356	30621	444	65.46%
17	86791	1601	50874	871	43.34%
18	67321	1743	52007	1263	24.03%
19	59570	813	50300	625	20.13%
20	84672	1115	60319	741	31.49%
21	222774	3841	161047	1452	59.69%
22	148906	1960	78166	717	61.79%
23	154519	1380	122951	897	32.60%
24	39574	847	27068	526	36.29%
25	40843	841	36629	719	12.70%
26	59243	929	35714	500	43.41%
28	20393	521	16543	420	18.91%
29	70285	1111	50007	730	30.72%
30	135632	1271	118735	1040	16.92%
31	97985	1975	35388	500	70.05%
32	69270	1075	57176	866	17.15%
33	33778	527	28898	416	19.25%
34	46957	764	30613	374	49.25%
35	177510	1675	147771	1096	29.99%
36	126481	1308	114866	1170	9.70%
37	15299	715	8610	297	57.62%
38	68352	1145	46430	640	41.01%
39	41049	502	27712	268	43.30%
40	79637	1099	49683	530	44.87%
41	64625	949	40032	485	46.18%
42	57400	1124	44670	575	47.07%
51	19169	353	17977	232	33.33%
52	20232	460	9913	184	56.90%
53	5557	107	954	15	85.00%
54	721	100	0	0	100.00%
55	18938	254	17087	209	15.55%
56	6640	122	6091	109	10.00%
57	1959	39	1893	32	13.51%
58	28208	300	21857	197	31.32%
59	3490	82	2749	63	19.23%
60	32008	247	26408	170	25.76%

COR	BEFORE		AFTER		PERCENT LOSS
	POINTS	NO. OF SUBTRIPS	POINTS	NO. OF SUBTRIPS	
61	3104	45	2550	31	36.36%
62	15655	401	7412	120	68.77%
63	19318	322	14224	186	37.50%
64	4993	70	3624	33	50.00%
65	5570	87	3748	68	16.88%
66	3573	129	2463	41	65.52%
67	24286	467	22903	408	10.84%
68	18562	226	15205	165	21.36%
69	11278	325	11128	318	1.57%
70	40485	716	9095	108	77.26%
71	44903	635	33077	459	24.54%
72	26665	475	22925	378	18.42%
73	65554	1476	46785	594	59.08%
74	26936	311	21029	198	30.00%
76	841	54	609	8	90.00%
77	80510	1837	27114	247	83.82%
78	11469	203	3018	42	74.70%
79	35016	548	18184	277	40.49%
80	18016	244	14812	197	13.78%
81	15882	283	13895	233	14.83%
82	25005	661	19472	370	39.87%
83	6804	314	2035	36	86.86%
84	52190	756	45689	626	15.83%
85	47676	557	42798	492	10.77%
86	45006	801	33573	497	34.71%
87	41246	687	40961	678	0.59%
88	14712	256	13826	192	21.63%
89	90924	1168	75845	917	17.55%
90	24443	348	19295	252	24.46%
91	17499	287	14272	233	16.49%
92	51475	700	45181	590	13.84%
93	11321	213	9618	176	16.35%
94	79815	1332	62846	814	34.54%
95	18057	307	14354	215	27.40%
96	13503	202	10075	146	27.27%
97	26585	339	24925	305	6.85%
98	41628	883	31203	646	23.91%
99	12893	197	9057	132	32.28%
100	34953	683	27125	473	28.53%

Table 30. Directional Distribution

COR	NO. OF TRIPS	DIST	COR	NO. OF TRIPS	**DIST**
01_EB	**574**	**50.35%**	**01_WB**	**566**	49.65%
02_NB	**526**	**42.05%**	**02_SB**	**725**	57.95%
03_EB	**892**	**47.05%**	**03_WB**	**1004**	52.95%
04_EB	**478**	**52.30%**	**04_WB**	**436**	47.70%
05_NB	**412**	**59.62%**	**05_SB**	**279**	40.38%
06_NB	**365**	**49.32%**	**06_SB**	**375**	50.68%
07_NB	**276**	**36.51%**	**07_SB**	**480**	63.49%
08_EB	**787**	**61.97%**	**08_WB**	**483**	38.03%
09_EB	**719**	**60.32%**	**09_WB**	**473**	39.68%
10_NB	**587**	**51.72%**	**10_SB**	**548**	48.28%
12_NB	**748**	**97.78%**	**12_SB**	**17**	2.22%
14_EB	**413**	**47.47%**	**14_WB**	**457**	52.53%
15_EB	**219**	**51.17%**	**15_WB**	**209**	48.83%
16_NB	**219**	**49.32%**	**16_SB**	**225**	50.68%
17_NB	**289**	**33.26%**	**17_SB**	**580**	66.74%
18_NB	**613**	**48.57%**	**18_SB**	**649**	51.43%
19_EB	**275**	**44.00%**	**19_WB**	**350**	56.00%
20_EB	**453**	**61.22%**	**20_WB**	**287**	38.78%
21_EB	**771**	**53.17%**	**21_WB**	**679**	46.83%
22_EB	**289**	**40.31%**	**22_WB**	**428**	59.69%
23_NB	**482**	**53.73%**	**23_SB**	**415**	46.27%
24_NB	**409**	**78.05%**	**24_SB**	**115**	21.95%
25_NB	**295**	**41.09%**	**25_SB**	**423**	58.91%
26_EB	**296**	**59.20%**	**26_WB**	**204**	40.80%
28_EB	**254**	**60.77%**	**28_WB**	**164**	39.23%
29_EB	**306**	**41.92%**	**29_WB**	**424**	58.08%
30_NB	**577**	**55.48%**	**30_SB**	**463**	44.52%
31_EB	**317**	**63.40%**	**31_WB**	**183**	36.60%
32_NB	**380**	**43.88%**	**32_SB**	**486**	56.12%
33_NB	**257**	**61.78%**	**33_SB**	**159**	38.22%
34_EB	**183**	**48.93%**	**34_WB**	**191**	51.07%
35_NB	**597**	**54.47%**	**35_SB**	**499**	45.53%
36_EB	**496**	**42.39%**	**36_WB**	**674**	57.61%
37_NB	**178**	**60.14%**	**37_SB**	**118**	39.86%
38_NB	**422**	**66.04%**	**38_SB**	**217**	33.96%
39_EB	**149**	**55.60%**	**39_WB**	**119**	44.40%
40_EB	**286**	**53.96%**	**40_WB**	**244**	46.04%
41_NB	**254**	**52.37%**	**41_SB**	**231**	47.63%
42_NB	**351**	**61.04%**	**42_SB**	**224**	38.96%
51_NB	**100**	**43.10%**	**51_SB**	**132**	56.90%
52_NB	**78**	**42.39%**	**52_SB**	**106**	57.61%
53_NB	**15**	**100.00%**	**53_SB**	**0**	0.00%
55_NB	**104**	**49.76%**	**55_SB**	**105**	50.24%
56_NB	**64**	**58.72%**	**56_SB**	**45**	41.28%
57_NB	**13**	**40.63%**	**57_SB**	**19**	59.38%
58_EB	**108**	**54.82%**	**58_WB**	**89**	45.18%
59_EB	**22**	**34.92%**	**59_WB**	**41**	65.08%
60_EB	**19**	**11.18%**	**60_WB**	**151**	88.82%
61_NB	**16**	**51.61%**	**61_SB**	**15**	48.39%
62_EB	**53**	**44.54%**	**62_WB**	**66**	55.46%

COR	NO. OF TRIPS	DIST	COR	NO. OF TRIPS	DIST
63_NB	108	58.06%	63_SB	78	41.94%
64_EB	17	51.52%	64_WB	16	48.48%
65_NB	24	35.29%	65_SB	44	64.71%
66_NB	11	26.83%	66_SB	30	73.17%
67_NB	173	42.40%	67_SB	235	57.60%
68_EB	90	54.55%	68_WB	75	45.45%
69_EB	156	49.06%	69_WB	162	50.94%
70_EB	51	47.22%	70_WB	57	52.78%
71_EB	154	33.55%	71_WB	305	66.45%
72_NB	229	60.58%	72_SB	149	39.42%
73_EB	248	41.75%	73_WB	346	58.25%
74_EB	83	41.92%	74_WB	115	58.08%
76_EB	6	75.00%	76_WB	2	25.00%
77_NB	166	67.21%	77_SB	81	32.79%
78_EB	23	54.76%	78_WB	19	45.24%
79_EB	178	64.26%	79_WB	99	35.74%
80_EB	83	42.13%	80_WB	114	57.87%
81_NB	106	45.49%	81_SB	127	54.51%
82_NB	164	44.32%	82_SB	206	55.68%
83_EB	0	0.00%	83_WB	36	100.00%
84_EB	268	42.81%	84_WB	358	57.19%
85_EB	242	49.19%	85_WB	250	50.81%
86_NB	183	36.82%	86_SB	314	63.18%
87_NB	355	52.36%	87_SB	323	47.64%
88_EB	132	68.75%	88_WB	60	31.25%
89_NB	414	45.15%	89_SB	503	54.85%
90_NB	114	45.24%	90_SB	138	54.76%
91_NB	28	12.02%	91_SB	205	87.98%
92_NB	200	33.96%	92_SB	389	66.04%
93_NB	58	32.95%	93_SB	118	67.05%
94_NB	281	34.56%	94_SB	532	65.44%
95_EB	96	44.65%	95_WB	119	55.35%
96_NB	60	41.10%	96_SB	86	58.90%
97_EB	138	45.25%	97_WB	167	54.75%
98_EB	236	36.53%	98_WB	410	63.47%
99_NB	57	43.18%	99_SB	75	56.82%
100_NB	319	67.44%	100_SB	154	32.56%

Table 31. Percent Data Loss after Application of Night Time Filter

COR	BEFORE (TRIPS)	AFTER (TRIPS)	PERCENT LOSS	COR	BEFORE (TRIPS)	AFTER (TRIPS)	PERCENT LOSS
01_EB	574	457	20.38%	01_WB	566	370	34.63%
02_NB	526	342	34.98%	02_SB	725	606	16.41%
03_EB	892	766	14.13%	03_WB	1004	869	13.45%
04_EB	478	435	9.00%	04_WB	436	342	21.56%
05_NB	412	357	13.35%	05_SB	279	235	15.77%
06_NB	365	314	13.97%	06_SB	375	316	15.73%
07_NB	276	238	13.77%	07_SB	480	341	28.96%
08_EB	787	578	26.56%	08_WB	483	386	20.08%
09_EB	719	501	30.32%	09_WB	473	394	16.70%
10_NB	587	288	50.94%	10_SB	548	363	33.76%
12_NB	748	584	21.93%	12_SB	17	14	17.65%
14_EB	413	348	15.74%	14_WB	457	327	28.45%
15_EB	219	197	10.05%	15_WB	209	168	19.62%
16_NB	219	187	14.61%	16_SB	225	194	13.78%
17_NB	289	246	14.88%	17_SB	580	521	10.17%
18_NB	613	514	16.15%	18_SB	649	552	14.95%
19_EB	275	246	10.55%	19_WB	350	312	10.86%
20_EB	453	402	11.26%	20_WB	287	249	13.24%
21_EB	771	541	29.83%	21_WB	679	455	32.99%
22_EB	289	217	24.91%	22_WB	428	328	23.36%
23_NB	482	389	19.29%	23_SB	415	356	14.22%
24_NB	409	262	35.94%	24_SB	115	92	20.00%
25_NB	295	240	18.64%	25_SB	423	343	18.91%
26_EB	296	230	22.30%	26_WB	204	196	3.92%
28_EB	254	181	28.74%	28_WB	164	108	34.15%
29_EB	306	270	11.76%	29_WB	424	308	27.36%
30_NB	577	462	19.93%	30_SB	463	325	29.81%
31_EB	317	199	37.22%	31_WB	183	94	48.63%
32_NB	380	293	22.89%	32_SB	486	284	41.56%
33_NB	257	149	42.02%	33_SB	159	128	19.50%
34_EB	183	89	51.37%	34_WB	191	170	10.99%
35_NB	597	333	44.22%	35_SB	499	433	13.23%
36_EB	496	385	22.38%	36_WB	674	438	35.01%
37_NB	178	141	20.79%	37_SB	118	95	19.49%
38_NB	422	255	39.57%	38_SB	217	191	11.98%
39_EB	149	128	14.09%	39_WB	119	92	22.69%
40_EB	286	204	28.67%	40_WB	244	160	34.43%
41_NB	254	202	20.47%	41_SB	231	167	27.71%
42_NB	351	269	23.36%	42_SB	224	144	35.71%
51_NB	100	93	7.00%	51_SB	132	116	12.12%
52_NB	78	67	14.10%	52_SB	106	93	12.26%
53_NB	15	14	6.67%	53_SB	0	---	---
55_NB	104	79	24.04%	55_SB	105	85	19.05%
56_NB	64	56	12.50%	56_SB	45	42	6.67%
57_NB	13	10	23.08%	57_SB	19	10	47.37%
58_EB	108	102	5.56%	58_WB	89	79	11.24%
59_EB	22	16	27.27%	59_WB	41	40	2.44%
60_EB	19	14	26.32%	60_WB	151	21	86.09%

COR	BEFORE (TRIPS)	AFTER (TRIPS)	PERCENT LOSS	COR	BEFORE (TRIPS)	AFTER (TRIPS)	**PERCENT LOSS**
61_NB	**16**	**10**	**37.50%**	**61_SB**	**15**	**9**	40.00%
62_EB	**53**	**48**	**9.43%**	**62_WB**	**66**	**65**	1.52%
63_NB	**108**	**103**	**4.63%**	**63_SB**	**78**	**54**	30.77%
64_EB	**17**	**16**	**5.88%**	**64_WB**	**16**	**14**	12.50%
65_NB	**24**	**21**	**12.50%**	**65_SB**	**44**	**39**	11.36%
66_NB	**11**	**9**	**18.18%**	**66_SB**	**30**	**28**	6.67%
67_NB	**173**	**159**	**8.09%**	**67_SB**	**235**	**222**	5.53%
68_EB	**90**	**50**	**44.44%**	**68_WB**	**75**	**54**	28.00%
69_EB	**156**	**94**	**39.74%**	**69_WB**	**162**	**158**	2.47%
70_EB	**51**	**41**	**19.61%**	**70_WB**	**57**	**51**	10.53%
71_EB	**154**	**67**	**56.49%**	**71_WB**	**305**	**289**	5.25%
72_NB	**229**	**189**	**17.47%**	**72_SB**	**149**	**122**	18.12%
73_EB	**248**	**189**	**23.79%**	**73_WB**	**346**	**289**	16.47%
74_EB	**83**	**62**	**25.30%**	**74_WB**	**115**	**98**	14.78%
76_EB	**6**	**5**	**16.67%**	**76_WB**	**2**	**2**	0.00%
77_NB	**166**	**166**	**0.00%**	**77_SB**	**81**	**76**	6.17%
78_EB	**23**	**17**	**26.09%**	**78_WB**	**19**	**15**	21.05%
79_EB	**178**	**158**	**11.24%**	**79_WB**	**99**	**88**	11.11%
80_EB	**83**	**67**	**19.28%**	**80_WB**	**114**	**83**	27.19%
81_NB	**106**	**73**	**31.13%**	**81_SB**	**127**	**99**	22.05%
82_NB	**164**	**119**	**27.44%**	**82_SB**	**206**	**162**	21.36%
83_EB	**0**	**---**	**---**	**83_WB**	**36**	**30**	16.67%
84_EB	**268**	**195**	**27.24%**	**84_WB**	**358**	**335**	6.42%
85_EB	**242**	**107**	**55.79%**	**85_WB**	**250**	**224**	10.40%
86_NB	**183**	**124**	**32.24%**	**86_SB**	**314**	**268**	14.65%
87_NB	**355**	**297**	**16.34%**	**87_SB**	**323**	**165**	48.92%
88_EB	**132**	**121**	**8.33%**	**88_WB**	**60**	**13**	78.33%
89_NB	**414**	**248**	**40.10%**	**89_SB**	**503**	**398**	20.87%
90_NB	**114**	**104**	**8.77%**	**90_SB**	**138**	**106**	23.19%
91_NB	**28**	**22**	**21.43%**	**91_SB**	**205**	**132**	35.61%
92_NB	**200**	**145**	**27.50%**	**92_SB**	**389**	**277**	28.79%
93_NB	**58**	**52**	**10.34%**	**93_SB**	**118**	**90**	23.73%
94_NB	**281**	**243**	**13.52%**	**94_SB**	**532**	**367**	31.02%
95_EB	**96**	**75**	**21.88%**	**95_WB**	**119**	**98**	17.65%
96_NB	**60**	**40**	**33.33%**	**96_SB**	**86**	**62**	27.91%
97_EB	**138**	**113**	**18.12%**	**97_WB**	**167**	**129**	22.75%
98_EB	**236**	**187**	**20.76%**	**98_WB**	**410**	**303**	26.10%
99_NB	**57**	**44**	**22.81%**	**99_SB**	**75**	**59**	21.33%
100_NB	319	234	26.65%	100_SB	154	97	37.01%

Table 32. Percent Data Loss after Application of Weather Filter

COR	BEFORE (TRIPS)	AFTER (TRIPS)	PERCENT LOSS	COR	BEFORE (TRIPS)	AFTER (TRIPS)	**PERCENT LOSS**
01_EB	**457**	**352**	**22.98%**	**01_WB**	**370**	**304**	17.84%
02_NB	**342**	**270**	**21.05%**	**02_SB**	**606**	**483**	20.30%
03_EB	**766**	**622**	**18.80%**	**03_WB**	**869**	**702**	19.22%
04_EB	**435**	**350**	**19.54%**	**04_WB**	**342**	**279**	18.42%
05_NB	**357**	**286**	**19.89%**	**05_SB**	**235**	**181**	22.98%
06_NB	**314**	**263**	**16.24%**	**06_SB**	**316**	**251**	20.57%
07_NB	**238**	**189**	**20.59%**	**07_SB**	**341**	**262**	23.17%
08_EB	**578**	**464**	**19.72%**	**08_WB**	**386**	**302**	21.76%
09_EB	**501**	**410**	**18.16%**	**09_WB**	**394**	**320**	18.78%
10_NB	**288**	**235**	**18.40%**	**10_SB**	**363**	**289**	20.39%
12_NB	**584**	**461**	**21.06%**	**12_SB**	**14**	**11**	21.43%
14_EB	**348**	**291**	**16.38%**	**14_WB**	**327**	**270**	17.43%
15_EB	**197**	**164**	**16.75%**	**15_WB**	**168**	**132**	21.43%
16_NB	**187**	**149**	**20.32%**	**16_SB**	**194**	**156**	19.59%
17_NB	**246**	**199**	**19.11%**	**17_SB**	**521**	**427**	18.04%
18_NB	**514**	**410**	**20.23%**	**18_SB**	**552**	**446**	19.20%
19_EB	**246**	**183**	**25.61%**	**19_WB**	**312**	**245**	21.47%
20_EB	**402**	**302**	**24.88%**	**20_WB**	**249**	**178**	28.51%
21_EB	**541**	**439**	**18.85%**	**21_WB**	**455**	**352**	22.64%
22_EB	**217**	**173**	**20.28%**	**22_WB**	**328**	**253**	22.87%
23_NB	**389**	**318**	**18.25%**	**23_SB**	**356**	**288**	19.10%
24_NB	**262**	**204**	**22.14%**	**24_SB**	**92**	**70**	23.91%
25_NB	**240**	**185**	**22.92%**	**25_SB**	**343**	**264**	23.03%
26_EB	**230**	**180**	**21.74%**	**26_WB**	**196**	**153**	21.94%
28_EB	**181**	**143**	**20.99%**	**28_WB**	**108**	**92**	14.81%
29_EB	**270**	**224**	**17.04%**	**29_WB**	**308**	**251**	18.51%
30_NB	**462**	**375**	**18.83%**	**30_SB**	**325**	**256**	21.23%
31_EB	**199**	**166**	**16.58%**	**31_WB**	**94**	**84**	10.64%
32_NB	**293**	**241**	**17.75%**	**32_SB**	**284**	**238**	16.20%
33_NB	**149**	**126**	**15.44%**	**33_SB**	**128**	**101**	21.09%
34_EB	**89**	**69**	**22.47%**	**34_WB**	**170**	**135**	20.59%
35_NB	**333**	**275**	**17.42%**	**35_SB**	**433**	**346**	20.09%
36_EB	**385**	**321**	**16.62%**	**36_WB**	**438**	**346**	21.00%
37_NB	**141**	**117**	**17.02%**	**37_SB**	**95**	**72**	24.21%
38_NB	**255**	**208**	**18.43%**	**38_SB**	**191**	**150**	21.47%
39_EB	**128**	**110**	**14.06%**	**39_WB**	**92**	**83**	9.78%
40_EB	**204**	**170**	**16.67%**	**40_WB**	**160**	**138**	13.75%
41_NB	**202**	**154**	**23.76%**	**41_SB**	**167**	**133**	20.36%
42_NB	**269**	**210**	**21.93%**	**42_SB**	**144**	**118**	18.06%
51_NB	**93**	**81**	**12.90%**	**51_SB**	**116**	**90**	22.41%
52_NB	**67**	**54**	**19.40%**	**52_SB**	**93**	**70**	24.73%
53_NB	**14**	**13**	**7.14%**	**55_NB**	**79**	**61**	22.78%
55_SB	**85**	**71**	**16.47%**	**56_NB**	**56**	**34**	39.29%
56_SB	**42**	**30**	**28.57%**	**57_NB**	**10**	**7**	30.00%
57_SB	**10**	**9**	**10.00%**	**58_EB**	**102**	**83**	18.63%
58_WB	**79**	**64**	**18.99%**	**59_EB**	**16**	**15**	6.25%
59_WB	**40**	**35**	**12.50%**	**60_EB**	**14**	**11**	21.43%
60_WB	**21**	**19**	**9.52%**	**61_NB**	**10**	**9**	10.00%

COR	BEFORE (TRIPS)	AFTER (TRIPS)	PERCENT LOSS	COR	BEFORE (TRIPS)	AFTER (TRIPS)	PERCENT LOSS
61_SB	9	7	22.22%	62_EB	48	37	22.92%
62_WB	65	51	21.54%	63_NB	103	86	16.50%
63_SB	54	43	20.37%	64_EB	16	14	12.50%
64_WB	14	12	14.29%	65_NB	21	19	9.52%
65_SB	39	31	20.51%	66_NB	9	9	0.00%
66_SB	28	23	17.86%	67_NB	159	138	13.21%
67_SB	222	182	18.02%	68_EB	50	41	18.00%
68_WB	54	45	16.67%	69_EB	94	75	20.21%
69_WB	158	117	25.95%	70_EB	41	36	12.20%
70_WB	51	38	25.49%	71_EB	67	50	25.37%
71_WB	289	220	23.88%	72_NB	189	159	15.87%
72_SB	122	108	11.48%	73_EB	189	154	18.52%
73_WB	289	232	19.72%	74_EB	62	52	16.13%
74_WB	98	79	19.39%	76_EB	5	3	40.00%
76_WB	2	1	50.00%	77_NB	166	121	27.11%
77_SB	76	67	11.84%	78_EB	17	12	29.41%
78_WB	15	13	13.33%	79_EB	158	130	17.72%
79_WB	88	69	21.59%	80_EB	67	56	16.42%
80_WB	83	70	15.66%	81_NB	73	67	8.22%
81_SB	99	86	13.13%	82_NB	119	90	24.37%
82_SB	162	118	27.16%	83_WB	30	22	26.67%
84_EB	195	161	17.44%	84_WB	335	256	23.58%
85_EB	107	93	13.08%	85_WB	224	172	23.21%
86_NB	124	96	22.58%	86_SB	268	212	20.90%
87_NB	297	229	22.90%	87_SB	165	131	20.61%
88_EB	121	86	28.93%	88_WB	13	11	15.38%
89_NB	248	199	19.76%	89_SB	398	313	21.36%
90_NB	104	90	13.46%	90_SB	106	84	20.75%
91_NB	22	19	13.64%	91_SB	132	100	24.24%
92_NB	145	124	14.48%	92_SB	277	224	19.13%
93_NB	52	43	17.31%	93_SB	90	73	18.89%
94_NB	243	202	16.87%	94_SB	367	289	21.25%
95_EB	75	63	16.00%	95_WB	98	80	18.37%
96_NB	40	30	25.00%	96_SB	62	51	17.74%
97_EB	113	86	23.89%	97_WB	129	107	17.05%
98_EB	187	144	22.99%	98_WB	303	249	17.82%
99_EB	44	37	15.91%	99_WB	59	46	22.03%
100_NB	234	196	16.24%	100_SB	97	75	22.68%

Table 33. Percent Data Loss after Application of 400-feet Queue Filter

COR	BEFORE (TRIPS)	AFTER (TRIPS)	PERCENT LOSS	COR	BEFORE (TRIPS)	AFTER (TRIPS)	PERCENT LOSS
01_EB	352	318	9.66%	01_WB	304	302	0.66%
02_NB	270	267	1.11%	02_SB	483	389	19.46%
03_EB	622	493	20.74%	03_WB	702	685	2.42%
04_EB	350	313	10.57%	04_WB	279	277	0.72%
05_NB	286	284	0.70%	05_SB	181	181	0.00%
06_NB	263	256	2.66%	06_SB	251	250	0.40%
07_NB	189	187	1.06%	07_SB	262	261	0.38%
08_EB	464	460	0.86%	08_WB	302	297	1.66%
09_EB	410	408	0.49%	09_WB	320	305	4.69%
10_NB	235	234	0.43%	10_SB	289	273	5.54%
12_NB	461	418	9.33%	12_SB	11	11	0.00%
14_EB	291	222	23.71%	14_WB	270	269	0.37%
15_EB	164	155	5.49%	15_WB	132	96	27.27%
16_NB	149	146	2.01%	16_SB	156	146	6.41%
17_NB	199	186	6.53%	17_SB	427	382	10.54%
18_NB	410	403	1.71%	18_SB	446	445	0.22%
19_EB	183	175	4.37%	19_WB	245	184	24.90%
20_EB	302	299	0.99%	20_WB	178	165	7.30%
21_EB	439	413	5.92%	21_WB	352	319	9.38%
22_EB	173	166	4.05%	22_WB	253	228	9.88%
23_NB	318	200	37.11%	23_SB	288	276	4.17%
24_NB	204	204	0.00%	24_SB	70	66	5.71%
25_NB	185	185	0.00%	25_SB	264	264	0.00%
26_EB	180	179	0.56%	26_WB	153	153	0.00%
28_EB	143	142	0.70%	28_WB	92	83	9.78%
29_EB	224	213	4.91%	29_WB	251	248	1.20%
30_NB	375	373	0.53%	30_SB	256	175	31.64%
31_EB	166	136	18.07%	31_WB	84	81	3.57%
32_NB	241	233	3.32%	32_SB	238	221	7.14%
33_NB	126	125	0.79%	33_SB	101	93	7.92%
34_EB	69	66	4.35%	34_WB	135	131	2.96%
35_NB	275	256	6.91%	35_SB	346	239	30.92%
36_EB	321	274	14.64%	36_WB	346	312	9.83%
37_NB	117	117	0.00%	37_SB	72	72	0.00%
38_NB	208	207	0.48%	38_SB	150	141	6.00%
39_EB	110	102	7.27%	39_WB	83	56	32.53%
40_EB	170	169	0.59%	40_WB	138	121	12.32%
41_NB	154	153	0.65%	41_SB	133	125	6.02%
42_NB	210	208	0.95%	42_SB	118	118	0.00%
51_NB	81	80	1.23%	51_SB	90	84	6.67%
52_NB	54	53	1.85%	52_SB	70	68	2.86%
53_NB	13	13	0.00%	55_NB	61	61	0.00%
55_SB	71	71	0.00%	56_NB	34	34	0.00%
56_SB	30	29	3.33%	57_NB	7	7	0.00%
57_SB	9	9	0.00%	58_EB	83	82	1.20%
58_WB	64	64	0.00%	59_EB	15	15	0.00%
59_WB	35	35	0.00%	60_EB	11	10	9.09%
60_WB	19	19	0.00%	61_NB	9	9	0.00%
61_SB	7	7	0.00%	62_EB	37	37	0.00%

COR	BEFORE (TRIPS)	AFTER (TRIPS)	PERCENT LOSS	COR	BEFORE (TRIPS)	AFTER (TRIPS)	**PERCENT LOSS**
62_WB	**51**	**51**	**0.00%**	**63_NB**	**86**	**86**	0.00%
63_SB	**43**	**42**	**2.33%**	**64_EB**	**14**	**13**	7.14%
64_WB	**12**	**12**	**0.00%**	**65_NB**	**19**	**19**	0.00%
65_SB	**31**	**31**	**0.00%**	**66_NB**	**9**	**9**	0.00%
66_SB	**23**	**23**	**0.00%**	**67_NB**	**138**	**138**	0.00%
67_SB	**182**	**180**	**1.10%**	**68_EB**	**41**	**41**	0.00%
68_WB	**45**	**44**	**2.22%**	**69_EB**	**75**	**74**	1.33%
69_WB	**117**	**116**	**0.85%**	**70_EB**	**36**	**36**	0.00%
70_WB	**38**	**38**	**0.00%**	**71_EB**	**50**	**48**	4.00%
71_WB	**220**	**220**	**0.00%**	**72_NB**	**159**	**158**	0.63%
72_SB	**108**	**104**	**3.70%**	**73_EB**	**154**	**154**	0.00%
73_WB	**232**	**222**	**4.31%**	**74_EB**	**52**	**52**	0.00%
74_WB	**79**	**79**	**0.00%**	**76_EB**	**3**	**2**	33.33%
76_WB	**1**	**1**	**0.00%**	**77_NB**	**121**	**113**	6.61%
77_SB	**67**	**62**	**7.46%**	**78_EB**	**12**	**12**	0.00%
78_WB	**13**	**13**	**0.00%**	**79_EB**	**130**	**129**	0.77%
79_WB	**69**	**69**	**0.00%**	**80_EB**	**56**	**54**	3.57%
80_WB	**70**	**69**	**1.43%**	**81_NB**	**67**	**64**	4.48%
81_SB	**86**	**81**	**5.81%**	**82_NB**	**90**	**90**	0.00%
82_SB	**118**	**118**	**0.00%**	**83_WB**	**22**	**22**	0.00%
84_EB	**161**	**152**	**5.59%**	**84_WB**	**256**	**250**	2.34%
85_EB	**93**	**93**	**0.00%**	**85_WB**	**172**	**163**	5.23%
86_NB	**96**	**96**	**0.00%**	**86_SB**	**212**	**190**	10.38%
87_NB	**229**	**224**	**2.18%**	**87_SB**	**131**	**129**	1.53%
88_EB	**86**	**86**	**0.00%**	**88_WB**	**11**	**11**	0.00%
89_NB	**199**	**197**	**1.01%**	**89_SB**	**313**	**294**	6.07%
90_NB	**90**	**87**	**3.33%**	**90_SB**	**84**	**84**	0.00%
91_NB	**19**	**19**	**0.00%**	**91_SB**	**100**	**100**	0.00%
92_NB	**124**	**123**	**0.81%**	**92_SB**	**224**	**217**	3.13%
93_NB	**43**	**43**	**0.00%**	**93_SB**	**73**	**72**	1.37%
94_NB	**202**	**200**	**0.99%**	**94_SB**	**289**	**288**	0.35%
95_EB	**63**	**57**	**9.52%**	**95_WB**	**80**	**76**	5.00%
96_NB	**30**	**30**	**0.00%**	**96_SB**	**51**	**51**	0.00%
97_EB	**86**	**85**	**1.16%**	**97_WB**	**107**	**102**	4.67%
98_EB	**144**	**140**	**2.78%**	**98_WB**	**249**	**249**	0.00%
99_EB	**37**	**37**	**0.00%**	**99_WB**	**46**	**43**	6.52%
100_NB	196	180	8.16%	100_SB	75	75	0.00%

Table 34. Percent Data Loss after Application of 10 mph Line Filter

COR	BEFORE (TRIPS)	AFTER (TRIPS)	PERCENT LOSS	COR	BEFORE (TRIPS)	AFTER (TRIPS)	PERCENT LOSS
01_EB	318	307	3.46%	01_WB	302	297	1.66%
02_NB	267	262	1.87%	02_SB	389	375	3.60%
03_EB	493	488	1.01%	03_WB	685	669	2.34%
04_EB	313	306	2.24%	04_WB	277	272	1.81%
05_NB	284	273	3.87%	05_SB	181	179	1.10%
06_NB	256	247	3.52%	06_SB	250	248	0.80%
07_NB	187	186	0.53%	07_SB	261	261	0.00%
08_EB	460	451	1.96%	08_WB	297	294	1.01%
09_EB	408	394	3.43%	09_WB	305	287	5.90%
10_NB	234	228	2.56%	10_SB	273	268	1.83%
12_NB	418	388	7.18%	12_SB	11	0	100.00%
14_EB	222	198	10.81%	14_WB	269	262	2.60%
15_EB	155	148	4.52%	15_WB	96	90	6.25%
16_NB	146	138	5.48%	16_SB	146	136	6.85%
17_NB	186	157	15.59%	17_SB	382	370	3.14%
18_NB	403	399	0.99%	18_SB	445	438	1.57%
19_EB	175	169	3.43%	19_WB	184	173	5.98%
20_EB	299	288	3.68%	20_WB	165	157	4.85%
21_EB	413	395	4.36%	21_WB	319	275	13.79%
22_EB	166	158	4.82%	22_WB	228	223	2.19%
23_NB	200	162	19.00%	23_SB	276	257	6.88%
24_NB	204	199	2.45%	24_SB	66	66	0.00%
25_NB	185	175	5.41%	25_SB	264	263	0.38%
26_EB	179	178	0.56%	26_WB	153	148	3.27%
28_EB	142	141	0.70%	28_WB	83	83	0.00%
29_EB	213	209	1.88%	29_WB	248	245	1.21%
30_NB	373	338	9.38%	30_SB	175	169	3.43%
31_EB	136	97	28.68%	31_WB	81	79	2.47%
32_NB	233	226	3.00%	32_SB	221	200	9.50%
33_NB	125	98	21.60%	33_SB	93	89	4.30%
34_EB	66	63	4.55%	34_WB	131	126	3.82%
35_NB	256	215	16.02%	35_SB	239	220	7.95%
36_EB	274	253	7.66%	36_WB	312	294	5.77%
37_NB	117	116	0.85%	37_SB	72	72	0.00%
38_NB	207	202	2.42%	38_SB	141	138	2.13%
39_EB	102	91	10.78%	39_WB	56	52	7.14%
40_EB	169	162	4.14%	40_WB	121	116	4.13%
41_NB	153	153	0.00%	41_SB	125	124	0.80%
42_NB	208	207	0.48%	42_SB	118	107	9.32%
51_NB	80	79	1.25%	51_SB	84	82	2.38%
52_NB	53	53	0.00%	52_SB	68	66	2.94%
53_NB	13	0	100.00%	55_NB	61	60	1.64%
55_SB	71	70	1.41%	56_NB	34	0	100.00%
56_SB	29	0	100.00%	57_NB	7	0	100.00%
57_SB	9	0	100.00%	58_EB	82	68	17.07%
58_WB	64	62	3.13%	59_EB	15	0	100.00%
59_WB	35	34	2.86%	60_EB	10	0	100.00%
60_WB	19	0	100.00%	61_NB	9	0	100.00%
61_SB	7	0	100.00%	62_EB	37	0	100.00%

COR	BEFORE (TRIPS)	AFTER (TRIPS)	PERCENT LOSS	COR	BEFORE (TRIPS)	AFTER (TRIPS)	PERCENT LOSS
62_WB	51	0	100.00%	63_NB	86	84	2.33%
63_SB	42	41	2.38%	64_EB	13	0	100.00%
64_WB	12	0	100.00%	65_NB	19	0	100.00%
65_SB	31	0	100.00%	66_NB	9	0	100.00%
66_SB	23	0	100.00%	67_NB	138	127	7.97%
67_SB	180	174	3.33%	68_EB	41	0	100.00%
68_WB	44	0	100.00%	69_EB	74	73	1.35%
69_WB	116	116	0.00%	70_EB	36	0	100.00%
70_WB	38	0	100.00%	71_EB	48	46	4.17%
71_WB	220	212	3.64%	72_NB	158	158	0.00%
72_SB	104	103	0.96%	73_EB	154	128	16.88%
73_WB	222	202	9.01%	74_EB	52	52	0.00%
74_WB	79	75	5.06%	76_EB	2	0	100.00%
76_WB	1	0	100.00%	77_NB	113	0	100.00%
77_SB	62	0	100.00%	78_EB	12	0	100.00%
78_WB	13	13	0.00%	79_EB	129	0	100.00%
79_WB	69	68	1.45%	80_EB	54	54	0.00%
80_WB	69	63	8.70%	81_NB	64	63	1.56%
81_SB	81	78	3.70%	82_NB	90	90	0.00%
82_SB	118	112	5.08%	83_WB	22	22	0.00%
84_EB	152	144	5.26%	84_WB	250	246	1.60%
85_EB	93	91	2.15%	85_WB	163	162	0.61%
86_NB	96	96	0.00%	86_SB	190	180	5.26%
87_NB	224	224	0.00%	87_SB	129	127	1.55%
88_EB	86	0	100.00%	88_WB	11	0	100.00%
89_NB	197	196	0.51%	89_SB	294	285	3.06%
90_NB	87	85	2.30%	90_SB	84	84	0.00%
91_NB	19	0	100.00%	91_SB	100	94	6.00%
92_NB	123	119	3.25%	92_SB	217	192	11.52%
93_NB	43	42	2.33%	93_SB	72	71	1.39%
94_NB	200	190	5.00%	94_SB	288	273	5.21%
95_EB	57	50	12.28%	95_WB	76	71	6.58%
96_NB	30	30	0.00%	96_SB	51	0	100.00%
97_EB	85	84	1.18%	97_WB	102	98	3.92%
98_EB	140	128	8.57%	98_WB	249	247	0.80%
99_EB	37	32	13.51%	99_WB	43	37	13.95%
100_NB	180	166	7.78%	100_SB	75	75	0.00%

Table 35. Percent Data Loss after Application of 4-Pattern Free-Flow Filter

COR	BEFORE (TRIPS)	AFTER (TRIPS)	PERCENT LOSS	COR	BEFORE (TRIPS)	AFTER (TRIPS)	PERCENT LOSS
01_EB	307	284	7.49%	01_WB	297	276	7.07%
02_NB	262	249	4.96%	02_SB	375	341	9.07%
03_EB	488	473	3.07%	03_WB	669	645	3.59%
04_EB	306	289	5.56%	04_WB	272	251	7.72%
05_NB	273	271	0.73%	05_SB	179	179	0.00%
06_NB	247	235	4.86%	06_SB	248	237	4.44%
07_NB	186	184	1.08%	07_SB	261	253	3.07%
08_EB	451	447	0.89%	08_WB	294	291	1.02%
09_EB	394	387	1.78%	09_WB	287	282	1.74%
10_NB	228	219	3.95%	10_SB	268	245	8.58%
12_NB	388	364	6.19%	12_SB	0	0	0.00%
14_EB	198	193	2.53%	14_WB	262	237	9.54%
15_EB	148	120	18.92%	15_WB	90	70	22.22%
16_NB	138	112	18.84%	16_SB	136	113	16.91%
17_NB	157	147	6.37%	17_SB	370	342	7.57%
18_NB	399	378	5.26%	18_SB	438	394	10.05%
19_EB	169	138	18.34%	19_WB	173	149	13.87%
20_EB	288	278	3.47%	20_WB	157	153	2.55%
21_EB	395	373	5.57%	21_WB	275	195	29.09%
22_EB	158	125	20.89%	22_WB	223	210	5.83%
23_NB	162	107	33.95%	23_SB	257	213	17.12%
24_NB	199	191	4.02%	24_SB	66	65	1.52%
25_NB	175	169	3.43%	25_SB	263	257	2.28%
26_EB	178	173	2.81%	26_WB	148	140	5.41%
28_EB	141	131	7.09%	28_WB	83	79	4.82%
29_EB	209	197	5.74%	29_WB	245	229	6.53%
30_NB	338	294	13.02%	30_SB	169	149	11.83%
31_EB	97	90	7.22%	31_WB	79	72	8.86%
32_NB	226	212	6.19%	32_SB	200	187	6.50%
33_NB	98	76	22.45%	33_SB	89	85	4.49%
34_EB	63	58	7.94%	34_WB	126	122	3.17%
35_NB	215	186	13.49%	35_SB	220	181	17.73%
36_EB	253	216	14.62%	36_WB	294	241	18.03%
37_NB	116	115	0.86%	37_SB	72	72	0.00%
38_NB	202	194	3.96%	38_SB	138	123	10.87%
39_EB	91	80	12.09%	39_WB	52	49	5.77%
40_EB	162	134	17.28%	40_WB	116	92	20.69%
41_NB	153	149	2.61%	41_SB	124	121	2.42%
42_NB	207	204	1.45%	42_SB	107	107	0.00%
51_NB	79	78	1.27%	51_SB	82	74	9.76%
52_NB	53	50	5.66%	52_SB	66	56	15.15%
55_NB	60	55	8.33%	55_SB	70	67	4.29%
58_EB	68	63	7.35%	58_WB	62	51	17.74%
59_WB	0	0	0.00	59_WB	34	32	5.88%
63_NB	84	74	11.90%	63_SB	41	0	100.00%
67_NB	127	109	14.17%	67_SB	174	162	6.90%
69_EB	73	70	4.11%	69_WB	116	115	0.86%
71_EB	46	39	15.22%	71_WB	212	196	7.55%
72_NB	158	147	6.96%	72_SB	103	101	1.94%
73_EB	128	124	3.13%	73_WB	202	164	18.81%

COR	BEFORE (TRIPS)	AFTER (TRIPS)	PERCENT LOSS	COR	BEFORE (TRIPS)	AFTER (TRIPS)	**PERCENT LOSS**
74_EB	**52**	**48**	**7.69%**	**74_WB**	**75**	**63**	16.00%
78_EB	**0**	**0**	**0.00%**	**78_WB**	**13**	**12**	7.69%
79_EB	**0**	**0**	**0.00%**	**79_WB**	**68**	**65**	4.41%
80_EB	**54**	**52**	**3.70%**	**80_WB**	**63**	**57**	9.52%
81_NB	**63**	**58**	**7.94%**	**81_SB**	**78**	**65**	16.67%
82_NB	**90**	**88**	**2.22%**	**82_SB**	**112**	**109**	2.68%
83_EB	**0**	**0**	**0.00%**	**83_WB**	**22**	**20**	9.09%
84_EB	**144**	**140**	**2.78%**	**84_WB**	**246**	**227**	7.72%
85_EB	**91**	**88**	**3.30%**	**85_WB**	**162**	**156**	3.70%
86_NB	**96**	**95**	**1.04%**	**86_SB**	**180**	**161**	10.56%
87_NB	**224**	**220**	**1.79%**	**87_SB**	**127**	**126**	0.79%
89_NB	**196**	**180**	**8.16%**	**89_SB**	**285**	**269**	5.61%
90_NB	**85**	**82**	**3.53%**	**90_SB**	**84**	**84**	0.00%
91_NB	**0**	**0**	**0.00%**	**91_SB**	**94**	**0**	100.00%
92_NB	**119**	**108**	**9.24%**	**92_SB**	**192**	**70**	63.54%
93_NB	**42**	**40**	**4.76%**	**93_SB**	**71**	**65**	8.45%
94_NB	**190**	**179**	**5.79%**	**94_SB**	**273**	**260**	4.76%
95_EB	**50**	**37**	**26.00%**	**95_WB**	**71**	**66**	7.04%
96_NB	**30**	**27**	**10.00%**	**96_SB**	**0**	**0**	0.00%
97_EB	**84**	**73**	**13.10%**	**97_WB**	**98**	**90**	8.16%
98_EB	**128**	**100**	**21.88%**	**98_WB**	**247**	**238**	3.64%
99_EB	**32**	**30**	**6.25%**	**99_WB**	**37**	**31**	16.22%
100_NB	166	136	18.07%	100_SB	75	70	6.67%

Table 36. Percent Data Loss after Application of Highly Deviated Trip Filter

COR	BEFORE (TRIPS)	AFTER (TRIPS)	PERCENT LOSS	COR	BEFORE (TRIPS)	AFTER (TRIPS)	**PERCENT LOSS**
01_EB	**284**	**257**	**9.51%**	**01_WB**	**276**	**253**	8.33%
02_NB	**249**	**228**	**8.43%**	**02_SB**	**341**	**299**	12.32%
03_EB	**473**	**440**	**6.98%**	**03_WB**	**645**	**593**	8.06%
04_EB	**289**	**274**	**5.19%**	**04_WB**	**251**	**228**	9.16%
05_NB	**271**	**265**	**2.21%**	**05_SB**	**179**	**177**	1.12%
06_NB	**235**	**230**	**2.13%**	**06_SB**	**237**	**224**	5.49%
07_NB	**184**	**178**	**3.26%**	**07_SB**	**253**	**240**	5.14%
08_EB	**447**	**432**	**3.36%**	**08_WB**	**291**	**277**	4.81%
09_EB	**387**	**375**	**3.10%**	**09_WB**	**271**	**265**	2.21%
10_NB	**219**	**209**	**4.57%**	**10_SB**	**245**	**233**	4.90%
12_NB	**364**	**316**	**13.19%**	**12_SB**	**0**	**0**	0.00%
14_EB	**193**	**175**	**9.33%**	**14_WB**	**237**	**231**	2.53%
15_EB	**120**	**101**	**15.83%**	**15_WB**	**70**	**61**	12.86%
16_NB	**112**	**106**	**5.36%**	**16_SB**	**113**	**108**	4.42%
17_NB	**147**	**138**	**6.12%**	**17_SB**	**342**	**312**	8.77%
18_NB	**378**	**364**	**3.70%**	**18_SB**	**394**	**373**	5.33%
19_EB	**138**	**110**	**20.29%**	**19_WB**	**149**	**129**	13.42%
20_EB	**278**	**252**	**9.35%**	**20_WB**	**153**	**147**	3.92%
21_EB	**373**	**334**	**10.46%**	**21_WB**	**195**	**175**	10.26%
22_EB	**125**	**100**	**20.00%**	**22_WB**	**210**	**176**	16.19%
23_NB	**107**	**84**	**21.50%**	**23_SB**	**213**	**160**	24.88%
24_NB	**191**	**169**	**11.52%**	**24_SB**	**65**	**58**	10.77%
25_NB	**169**	**158**	**6.51%**	**25_SB**	**257**	**242**	5.84%
26_EB	**173**	**168**	**2.89%**	**26_WB**	**140**	**135**	3.57%
28_EB	**131**	**122**	**6.87%**	**28_WB**	**79**	**74**	6.33%
29_EB	**197**	**185**	**6.09%**	**29_WB**	**229**	**213**	6.99%
30_NB	**294**	**244**	**17.01%**	**30_SB**	**149**	**128**	14.09%
31_EB	**90**	**83**	**7.78%**	**31_WB**	**72**	**67**	6.94%
32_NB	**212**	**186**	**12.26%**	**32_SB**	**187**	**168**	10.16%
33_NB	**76**	**66**	**13.16%**	**33_SB**	**85**	**75**	11.76%
34_EB	**58**	**55**	**5.17%**	**34_WB**	**122**	**113**	7.38%
35_NB	**186**	**143**	**23.12%**	**35_SB**	**181**	**145**	19.89%
36_EB	**216**	**167**	**22.69%**	**36_WB**	**241**	**185**	23.24%
37_NB	**115**	**115**	**0.00%**	**37_SB**	**72**	**70**	2.78%
38_NB	**194**	**179**	**7.73%**	**38_SB**	**123**	**112**	8.94%
39_EB	**80**	**73**	**8.75%**	**39_WB**	**49**	**37**	24.49%
40_EB	**134**	**118**	**11.94%**	**40_WB**	**92**	**81**	11.96%
41_NB	**149**	**128**	**14.09%**	**41_SB**	**121**	**111**	8.26%
42_NB	**204**	**172**	**15.69%**	**42_SB**	**107**	**91**	14.95%
51_NB	**78**	**60**	**23.08%**	**51_SB**	**74**	**60**	18.92%
52_NB	**50**	**46**	**8.00%**	**52_SB**	**56**	**53**	5.36%
55_NB	**55**	**51**	**7.27%**	**55_SB**	**67**	**60**	10.45%
58_EB	**63**	**55**	**12.70%**	**58_WB**	**51**	**38**	25.49%
63_NB	**74**	**66**	**10.81%**	**59_WB**	**32**	**31**	3.13%
67_NB	**109**	**97**	**11.01%**	**67_SB**	**162**	**132**	18.52%
69_EB	**70**	**62**	**11.43%**	**69_WB**	**115**	**99**	13.91%
71_EB	**39**	**34**	**12.82%**	**71_WB**	**196**	**170**	13.27%
72_NB	**147**	**129**	**12.24%**	**72_SB**	**101**	**88**	12.87%

COR	BEFORE (TRIPS)	AFTER (TRIPS)	PERCENT LOSS	COR	BEFORE (TRIPS)	AFTER (TRIPS)	PERCENT LOSS
73_EB	124	111	10.48%	73_WB	164	139	15.24%
74_EB	48	30	37.50%	74_WB	63	17	73.02%
78_EB	0	0	0.00%	78_WB	12	9	25.00%
79_EB	0	0	0.00%	79_WB	65	55	15.38%
80_EB	52	49	5.77%	80_WB	57	56	1.75%
81_NB	58	52	10.34%	81_SB	65	55	15.38%
82_NB	88	76	13.64%	82_SB	109	100	8.26%
83_EB	0	0	0.00%	83_WB	20	18	10.00%
84_EB	140	121	13.57%	84_WB	227	177	22.03%
85_EB	88	84	4.55%	85_WB	156	155	0.64%
86_NB	95	86	9.47%	86_SB	161	151	6.21%
87_NB	220	159	27.73%	87_SB	126	118	6.35%
89_NB	180	155	13.89%	89_SB	269	246	8.55%
90_NB	82	67	18.29%	90_SB	84	65	22.62%
92_NB	108	98	9.26%	92_SB	70	52	25.71%
93_NB	40	34	15.00%	93_SB	65	50	23.08%
94_NB	179	143	20.11%	94_SB	260	230	11.54%
95_EB	37	34	8.11%	95_WB	66	59	10.61%
96_NB	27	22	18.52%	96_SB	0	0	0.00%
97_EB	73	55	24.66%	97_WB	90	73	18.89%
98_EB	100	94	6.00%	98_WB	238	209	12.18%
99_EB	30	24	20.00%	99_WB	31	25	19.35%
100_NB	136	118	13.24%	100_SB	70	55	21.43%

Table 37. Percent Data Loss after Application of GPS Signal Quality Filter

COR	BEFORE (TRIPS)	AFTER (TRIPS)	PERCENT LOSS	COR	BEFORE (TRIPS)	AFTER (TRIPS)	PERCENT LOSS
01_EB	257	138	46.30%	01_WB	253	132	47.83%
02_NB	228	153	32.89%	02_SB	299	188	37.12%
03_EB	440	343	22.05%	03_WB	593	483	18.55%
04_EB	274	221	19.34%	04_WB	228	188	17.54%
05_NB	265	128	51.70%	05_SB	177	61	65.54%
06_NB	230	209	9.13%	06_SB	224	206	8.04%
07_NB	178	149	16.29%	07_SB	240	191	20.42%
08_EB	432	409	5.32%	08_WB	277	237	14.44%
09_EB	375	359	4.27%	09_WB	265	15	94.34%
10_NB	209	199	4.78%	10_SB	233	219	6.01%
12_NB	316	286	9.49%	12_SB	0	0	0.00%
14_EB	175	150	14.29%	14_WB	231	206	10.82%
15_EB	101	71	29.70%	15_WB	61	39	36.07%
16_NB	106	99	6.60%	16_SB	108	99	8.33%
17_NB	138	109	21.01%	17_SB	312	221	29.17%
18_NB	364	338	7.14%	18_SB	373	363	2.68%
19_EB	110	58	47.27%	19_WB	129	60	53.49%
20_EB	252	228	9.52%	20_WB	147	137	6.80%
21_EB	334	227	32.04%	21_WB	175	135	22.86%
22_EB	100	62	38.00%	22_WB	176	112	36.36%
23_NB	84	63	25.00%	23_SB	160	128	20.00%
24_NB	169	127	24.85%	24_SB	58	42	27.59%
25_NB	158	156	1.27%	25_SB	242	235	2.89%
26_EB	168	160	4.76%	26_WB	135	132	2.22%
28_EB	122	118	3.28%	28_WB	74	72	2.70%
29_EB	185	180	2.70%	29_WB	213	210	1.41%
30_NB	244	242	0.82%	30_SB	128	122	4.69%
31_EB	83	77	7.23%	31_WB	67	64	4.48%
32_NB	186	144	22.58%	32_SB	168	142	15.48%
33_NB	66	38	42.42%	33_SB	75	54	28.00%
34_EB	55	43	21.82%	34_WB	113	95	15.93%
35_NB	143	118	17.48%	35_SB	145	129	11.03%
36_EB	167	163	2.40%	36_WB	185	171	7.57%
37_NB	115	51	55.65%	37_SB	70	15	78.57%
38_NB	179	115	35.75%	38_SB	112	61	45.54%
39_EB	73	46	36.99%	39_WB	37	22	40.54%
40_EB	118	67	43.22%	40_WB	81	49	39.51%
41_NB	128	101	21.09%	41_SB	111	74	33.33%
42_NB	172	170	1.16%	42_SB	91	89	2.20%
51_NB	60	54	10.00%	51_SB	60	56	6.67%
52_NB	46	44	4.35%	52_SB	53	52	1.89%
55_NB	51	45	11.76%	55_SB	60	54	10.00%
58_EB	55	48	12.73%	58_WB	38	35	7.89%
63_NB	66	64	3.03%	59_WB	31	25	19.35%
67_NB	97	32	67.01%	67_SB	132	30	77.27%
69_EB	62	60	3.23%	69_WB	99	93	6.06%
71_EB	34	33	2.94%	71_WB	170	164	3.53%
72_NB	129	43	66.67%	72_SB	88	31	64.77%

COR	BEFORE (TRIPS)	AFTER (TRIPS)	PERCENT LOSS	COR	BEFORE (TRIPS)	AFTER (TRIPS)	**PERCENT LOSS**
73_EB	**111**	**61**	**45.05%**	**73_WB**	**139**	**103**	25.90%
74_EB	**30**	**15**	**50.00%**	**74_WB**	**17**	**15**	11.76%
78_WB	**0**	**0**	**0.00%**	**78_WB**	**9**	**8**	11.11%
79_WB	**0**	**0**	**0.00%**	**79_WB**	**55**	**20**	63.64%
80_EB	**49**	**44**	**10.20%**	**80_WB**	**56**	**49**	12.50%
81_NB	**52**	**41**	**21.15%**	**81_SB**	**55**	**47**	14.55%
82_NB	**76**	**73**	**3.95%**	**82_SB**	**100**	**82**	18.00%
84_EB	**121**	**83**	**31.40%**	**83_WB**	**18**	**16**	11.11%
85_EB	**84**	**83**	**1.19%**	**84_WB**	**177**	**122**	31.07%
86_NB	**86**	**85**	**1.16%**	**85_WB**	**155**	**154**	0.65%
87_NB	**159**	**130**	**18.24%**	**86_SB**	**151**	**139**	7.95%
89_NB	**155**	**142**	**8.39%**	**87_SB**	**118**	**89**	24.58%
90_NB	**67**	**65**	**2.99%**	**89_SB**	**246**	**214**	13.01%
91_NB	**0**	**0**	**0.00%**	**90_SB**	**65**	**62**	4.62%
92_NB	**98**	**71**	**27.55%**	**92_SB**	**52**	**42**	19.23%
93_NB	**34**	**23**	**32.35%**	**93_SB**	**50**	**32**	36.00%
94_NB	**143**	**82**	**42.66%**	**94_SB**	**230**	**160**	30.43%
95_EB	**34**	**32**	**5.88%**	**95_WB**	**59**	**57**	3.39%
96_NB	**22**	**11**	**50.00%**	**96_SB**	**0**	**0**	0.00%
97_EB	**55**	**17**	**69.09%**	**97_WB**	**73**	**11**	84.93%
98_EB	**94**	**61**	**35.11%**	**98_WB**	**209**	**162**	22.49%
99_EB	**24**	**19**	**20.83%**	**99_WB**	**25**	**21**	16.00%
100_NB	118	94	20.34%	100_SB	55	45	18.18%