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OPERATIONAL ANALYSIS OF TRADITIONAL ACCESS MANAGEMENT STRATEGIES & DEMAND-RESPONSIVE ACCESS CONTROL ON ARTERIALS

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ABSTRACT

Arterials are typically characterized by closely-spaced signalized intersections, high driveway density, and high traffic volumes. These characteristics contribute to congestion, as well as crashes. Access management strategies can address both operational and safety issues on urban arterials. This research focuses on the operational impacts of access management with two objectives: (1) quantify the impacts of 'traditional' access management strategies and (2) quantify the impacts of demand-responsive access control. To satisfy Objective 1, four traditional access management strategies were tested – (i) access spacing, (ii) corner clearance, (iii) access restriction, and (iv) raised median implementation. These were analyzed in four respective alternative scenarios using microscopic simulation (VISSIM) of two existing corridors; one 5-lane and one 7-lane and measures of effectiveness (MOEs) of mainline travel times and driveway ingress and egress traffic total and stopped delay were compared. The analysis revealed that operational impacts of traditional access management techniques are site-specific. However, considering both sites, the access spacing strategy, which consolidates driveways such that they achieve the SCDOT ARMS Manual spacing requirements, performed best from the standpoint of the MOE's observed and is most recommended for implementation.

In order to test demand-responsive access control for Objective 2, simulation of the same two existing corridors used for traditional access management tests was conducted for a period including both peak and off-peak traffic conditions for three scenarios (i) existing conditions, (ii) a raised median (permanent access control), and (iii) dynamic access control, which includes restriction of driveways to right-in, right-out enforced

during intervals in which traffic volumes exceed given thresholds. Simulation analysis indicated that while the raised median performed differently on each corridor, the demand-responsive strategy lowered travel times and delays. Therefore, it is the conclusion of this research that alternating access between fully-open to right-in/right-out based on prevailing traffic conditions, has the potential to improve traffic operations on a corridor, by producing lower travel times and delays during both peak and off-peak traffic conditions.

DEDICATION

To my friends and family. I love you.

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CHAPTER ONE

INTRODUCTION

1.1. Introduction and Problem Statement

Urban arterials are typically characterized by closely-spaced signalized intersections, high driveway density, and high traffic volumes *(1)*. These characteristics contribute to high urban arterial crash rates and severities, over 50% of which are access-related *(2)*. In addition to safety issues, urban arterials also experience high levels of congestion, travel times, and delays. Access management, "the coordinated planning, regulation, and design of access between roadways and land development" *(3)*, is an integrated approach that can be used to alleviate both the safety and operational issues on urban arterials. Access management techniques make provisions for signal spacing, driveway spacing, turning movement restrictions, corner clearance, auxiliary lanes, and median treatment alternatives, among others *(4)*. These techniques have safety, operational, and economic impacts on corridors in which they are implemented as well as on surrounding areas.

The safety benefits of access management strategies are widely documented and accepted with little contention. For example, multiple statewide studies have indicated that crash rates tend to increase as access density increases *(3)*. Roadways with non-traversable medians have also been shown to have lower crash rates than those with two-way-left-turn-lanes (TWLTL) and those that are undivided *(3).* There is slightly more ambiguity, however, concerning operational and economic impacts, which has led to a growing interest in quantifying these impacts in order to provide a more holistic justification for the implementation of various access management measures. Such an interest led to the focus of this research: the operational impact on urban/suburban arterials of a selected variety of access management techniques, including (i) Access Spacing, (ii) Corner Clearances, (iii) Access Restriction of Selected Driveways, and (iv) Non-Traversable Medians. While there are previous studies focusing on different operational elements of these strategies, there is still an interest at the SCDOT level concerning the operational impacts of these strategies in a corridor-wide implementation on South Carolina arterials. As stated below in the research objectives, addressing this issue is Objective (1) of this thesis.

Among the aforementioned techniques, prohibiting direct left turns (DLT) from driveways in favor of right-turn-U-turn (RTUT) movements has been widely studied and recommended in the literature. A number of studies have investigated the operational and safety impacts of DLT alternative movements, and many of them have concluded that their impacts vary according to traffic conditions. According to one study, within a certain range of arterial volume, DLT movements are advantageous over RTUT movements from an average network delay standpoint *(10)*. Another study noted that as the volumes of through traffic and left turns from driveways increase, RTUT movements resulted in substantially less delay than DLT movements *(11)*. Another study found the range of arterial volumes at which restricting access to right-in-right-out becomes advantageous *(21)*. While these past research efforts have found volume thresholds that would make access management strategies effective, they have not considered the effect of dynamic strategies that would change access restrictions according to prevailing traffic conditions in order to optimize travel times and delays. To this end, this thesis, in addition to the first objective, seeks to answer the following research question: *how would dynamic, demand-responsive management of access point movements impact the operational performance of an urban arterial?* These two research objectives are shown in the following section in listed form.

1.2. Research Objectives

There are two objectives of this research:

- First, to quantify and compare the operational impacts of traditional access management strategies (those listed in the previous subsection) on arterials and,
- Second, to quantify and compare operational impacts of demand-responsive access control with permanent access control and no access control conditions.

1.3. Potential Benefits of This Research

This research will quantify of the impacts of four (4) access management techniques in a corridor-wide implementation, allowing for a comparison of the effectiveness of each, in a case-study basis. The potential benefits of the satisfying the first objective are for the South Carolina Department of Transportation, as well as other state transportation agencies and professionals, to gain an insight into the possible operational impacts of raised medians, providing adequate driveway spacing through the consolidation of driveways, providing adequate corner clearance, and selecting certain driveways to be right-in/right-out. The potential benefits of satisfying the second objective, is an understanding of the impacts alternating restrictions of driveways along a corridor could have on travel times and delays during both off-peak and peak hours. In other words, it may begin to answer the question of whether a system could be optimized over the varying traffic conditions it experiences throughout a typical day by alternating when accesses are restricted and when they are not.

1.4. Thesis Organization

This thesis is organized into five (5) chapters. Chapter 2 provides a review national guidelines and resources, state of the art (literature), and state of the practice (state agency guidelines) as they relate to the operational impacts and design of the aforementioned access management strategies in question. Chapter 3 is divided into two sections, each corresponding to one of the objectives of this research, and discusses the research methodology used, including base model development, how access management strategies were tested in alternative scenario models, and methods of analysis. Chapter 4 is likewise divided into two sections, and discusses the results of the analysis for each objective. Chapter 5 concludes the paper with a discussion of conclusions and recommendations based on the analysis.

CHAPTER TWO

LITERATURE REVIEW

The literature review chapter is divided into three sections. The first is a review of national guidelines and resources that discuss the operational impacts of access management. The second is a review of relevant literature, with three focus areas as follows:

(1) Methods used to analyze the operational impacts of access management

(2) Findings as they relate to operational measures of effectiveness

(3) Design recommendations relevant to the testing of such strategies in this thesis' research.

The third and final section is a review of current state agency manuals regarding warrants and design guidelines for the access management strategies that are the focus of this research. Many states provide such guidelines for a wide spectrum of roadway types and characteristics. Therefore, for comparability and brevity, only those warrants and guidelines pertaining to roadways with characteristics similar to the ones tested in this research (principal/minor arterials with 45 mph speed limits) are presented.

Each section concludes with a summary of noteworthy conclusions and trends gleaned from the review prior.

As stated earlier, the access management strategies studied in this research are (i) Access Spacing, (ii) Corner Clearances, (iii) Access Restriction of Selected Driveways, and (iv) Non-Traversable Medians. Definitions of these terms (as given in the TRB Access Management Manual, 2014) are provided below, for the sake of clarity *(3)*:

<u>Access Spacing</u> — *The distance between adjacent private driveways, between adjacent public roadways, or between a public roadway and a private driveway. It is measured from centerline to centerline or near edge to near edge of the access connections according to agency practice.*

<u>Corner Clearance</u> — *The distance from an intersection of a public or private road to the nearest access connection, measured from the closest edge of the pavement of the intersection road to the closest edge of the pavement of the connection along the traveled way.*

<u>Access Restriction</u> — *Using channelization in a driveway throat, at its intersection with the public road, to restrict left-turn movements into or out of the driveway.*

<u>Non-Traversable Median</u> — *A divider that separates opposing traffic streams. The medians design actively discourages or prevents vehicles from crossing the divider. A non-traversable effectively restricts access at driveways to right-in/right-out except at those with median openings.*

The first section of the literature review begins on the following page. The relevant information from the reviewed national guidelines and resources as they apply to the four access management strategies are presented. It should be noted that while these documents have much to say in many different areas of access management design principles, only those relevant to this research are presented.

2.1. Review of National Guidelines and Resources

2.1.1. TRB Access Management Manual (3) – The TRB Manual is a synthesis of policy, warrant, and design information from national studies, peer-reviewed research, and state practice. The ways in which it speaks to the strategies of consideration in this thesis are presented below.

Access Spacing – Average driveway entry speeds are typically between 8 and 13 mph, creating high speed differentials occur in advance of the location where a turning maneuver is executed. Proper spacing of access points is critical for safe and efficient operation of an arterial. Poor spacing, design, and location of driveways can reduce average speeds by up to 5 to 10 mph. Spacing criteria has been addressed in a number of different methods. These methods, and the resulting suggested spacing for a 45 mph roadway [ft] are as follows. (1) *Independent access connections* – defining spacing based on the upstream and downstream functional distances from adjacent access points – this tends to lead to long and typically unreasonable access spacings [1,045 ft.]; (2) *Upstream functional distance* – defines the spacing by the upstream functional distance only [280-410 ft. – depending on functional distance calculation method]; (3) *Turn lane design* – defines the spacing such that it is larger than the right-turn auxiliary lane length so that there is no overlap between driveways and the lane [369 ft.]; (4) *Safety*; (5) *Stopping sight distance* – spaces access at distances equal to or longer than the SSD [360 ft.]; (6) *Intersection sight distance* – bases the spacing on the distance needed to provide a driver waiting at an access an opportunity to enter or cross the major roadway [430-500 ft.]; (7) *Decision sight distance* – spaces access in terms of the sight distance from the perspective of the driver traveling on the roadway [395-960 ft. – depending on maneuver]; (8) *Right-turn conflict overlap* – spaces access such that a driver on the mainline does not have to monitor more than one right-turn ingress movement at a time [350 ft.]; and (9) *Egress capacity* – spaces access such that the egress capacity of driveways is maximized [870 ft.]. Depending on the approach employed, recommended unsignalized access spacings (for a 45 mph roadway) range from 280 to 1,045 ft.

Corner Clearance – Driveways should not be located within the functional area of an intersection or in the influence area of another driveway. When an access connection within the functional distance cannot be avoided, movements should be restricted to right-in/right-out only. Having adequate corner clearance improves signal capacity and safety. For a 40-50 mph design speed, the recommended minimum upstream and downstream corner clearance is 410-585 ft. and 360 ft. respectively.

Nontraversable Medians – Nontraversable medians are recommended for implementation on major roadways in new locations, existing major roadways with current or projected ADT in excess of 24,000 to 28,000 vehicles, undivided roadways and roadways with a TWLTL on which operational or safety problems are evident, and generally on roadways of four or more lanes. Nontraversable medians drastically reduce conflict points, leading to improved safety. The TRB Manual heavily recommended using directional median openings as opposed to full median openings, as they further reduce conflict points and reduce crashes. The distance needed between signals to accommodate directional median openings is determined the sum of length of turn bays at the signals, turn bays at the directional openings, and minimum width of full median width. The TRB Manual also presents median separator widths needed for U-turn movements. For a passenger car (P) on a four-lane road with a dedicated left-turn lane, a median width of 30 ft. is required. On a six-lane road with a dedicated left-turn lane, a median width of 18 ft. is required.

2.1.2. *NCHRP Report 420 – Impacts of Access Management Techniques (5)* is a comprehensive review of the impacts of a wide range of strategies. Three policy-related techniques and 21 design-related techniques were identified. Of these strategies, establishing spacing for unsignalized access, establishing corner clearance criteria, and replacing TWLTLs with nontraversable medians, and installing U-turns as alternatives to direct left turns were all ranked in the highest category of importance to access management. Consolidating driveways was rated as medium importance. The key conclusions from this report as they relate to the strategies of consideration in this thesis are presented below.

Access Spacing – One general finding of the report was that an increase in the number of access points translates to higher accident rates. Operationally, the report references the 1994 HCM which shows a reduction of 2.5 mph in free-flow speed with every additional 10 access points per mile. Another referenced study in the report showed a speed reduction of 0.15 mph per access point.

Nontraversable Medians – The safety finding is that raised medians have reduced crash rates when compared to TWLTL and Undivided highways and replacing direct left turns with U-turn movements can result in a 20 % accident reduction rate. The report notes that most operational analysis (at the time of writing) has focused on TWLTLs. Various studies cited in the report show that TWLTLs generally result in lower delays than raised medians, however the differences are not statistically significant. The travel time impacts of providing U-turns as direct-left-turn (DLT) alternatives were studied and presented. It is estimated that when arterial traffic exceeds 375 to 500 vphpl on a four-lane facility, the delays of direct left turning traffic exceed those of the alternative right-turn-U-turn (RTUT) traffic. In general, the report claims that RTUT movements can provide comparable, in not shorter, travel times than direct left turns from driveways under heavy volume conditions when the diversion distances are generally less than 0.5 miles.

2.1.3. *NCHRP Report 524 – Safety of U-Turns at Unsignalized Median Openings (6)* concluded that there was no indication that U-turns at unsignalized median openings constitute a major safety concern. Additionally, there was no indication that safety problems result from the occasional use of median opening spacings as short as 300 to 500 ft.

2.1.4. *NCHRP Report 348 – Access Management Guidelines for Activity Centers (7)* defines the concept of access management, reviews current practice, and sets forth policy, planning, and design guidelines for spacing standards, design concepts, and criteria. The report states that driveways should be located opposite other access or street and placed beyond normal backups of traffic from signalized intersections. It is recommended closing/relocating driveways within 100 ft. from a signalized driveway. The general guidelines for unsignalized access spacing present spacings of 300-550 ft. for 45 mph roadways, and 300-800 ft. on roadways with ADT volumes of 1,500 or more. The report also recommends median opening spacing of 670 ft. for 45 mph roadways.

2.1.5. *TRC 456 – Driveway and Street Intersection Spacing (8)* presents general considerations for establishing spacing criteria. These considerations are very similar to the nine (9) presented in the TRB Access Management Manual, which were discussed prior.

Summary of National Guidelines and Resources

There is a general consensus that increased spacing of driveways (and corner clearances) is both safer and more operationally efficient. The suggested values for these spacings vary by source and by the approach used to determining them. There is also general agreement that there is no indication that right turns followed by U-turns provide an increased safety risk as opposed to direct left turns and that they can lead to improved travel times for turning vehicles. While there are design guidelines presented for channelization of driveways, there do not seem to be, in the national guidelines, suggestions for when to restrict access to right-in-right-out.

2.2. Review of State of the Art

The purpose of this section of the literature review is to provide an overview of research methodology, findings of said research, and any design guidelines, simulation parameters, and/or other recommendations from past research relevant to the research of this thesis. This section is divided into five (5) subparts, each addressing a distinct access management strategy or other element of the research. At the end of this section, there will be a summary of the literature review summarizing the main findings from the review.

2.2.1. Nontraversable (Raised Medians)

Eisele et al. (2005) *(9)* investigated the impacts of raised medians on travel time, speed, and delay. The authors performed micro-simulation in VISSIM (and signal optimization in SYNCHRO) on three existing corridors and three theoretical corridors with different driveway spacings, median treatments, and traffic volumes. The three test corridors ranged in length, signal and access density, median opening spacing, number of lanes, existing ADT, and estimated future ADT. The theoretical corridors were given different lane, driveway density, driveway spacing, and estimated future ADT characteristics to study the effects of these variables on the MOE's (time, speed, and delay). Both 2-lane and 3-lane (in each direction) scenarios were tested, and the ATD of the simulated corridors ranged from 18,000 to 48,000, the raised median opening spacing tested was 660 ft., and the driveway spacing tested ranged from 165 ft. to 660 ft. In all theoretical corridors, there were an equal number of driveways on both sides of the road, driveway centerlines were aligned, trips generated from the driveways were estimated from the ITE Trip Generation Manual, and the trips entering and exiting driveways were equally divided between left-turning and right-turning movements. Results from simulation of the existing corridors showed differing travel time effects for each corridor, revealing access management impacts to be case specific. For the lowest length corridor, decreases in travel times were found for both low and high ADT levels tested. For the second-longest, and longest corridors studied, however, travel times were shown to increase with the addition of the raised median. Results from the theoretical corridor simulation studies showed a general increase in travel time for through moving vehicles with the addition of the raised median, with an average reduction in speed of 3 mph. The author's explained that this increase in travel time (and decrease in speed) with the addition of raised medians was due to more U-turn traffic at signalized intersections as well as added through volume traffic from right-turn-U-turn movements.

Chowdhury et al. (2005) *(10)* studied the effect of different left turn treatment alternatives on network-wide average delay per vehicle. Microsimulation in CORSIM & signal optimization in SYNCHRO was used to analyze the alternative scenarios. The sites analyzed were a combination of divided, undivided, and 2-lane roads, each having signalized intersections on either end, and unsignalized driveways leading to major traffic generators exiting onto the main road. The five alternatives to direct left turns analyzed were (1) No restriction of direct left turns, (2) No direct left turns in or out of driveways with diverted traffic making a U-turn at the next available intersection, (3) No direct left turns in or out of driveways with diverted traffic making a U-turn at the mid-block, (4) Use of a jughandle left-turn at the signalized intersection to accommodate left turns, and (5) No direct left turns except for on one driveway consisting of a concentration of all driveway volume. Each classification of roadway and alternative was analyzed for varying levels of mainline and driveway volumes. In general, it was found that increases in mainline volume had a far greater impact on network wide average delay per vehicle than increases in driveway volume. For multilane divided highways, the direct left-turn alternative was preferable until the 650 vphpl volume threshold was reached, beyond which, the RTUT with U-turns occurring at nearest signalized intersections became preferable. The concentrated left turn treatment performed very well operationally, and was therefore recommended where the existence of internal circulation allows for its implementation. Overall, the study found the operational differences between direct-left-turn movements and the U-turn alternative movements to be negligible, and that operational impacts need to be assessed on a site-by-site basis.

<u>Zhou et al. (2002)</u> *(11)* studied the operational effects of U-turns as alternatives to direct left turns from driveways. Field data was collected using video cameras at eight study sites (all 6-lane sites with signal spacing less than 2-miles) in order to compare the delay experienced by direct left turning (DLT) and that of right-turn-U-turn (RTUT) vehicles. From this data, two exponential regression equations for total delay and two exponential regression equations for travel time were developed for the DLT and RTUT movements respectively. For the DLT equation, regression variables included through volume, left-turn volume, left-turn-in volume, and the SPLIT (distribution of through volume in either direction). For the RTUT equation, regression variables included through volume, RTUT flow rate, speed, and the SPLIT. Curves for varying roadway characteristics can be developed from these equations to estimate delay and travel times of DLT and RTUT vehicles. Based on an overview of these curves, it can be demonstrated that U-turns can have better operational performance than direct-left-turns under certain traffic conditions.

<u>Liu et al. (2007)</u> *(12)* studied the operational effects of U-turns as alternatives to direct left turns using delay and travel time as measures of effectiveness. The study also examined the average running time for vehicles making right-turn U-turn left turns at variously separation distances between driveways and U-turn locations. Field data was collected at 34 roadway segments in central Florida to analyze delay and travel time data for three left turn alternatives: (1) Direct left-turns, (2) Right-turns followed by U-turns at *median* openings, and (3) Right-turns followed by U-turns at *signalized* intersections. Results from the study for the first and primary objective showed that with the increase of driveway and major road through volumes, delay for direct left-turns increases, and the delay from a right-turn-U-turn movement can be 1-3 seconds less on average as these volumes increase. In short, the higher the roadway volumes, the more attractive the right-turn-U-turn at a median alternative is from a delay standpoint. Regardless of the volumes on the road, vehicles making right-turn-U-turns at signalized intersections experienced more delay than the other two alternatives. On average over all 34 segments, the median U-turn alternative performed the best from a delay perspective, with the direct left turn being a close second, and the signal U-turn being a distant third. Results from the study for the second objective created a travel time (of left-turning alternative movements) comparison graph linking separation distance with total travel time. The travel time of vehicles making U-turns at signalized intersections far exceeded those of direct left-turners and vehicles making U-turns at mid-block median openings.

Yang and Zhou (2004) *(13)* to evaluated the delay and travel time of direct-left-turns versus right-turn-U-turn movements using a CORSIM-based simulation approach. Data was collected from 6 existing sites in order to calibrate the simulation model, which was then used to estimate delays and travel times for DLT and RTUT movements at varying levels of driveway volume (150-350 vph) and two-way through volume (3000-7000 vph). Resulting curves for delay and travel time were generated for each site-based model for a total of 6-sets of curves. From these curves, breakpoints (points at which RTUT movements experienced favorable travel times/delays) could be determined for the different driveway and through volume thresholds. While these breakpoints vary by site, the general trend observed was that the lower the driveway volumes, the higher the mainline through volume at the breakpoint, and vice versa.

Reid and Hummer (1999) *(14)* compared traffic operations along a typical arterial under two-way-left-turn-late (TWLTL), Median U-turn Crossover (MUT), and Super-Street Median Crossover (SSM) design using microsimulation in CORSIM. The ITE Trip Generation Manual was used to assign trip rates for driveways along the corridor, and these trip rates were kept constant between each of the three scenarios tested. Four time periods (morning-peak, noon, mid-day, and afternoon peak hour) were tested, with each time period having varying driveway and through-trip intensities. SYNCHRO was used to optimize signal timings, and the same set of random number seeds were used for each scenario for uniformity. The results of the simulation runs show that while the TWLTL scenario had fewer average stops per vehicle than the MUT and SSM scenarios, it had a higher system travel time and average speed. The MUT performed best in these categories on average. When considering the four different time periods analyzed, the results showed that the MUT and SSM scenarios outperformed the TWLTL in peak hours but also performed similarly to the TWLTL in off-peak hours. In other words, this research found that the alternative designs did not compromise travel times during off-peak hours.

Shadewald et al. (2003) *(15)* studied the effects of varying access control improvements on a test-corridor using total delay (sec/veh), travel time (VHT), speed (mph), and fuel efficiency (MPG) as measures of effectiveness. Synchro and Netsim were used to model the different scenarios, which included (1) Existing Conditions: 40 access points/mile, no center median, 5 signalized intersections, (2) Improved Access-Controlled Alternative: 25 access points/mile, addition of center median, addition of backage road, and (3) Full Access-Controlled Alternative: 10 access points/mile, fully center median controlled, backage roads. Driveway trips were estimated using the ITE Trip Generation Manual. The results from the study showed that the Improved and Full Access Control reduced total delay and travel time, while increasing fuel efficiency and speed. The improved access scenario (2) increased capacity by 25-45 percent, decreased total delay by 65-170 seconds per vehicle, decreased stop delay by 100-200 seconds per vehicle, and increased speeds by 20-33 percent. The full access-controlled scenario (3) increased capacity by 50-100 percent, decreased total and stop delay per vehicle by 83-91 percent, and increased speeds by 14-24 mph, while reducing fuel consumption by 30-40 percent. An important note about this study is that right-of-way and feasibility of altering and/or constructing new backage roads was not considered.

Lu et al. (2005) *(16)* proposed minimum acceptable offset distances for vehicles making right-turns followed by U-turns on 4-lane and 6-lane urban/suburban multilane divided arterials, with offset distance defined as the separation distance between the driveway exit and downstream median opening or signalized intersection at which the U-turn will take place. Determination of the minimum offset distances was made by taking into account crash analysis, conflict analysis, and operations analysis of 68 field sites. The minimum offset distances recommended by the study varied by U-turn location (median opening vs. signalized intersection) *and* by the number of lanes (4 vs. 6 or more). The resulting recommended offset distances are shown below:

U-turn Location	Number of Lanes	Offset Distance (ft.)
Median Opening	4	400
	6 or more	500
Signalized Intersection	4	550
	6 or more	750

Carter et al. (2005) *(17)* investigated the operational and safety effects of U-turns at signalized intersections. The operational impacts were estimated by quantifying U-turn behavior at 14 sites with exclusive left-turn lanes and protected phasing. The research team collected saturation headway measurements and volume counts at each site in order to develop a regression equation to predict a saturation flow adjustment factor in terms of U-turn percentage and the existence of conflicting right-turn protected overlap, which were both found to be statistically significant regression variables. This resulting regression equation showed a 1.8% saturation flow rate loss for every 10% increase in average U-turn percentage, with an additional 1.5% loss per 10% U-turns where there is an opposing protected-right-turn overlap from the cross-street. The safety impacts were estimated by analyzing the history of collisions involving U-turns at 78 sites. The crash analysis indicated that 65 of 78 sites had no collisions involving U-turns in the 3-year study period, and the sites that did have collisions had crash rates ranging from 0.33 to 3.0 collisions per year. Overall, the study found that both operationally and safety-wise, U-turns *do not have a large negative effect* at signalized intersections, with minimal crash histories involving U-turns and only 1.5s of increased stopped delay per 10% increase in U-turns. However, a conclusion of note from the study was that protected right-turn overlap on the cross street does have a negative effect both operationally and safety-wise in intersections where U-turns are allowed/prevalent.

<u>Qi et al. (2013)</u> *(18)* developed guidelines for operationally effective raised medians and alterative movements on urban roadways. The critical design issues addressed included median widths, median left-turn lane lengths, placement of median openings, and directional vs. full median openings. The study was performed by reviewing national and peer-reviewed literature, conducting a nation-wide survey of traffic engineers, conducting field studies, and performing simulation analysis. An overarching finding from the research was that there were fewer existing research initiatives relating to the operations of raised medians than there were concerning their safety. Additionally, the existing research seemed to be inconclusive about whether raised medians were more operationally favorable to TWLTLs as there are a plethora of factors influencing their effectiveness. The research also found directional medians within an intersection influence area to be less favorable than full median openings from an operational standpoint. The guidelines developed from the initiative were: (1) An ADT greater than 20,000 vpd warrants consideration of implementing a raised median; (2) Typical median width should be at least 16 ft., however on roadways allowing U-turns, widths need to be wider to accommodate the design vehicle. The authors developed recommended minimum median widths and necessary right-of-way (ROW) in order to provide adequate space for U-turn movements based on a swept path analysis. Based on this analysis, for the passenger car design vehicle (P), the minimum median width on a four lane road with a dedicated left-turn lane is 30 ft., and the necessary right-of-way for the road is 100 ft; (3) Median openings should be placed to provide openings at all public roads and major traffic generators, and additional openings should be provided so as to not exceed 2,640 ft. to minimize travel distance for right-turn-U-turn movements. (4) Median opening lengths should be at least 40 ft. (5) Lengths of deceleration lanes at median openings should be determined depending on speed and assumed speed differential. The operational impacts of shorter-than-approved left-turn lanes were found to be minimal in isolated instances. However, where short left-turn lanes were used successively on a corridor, negative impacts compounded; (6) Median left-turn lanes should be considered according to previously established left-turn lane warrants; and (7) Full median openings are recommended under most circumstances, though directional median openings can be considered as replacement if the opening is in the influence area of an intersection.

Chowdhury et al. (2004) *(76)* conducted a survey aimed at determining the state of knowledge and practice in providing alternatives to direct-left turns. A survey was developed and sent to all 50 states, with responses received from half (25) of them. The survey results provided a basis for an ongoing inventory of current practices at the State Agency level. Results from the survey indicated that most states did not have formal policies or guidelines for restricting direct-left-turn movements and/or for providing alternative movements for left-turn deterred traffic in the case of restricting such movements. Instead, it was found that most states handle these situations on a case-by-case basis, likely due to the fact that there is no national standard in place for prohibiting direct-left-turn movements. When these movements are accommodated, the majority of states prefer mid-block U-turns or Jughandles. The survey study concluded that there were a lack of standards at the state agency level concerning restriction of direct left-turns and how to accommodate deterred direct left-turn traffic. The paper also recommends additional research towards the end of developing national policies and guidelines for these access management strategies.

2.2.2. Access Density, Restriction, and Corner Clearance

Siddiqui (2011) *(19)* investigated the operational impacts of access modifications at midblock and corner driveways on 5-lane roads with a TWLTL. Microsimulation in VISSIM (with signals optimized in Synchro) was used to model 142 different theoretical models (calibrated from a field-studied road model) with varying driveway location (midblock, corner) density (0-44 access points /mile), and restrictions (full access, right-in/right-out, combination of both) while also varying mainline volumes (1500, 1700, and 1900 vph – each direction) and driveway volumes (25 to 200 vph). The main finding of the research was that mainline volume has a much greater effect on driveway operations than on increased driveway density. In other words, cases with high access density and high driveway volume, but low mainline volume did not have significant impacts on driveway delays.

Gluck et al. (1999) *(20)* investigated the relation of traffic operations to access spacing by conducting observational analysis at 22 sites in the Northeastern United States. Researchers recorded the number and percentage of through vehicles that were impacted by right turns at unsignalized driveways for major traffic generators without deceleration lanes in order to estimate the percent of right lane through vehicles impacted by the right-turn-in movement as a function of right-turn-in volume. A linear fit of the data revealed that about that the percentage of right lane through vehicles impacted was roughly 0.18 times right-turn-in volume. A cumulative distribution of impact length curve was prepared from the data and multiplied by the percent of right-lane through vehicles impacted by right-turn-in movements to yield cumulative frequency distribution curves of impact lengths that show the percentage of through vehicles impacted by right-turn-in movements for varying levels of right-turn-in volume at different distances from a driveway. These curves were then shifted to account for additional influence length (which included the car length and perception reaction distance) to yield curves for different levels of right-turn-in volume showing the percentage of cars impacted according to different influence lengths. These curves were then used to propose spacing guidelines for driveways according to both right-turn-in volume and spillback percentage (percent of impacted vehicles) allowed. For example, on a roadway with a 45 mph speed limit, driveways with right-turn-in volume less than 30 vph, and a 10% allowable spillback rate, a driveway spacing of 270 feet is proposed. The proposed guidelines were compared to existing state guidelines and found to fall within acceptable ranges.

Lyles et al. (2009) *(21)* conducted a simulation study (in VISSIM) to assess traffic flow impacts of right-in/right-out treatments and develop guidelines for when such strategies should be implemented. A total of eight models were developed and simulated (6 simulating corner driveways and 2 simulating mid-block driveways). In each model, four variables were varied to determine their impact on right-in/right-out restricted driveways: Corner Clearance (150-350 ft.), Mainline Volume (250-2000 vph), Driveway Volume (25-150 vph), and left-turn-in and –out volume (10-50 vph). In each model, 5 access control scenarios were tested: (1) no driveway, (2) right-turn-in only, (3) right-in/right-out, (4) right-in/right-out and left-turn-in, and (5) full access. Each model was calibrated to a field-observed site using average travel time and queue length. For changes in mainline volume, volume was assumed to change in both directions of travel but not at the other intersection approaches. Resulting U-turning traffic from access restriction was ignored in these tests, and assumed to leave the network in the direction that it exited the driveway in question. The measures of effectiveness in this study were average delay (sec/veh) for mainline traffic, average delay (sec/veh) for left-turn-in and –out traffic, and 50th percentile queue length. These measures were expressed in individual plots according to the different aforementioned variables. The main finding of the research was that increases in mainline volume had a greater impact on average delay/queue length for mainline traffic than increases in driveway volume. It was also found that impacts of increases in mainline, driveway, and left-turn volume were greater when corner clearance was less than 150 feet. Additionally, it was found that the delay for left-out traffic was greater than delay for left-in traffic, and that the impact of driveway volume on average delay was greater as the mainline volume approached 1500 vph. Another key contribution of this research were guidelines/thresholds for implementing certain access restrictions. For both corner and mid-block driveways, it was recommended that left-ins and left-outs be restricted when mainline volume is greater than 1500 vph. Additional provisions for restricting these movements for mainline volumes less than 1500 vph included when corner clearance is less than 100 feet, driveway volume is greater than 150 vph, and left-turn-in/out volume greater than 50 vph.

Gan and Long (1997) *(22)* highlighted key operational effects due to inadequate driveway corner clearances. These problems include: (1) blockage of driveway egress movement, (2) blockage of driveway ingress movement, (3) incomplete turning maneuvers in left-turn lanes, (4) conflict with intersection turning movements, (5) dual interpretations of right-turn signals, (6) merging bay vehicular conflict and reduced merging length, (7) insufficient weaving section length, and (8) emerging vehicular conflicts from driveways on right-turn bays. Driveway and intersection capacity are also negatively affected by inadequate corner clearance in that adequate gaps in platoons are not available for driveway egress traffic and right-turn egress from driveways in the functional area of the intersection reduces the saturation flow rate in the intersection.

Long and Gan (1997) *(23)* in a companion study to the one previously referenced, developed a model for determining minimum allowable corner clearances, similar to that in the HCM for computing saturation flow rates, in which an initial MCC (minimum corner clearance) is adjusted according 9 distinct site-specific factors (i.e. facility type, median type, driveway traffic volume etc.). This model makes up for deficiencies in existing models which are rigid, discrete, and provided for little consideration of the many different driveway design features. The model also allowed for MCCs relative to unsaturated and saturated flow conditions.

Prassas and Chang (2000) *(24)* investigated the effect of arterial volume, driveway volume, and driveway interactions as measured by average speed, driveway delay, and driveway queuing. The CORSIM simulation study modeled single driveway and multiple-driveway scenarios to determine the effect of upstream and downstream driveways on each other. These studies found that – when compared to the single driveway case – as the number of driveways increases, the negative effects on the MOE's increases by a factor of 2 (for two driveways) and by a factor of 4 to 5 (for three driveways). Additionally, it was found that the addition of downstream driveways reduced driveway capacity of the first upstream driveway by 30-50%. Conversely, the downstream driveways showed improved capacity – when compared to the single driveway case – due to a sheltering effect at the upstream driveway.

2.2.3. Microscopic Simulation

Park and Schneeberger (2003) *(25)* proposed a 9-step process for calibrating VISSIM simulation models: (1) measure of effectiveness selection, (2) data collection, (3) calibration parameter identification, (4) experimental design, (5) run simulation (6) surface function development, (7) candidate parameter set generations (8) evaluation, and (9) validation through new data collection. This process was applied to a case-study calibration scenario. Important and relevant conclusions and recommendations from the outworking of this process include:

- Run the simulation multiple times for each scenario
- Use visualization in the calibration process. Ensuring that vehicle movements and traffic operations represent real-world expectations is crucial to calibration of microscopic simulation models
- Identify controllable input parameters (and acceptable ranges of these parameters) which can be manipulated during the calibration process. Controllable input parameters in VISSIM include: emergency stopping distance, Lane-change distance, Desired speed distribution, Number of observed preceding vehicles, Average standstill distance, Waiting time before diffusion, and Minimum headway
- Perform statistical comparison of chosen MOEs to verify model is calibrated.

Liu et al. (2012) *(26)* developed a procedure for developing and calibrating VISSIM models for U-turns as unsignalized intersections, including relevant design and parameter recommendations for such simulation. Researchers modeled U-turns using VISSIM's *priority rules*, in which lines are placed for turning vehicles defining the necessary headway and gap-time before a turning movement will be made. The other important factors involved in properly calibrating U-turning movements were U-turning speed and the percentage of vehicles turning to the outermost lane. These factors were varied in VISSIM, and U-turning capacities were compared to HCM U-turning capacities to yield mean absolute percent errors (MAPE) for different combinations. The optimal solution was found for both 4-lane and 6-lane roadways. For 4-lane roads, the combination of parameters with minimal MAPE was: Gap Time = 6.3 seconds, Turning Speed = 8 mi/hr., and Percentage of Vehicles to Outside Lane = 99%. For 6-lane roads, these optimal parameters were: Gap Time = 5.1 seconds, Turning Speed = 9 mi/hr., and Percentage of Vehicles to Outside Lane = 63%. These parameters yielded U-turn capacities very similar to those found in both field measurements and the HCM estimation model.

Siddiqui (2011) *(19)* provided a detailed description of modeling TWLTLs in VISSIM by using a combination of overlapping links and priority rules at all driveway turning movements and determined that VISSIM could successfully simulate TWLTL operations. The important parameters associated with the priority rules included minimum gap times for left-out, left-in from TWLTL, and right-out movements. Field observation found these minimum gap values to be 3.1, 3.6, and 3.0 seconds respectively. As with many of the other VISSIM simulation research initiatives reviewed, Synchro was used to optimize signals for alternative scenarios. A warm-up time (of 10 minutes) was also used to ‘populate’ the network prior to collecting data. The base model was considered calibrated when travel times were within 2% of recorded field values for both mainline directions of travel.

2.2.4. *Summary of State of the Art Review*

A review of the literature as it relates to operational impacts of raised medians (and thus indirect left-turn movements – U-turns), driveway density, corner clearance, and left-turn-in and –out restriction revealed several similar trends. In general, past research has found that U-turns do not significantly negatively impact operations at signalized intersections, and that RTUT movements as alternatives to DLT movements can have better operational performance under certain traffic conditions. Different studies did measure ‘operational impact’ through different measures of effectiveness (MOE’s). Some studies analyzed delay to turning vehicles at driveways, while others investigated traffic operations along the mainline direction of travel by analyzing delay, travel time, and average speed for these movements. Several studies came to the similar conclusion that changes in mainline volume were more impactful to mainline traffic operations than other factors (i.e. access density and volume). A number of studies also noted that there are volume thresholds (driveway and mainline) at which access management techniques (RTUT instead of DLT; restricting left-in/left-out) become advantageous operationally. Additionally, past research initiatives have noted that increased access density has negative effects on both through traffic and driveway delays/capacities and have presented alternative methods of establishing guidelines for access spacing and corner clearance according to these findings – which are comparable to current practice but (according to the claim of the research) more justifiable. Finally, there is a relatively established history of using microsimulation to operationally evaluate access management strategies; many of which use VISSIM and Synchro. Several studies have also commented on calibration processes for microsimulation and provided useful recommendations for parameter values to use in this process.

2.3. Review of Practice

The purpose of this final section of the literature review is to provide warrants, recommendations, and guidelines currently adopted by state transportation agencies relating to the access management strategies studied in this thesis. An overviews of these findings are presented in the sub-sections that follow, with comparison tables included at the end of the section. This information is relevant in determining if/where there is a consensus about warranting and designing certain access management strategies, and in determining values to use and test in the simulation analysis of this research.

2.3.1. Non-Traversable Median Recommendations

Connecticut *(27)* warrants raised medians on roadways where design speeds are 50 mph or less.

Florida *(28)* requires all roadways over 40 mph in design speed have *some* restrictive median treatments. All 7-lane roadway sections have highest priority for retrofit, while all 5 lane sections and facilities with over 28,000 in daily traffic have high priority for retrofit.

Georgia *(29)* recommends raised medians on multilane roadways with design speed greater than 45 mph and on multilane roadways with 3 or more lanes in each direction. Georgia also recommends spot improvements of raised medians at intersections with: 18,000 base year ADT and 24,000 design year ADT, an accident rate greater than state average, and excessive queue lengths.

Idaho *(30)* recommends raised medians on all new multiline state highways, on modernization of multilane state highways of posted speeds of 45 mph or greater, on all undivided state highways where annual collision rate is greater than statewide annual average collision rate for similar roadways, on state highways when ADT exceeds 28,000 vehicles per day both directions and on all multi-lane state highways undergoing resurfacing, restoration, and/or rehabilitation.

Kansas *(31)* provides that raised medians are usually used in developed locations and should only be used when speeds are equal to or less than 45 mph and when volumes are above 20,000 AADT on 5-lane roadways.

Kentucky *(32)* recommends raised medians on all new multilane arterials and on existing roads where ADT, access density, and/or turning volumes exceed thresholds for TWLTL's. Kentucky's guidelines for TWLTLs are as follows:

- TWLTL generally appropriate for:
    - Urban/suburban multi-lane roadways with:
        - Projected ADT < 24,000
        - 10 accesses/mi < Access Density < 85 accesses/mi
        - Left-turn volume < 100 vph

Kentucky also recommends raised medians on any (2-lane and Multilane) Urban Principal Arterial with speeds greater than 45 mph and speeds less than 45 mph <u>but</u> volume greater than 10,000; on Multilane Urban Principal Arterials; on any (2-lane and Multilane) Urban Minor Arterial with speeds greater than 45 mph <u>and</u> volume greater than 10,000; and on Multilane Urban Minor Arterials with speeds greater than 45 mph or with speeds less than 45 mph <u>but</u> volume greater than 5,000.

Maine *(33)* and ***Michigan*** *(34)* warrant raised medians on multilane roadways with AADT of 25,000 or greater

Mississippi *(35)* has separate warrants raised medians in a spot improvement type implementation and in a corridor wide implementation. Roadways with speed limit greater than 40 mph and ADT greater 30,000 should have median along length of corridor. Roadways with speed limit less than 40, and ADT less than 30,000 should have spot medians to improve safety where deemed necessary.

Missouri *(36)* recommends raised medians, in general, where current and projected volume is greater than 28,000 AADT. They are especially recommended in corridors where traffic volume is high, density of commercial driveways is high (over 24/mile in both directions), and other access management strategies (like driveway consolidation and corner clearance) are not practical. Raised medians should be used on arterial facilities with 3 or more through traffic lanes in each direction

New York *(37)* recommends nontraversable medians where high traffic volumes, sight restrictions, rates of left turning traffic and possibly traffic speeds indicate that a problem may be expected due to the left turning movements.

Oregon *(38)* recommends raised medians on all new, multilane expressways on new alignments; all other existing urban expressways should consider construction of non-traversable median when projects are developed along these highways.

Pennsylvania *(39)* provides a general criteria for raised medians on roadways of a history of crash rates caused by conflicting turning movements, high average daily traffic volumes, and unacceptable LOS along the corridor and at intersections.

Texas *(40)* recommends raised medians on roadways when ADT volumes are greater than 20,000 vpd, and the demand for mid-block turns is high.

Washington *(41)* recommends considering restrictive medians on multilane limited access highways and multilane managed access highways when design hourly volume (DHV) is over 2000 vph.

The results from the state of practice review of state transportation agencies for restrictive median recommendations (by design speed, number of lanes, traffic volume, accident rate, access density, and left-turn volume where applicable) are shown on the following page in Table 1. The most common warrant variable cited by states is traffic volume. Of the 13 states which had raised median warrants, 12 include a traffic volume threshold above which non-traversable medians should be considered. ADT volumes cited range from 20,000 to 30,000 vpd, and one state recommends using design hourly volume (DHV) of 2,000 vph. The other common warrant variables are design speed and the number of lanes. Typically, states recommend implementing raised medians on roadways with design speeds greater than or equal to 45 mph, however a few states recommend raised medians on roadways with design speeds *less* than this value. For states that referenced the type of facility, all recommended raised medians on multilane facilities.

Table 1: *Comparison Summary of State Agency Non Traversable Median Recommendations*

	Design Speed	Number of Lanes (in one direction)	Traffic Volume	Accident Rate	Access Density	Left-Turn Volume
Connecticut	< 50 mph	--------	--------	--------	--------	--------
Florida	> 40 mph	2 & 3 lanes	ADT > 28,000 vpd	--------	--------	--------
Georgia	> 45 mph	≥ 3 lanes	ADT ≥ 24,000 vpd	> state average	--------	--------
Idaho	> 45 mph	≥ 2 lanes	ADT ≥ 28,000 vpd	> state average	--------	--------
Kansas	≤ 45 mph	--------	ADT > 20,000 vpd	--------	--------	--------
Kentucky	> 45 mph	≥ 2 lanes	ADT > 24,000 vpd	--------	> 85 access/mile	> 100 vph
Maine	--------	--------	ADT > 25,000 vpd	--------	--------	--------
Michigan	--------	--------	ADT > 25,000 vpd	--------	--------	--------
Mississippi	> 40 mph	--------	ADT > 30,000 vpd	--------	--------	--------
Missouri	--------	≥ 3 lanes	ADT > 28,000 vpd	--------	> 24 access/mile (in both directions)	--------
Texas	--------	--------	ADT > 20,000 vpd	--------	--------	--------
Washington	--------	--------	DHV > 2,000 vph	--------	--------	--------

2.3.2. Nontraversable Median Opening Spacing Guidelines

Many states provide median opening spacing guidelines according to different roadway functional classes, speed limits, and degree of urban development. For the sake of comparison and brevity, rather than providing these varying guidelines here, only those guidelines relevant to the corridors to studied in this research are presented: four-six lane urban and/or suburban minor and/or principal arterials that are fully developed and have a 45 mph posted speed. Thus, unless otherwise noted, the spacing presented is the spacing the state provides for roadways with those said characteristics. Full median crossovers/openings are those openings that allow all movements, whereas directional median crossovers/openings are those that only allow left-in/U-turns. Where the state has not specified between full and directional median opening, full median opening has been assumed.

Alabama *(42)*, ***Florida*** *(28)*, ***Kansas*** *(31),* ***Missouri*** *(36), and* ***Montana*** *(43)* recommend a full median crossover spacing of 1,320 ft. and a directional median crossover spacing of 660 ft.

Connecticut *(27)* provides median openings at all intersections and recommends full median crossover spacing be between 1,320 and 2,640 ft.

Delaware *(44)* recommends full median crossover spacings of 1,000 to 1,500 ft.

Georgia *(29)* recommends a preferred full median crossover spacing of 2,000 ft. and a minimum spacing of 1,000 ft.

Idaho *(30)* recommends full median crossovers at all signalized intersections, locations meeting the criteria for a signal warrant, locations anticipated to meet future traffic signal considerations, locations where a median opening would pose no significant reduction in safety or operational efficiency. Openings are subject to Idaho DOT approach spacing guidelines.

Illinois *(45)* recommends full median crossover spacing be between 660 ft. and 1,320 ft.

Indiana *(46)* recommends that new median openings be spaced at least 400 ft. from an existing crossover given that it would improve the safety of the corridor.

Kentucky *(32)* recommends a full median crossover spacing of 2,400 ft. and a directional median crossover spacing of 1,200 ft. Mid-block median openings (used for U-turns only) may be located 300 feet from an intersection at which left-turns are restricted if the following conditions are met: adequate sight distance, adequate space for accommodating U-turn design vehicle, adequate space for incorporation of “left-turn” auxiliary lane (including taper and storage), and there is not potential for use by drivers desiring to turn left from nearby driveways

Louisiana *(47)* recommends U-turn median openings for passenger cars be spaced at 1,320 ft., partial median crossovers be spaced at 2,640 ft., and full median crossovers be allowed only if traffic signal spacing requirements are met.

Maine *(33)* recommends full median openings at all public roads and major traffic generators and/or at a spacing of 100 feet plus the left-turn lane length.

Maryland *(48)* recommends full median opening spacing be 750 ft. on urban arterials (densely developed with posted speed limits of 40 mph or less) and 1,500 ft. on suburban arterials.

Michigan *(34)* recommends that as long as medians are 30 ft. or more in width, median crossovers may be spaced at 660 ft. apart, and adjusted 100 ft. either way according to design needs.

Mississippi *(35)* recommends full and directional median crossovers be spaced 1,760 ft. apart.

New York *(37)* recommends that openings be provided only at major cross streets and at locations that serve large traffic generators or emergency vehicles, and to avoid opening the median for low volume (one-way, design-hour volume of 100 vph or less) intersecting streets and left movements from the arterial.

North Carolina *(49)* states that median crossover spacing is largely dependent upon the need for adequate storage for left turning and U-turn vehicles at intersections. A crossover shall not be placed where it interferes with storage requirements for existing intersections. All movement crossovers shall not be spaced any closer than 1,200 ft. apart. Where this spacing requirement is not met and there is a defined need for left-turn access, then a directional crossover will be considered.

Oregon *(50)* recommends that for major arterials, the full median opening spacing be 1,320 ft. and that for minor arterials this spacing be 330 ft.

Pennsylvania *(39)* recommends that the spacing of median breaks shall be in accordance with the minimum driveway spacing, traffic signal spacing and corner clearance requirements.

South Carolina *(51)* spacing for full median crossovers is 500 ft.

South Dakota *(52)* recommends that both full and directional median openings be spaced at 1,320 ft. apart.

Texas *(53)* recommends providing median openings at all public roads and at major traffic generators (industrial sites or shopping centers). Additional openings should be provided so as not to surpass a maximum of 2,640 ft. Openings should be located where adequate sight distance is available and where median is sufficiently wide to permit an official design vehicle to turn between inner freeway lanes.

Utah *(54)* does not allow median openings within the functional area of an existing or planned interchange, signalized intersection, or major unsignalized intersection.

Virginia *(55)* provides different spacing regulations from different types of intersections/access. For principal and minor arterials, the spacing from unsignalized intersections and full median crossovers to signalized or unsignalized intersections and full median crossovers is 1,050 ft. and 660 ft. respectively.

Washington *(41)* recommends that median opening used only for U-turns be spaced at 1,000 ft., with a minimum acceptable spacing of 300 ft. plus the acceleration lane length from a stop. For full median openings, the Washington guideline is 1,320 ft.

A summary comparison table of the findings from the review of state practices is shown on the following page in Table 2. While numbers vary for each state, a common recommended spacing for full and directional median openings is 1,320 ft. and 660 ft. respectively.

***Table** 2: Comparison Summary of State Agency Median Opening Spacing Guidelines*

	Full Openings (ft.)	Directional Openings (ft.)	For U-Turns Only (ft.)
Alabama	1,320	660	--------
Connecticut	1,320 - 2,640	--------	--------
Delaware	1,000 - 1,500	--------	--------
Florida	1,320	660	--------
Georgia	2,000 (preferred) | 1,000 (minimum)	--------	--------
Idaho	At all signalized intersections	--------	--------
Illinois	660 - 1,320	--------	--------
Indiana	400	--------	--------
Kansas	1,320	660	--------
Kentucky	2,400	1,200	300 (from an intersection)
Louisiana	If signal spacing requirements met	2,640	1,320
Maine	100 + left-turn lane length (and at public roads and major traffic generators)	--------	--------
Maryland	750 (urban) | 1,500 (suburban)	--------	--------
Michigan	660 (± 100)	--------	--------
Mississippi	1,760	1,760	--------
Missouri	1,320	660	--------
Montana	1,320	660	--------
New York	At major cross-streets, and large traffic generators (≥100 vph)	--------	--------
North Carolina	1,200 (minimum)	When 1,200 not available	--------
Oregon	1,320 (major arterials) | 330 (minor arterials)	--------	--------
Pennsylvania	According to minimum driveway spacing, signal, corner clearance spacings	--------	--------
South Carolina	500	--------	--------
South Dakota	1,320	1,320	--------
Texas	All public roads and major traffic generators | 2,640 (maximum)	--------	--------
Utah	Outside of functional area of interchange, intersection	--------	--------
Virginia	1,050 (major arterials) | 660 (minor arterials)	--------	--------
Washington	1,320	--------	1,000

2.3.3. Driveway Spacing Guidelines

Similar to median opening spacing guidelines, many states provide driveway access spacings in terms of speed. Again, for the sake of comparability and brevity, only spacings for the 45 mph posted speed are presented here, with other qualifiers noted for each state as they pertain.

Alabama *(42)* specifies access spacing according to the presence of a median. Without a median, directional access can be spaced 440 ft. apart and full access 660 ft. With a median, directional access is to be spaced 440 ft. apart and full access 1,320 ft. apart. Shared or individual direct connections to out-parcels may be provided if twice the normal spacing requirements are met. Multiple Driveways will only be considered on parcels with frontage greater than 660 ft. If 3 driveways are desired on one parcel, there must be frontage in excess of 1,980 ft.

Colorado *(56)* permits one access per parcel if reasonable access cannot be obtained from a local street or road system. Additional right-turn only access is allowed where acceleration and deceleration lanes can be provided. Access spacing guidelines follow allowable sight-distance. This results in a recommended spacing of 325 ft.

Connecticut *(27)* permits parcels with frontage between 50 and 100 ft. to have 2 entrances if one-third of total frontage is used to separate driveways.

Delaware *(44)*, ***Indiana*** *(46)*, and ***Utah*** *(54)* provide an ideal driveway spacing of 350 ft.

Florida *(57)* provides a driveway spacing of 245 ft.

Georgia *(58)* recommends a spacing of 230 feet for access without a right-turn lane and 369 feet for access with a right turn lane.

Idaho *(30)* recommends a driveway spacing of 150 ft.

Illinois *(45)* allows two driveways for an average commercial property. Between entrances into shopping centers and similar developments that generate high traffic volumes, a minimum of at least 440 ft., and preferably 660 ft. is required.

Iowa *(59)* recommends a spacing of 300 to 600 ft.

Kansas *(31)* recommends a driveway spacing of 300 ft.

Kentucky *(32)* recommends a commercial, industrial, recreational driveway spacing of 1,200 ft.

Louisiana *(60)* provides for a spacing of 550 ft., however the spacing may be reduced by one-half if a non-traversable median exists within 200 ft. of both sides of the access and connection and a right-in/right-out access connection is installed.

Maine *(61)* recommends a driveway spacing of 265 ft.

Maryland *(48)* requires a minimum 20’ tangent between adjacent entrances on the same side

Michigan *(34)* recommends an unsignalized driveway spacing of 350 ft., while spacing to/from other intersections is given by the information below:

From:	To Full movement driveway or other access point	To right in/right out driveway
Median Opening	75’	75’
Along arterial or from another intersecting arterial	300’	120’
Along arterial intersecting a collector	200’	125’

Minnesota *(62)*, ***Texas*** *(40, 53)*, and ***Vermont*** *(63)* recommend a driveway spacing of 360 ft.

Mississippi *(35)* recommends that for a commercial drive with greater than 50 peak hour trips and a driveway ADT of less than or equal to 2000 ADT the driveway spacing by 350 ft. and for a commercial drive with less than or equal to 50 peak hour trips and ADT less than 2000 ADT the driveway spacing be 100 ft.

Missouri *(36)* recommends that for principal and minor arterials with nontraversable medians the spacing be 220-330 ft. and 165 ft. respectively, and for principal and minor arterials with traversable medians, the spacing be 440-660 ft. and 330 ft. respectively.

Montana *(43)* provides a spacing of 325-375 ft. on undivided highways and 150 ft. on divided highways.

Nebraska *(64)* permits access to all properties but recommends that the consolidation of driveways be considered wherever feasible.

Nevada *(65)* recommends a spacing of 350 ft. on principal arterials with full access driveways. On principal arterials where only right-turns are allowed, a spacing of 250 ft. is recommended, and on minor arterials, a 250 ft. spacing is recommended.

New Mexico *(66)* recommends the following spacings for principal and minor arterials:

Principal Arterials

Non-Traversable Median		Traversable Median
Full Access	Partial Access	
1,320 ft.	450 ft.	450 ft.

Minor Arterials

Non-Traversable Median		Traversable Median
Full Access	Partial Access	
660 ft.	400 ft.	400 ft.

New York *(37)* states that the optimal driveway spacing cannot be precisely determined, but there is a consensus that the driveway spacing on the order of (300 to 500ft), depending on the operation speed on the highway and traffic generation of the development is desirable to reduce accidents and maintain the flow of traffic.

North Carolina *(67)* permits, normally, one driveway connection for a single property or commercial site. However, the NCDOT may consider additional entrances or exits as justified and if such access does not negatively impact traffic operations and public safety. Only one combined entrance and exit connection will be permitted where the frontage is less than 100 feet. On most State maintained routes, the minimum distance between the centerlines of full-movement driveways into developments that generate high traffic volumes should be at least 600 feet. However, on routes with safety, congestion, or operational problems, 1,000 feet or more may be required between the centerline of any left turn access points and any adjacent street and driveways. The minimum distance between drives does not apply to service drives not used by the general public.

Ohio *(68)* recommends a driveway spacing of 425 ft.

Oregon *(50)* recommends 860 ft. spacing as the minimum access spacing to provide maximum egress capacity. For statewide highways with AADT greater than 5,000, the driveway spacing recommended is 800 ft. For regional highways with AADT greater than 5,000, the driveway spacing recommended is 500 ft.

Pennsylvania *(39)* permits only one access to be permitted for a property. An additional access or accesses shall be permitted if the applicant demonstrates that an additional access or additional accesses are necessary to accommodate traffic to and from the site and it can be achieved in a safe and efficient manner. The municipality shall restrict access to right turn only ingress and egress or to another state maintained road or local road if safe and efficient movements cannot be accommodated. For principal arterials, the desirable spacing is 600 ft., and for minor arterials, this desirable spacing is 400 ft.

South Carolina *(51)* recommends a spacing of 325 ft.

South Dakota *(52)* recommends that the driveway spacing be between 100 and 660 ft., depending on the level of development.

Virginia *(55)* provides different spacing regulations from different types of intersections/access. For principal and minor arterials, spacing from full access entrances and directional median to other full access entrances and any intersection or median crossover is 565 ft. and 470 ft. respectively. For principal and minor arterials, the spacing from partial access one or two way entrances of any type of entrance, intersection or median crossover is 305 ft. and 250 ft. respectively.

Washington *(41)* provides different spacing guidelines by class. In Class 1 (mobility is the primary function), the spacing is 1,320 ft. In Class 2 (mobility is favored over access), the spacing is 660 ft. In Class 3 (balance between mobility and access in areas with less than maximum buildout), the spacing is 330 ft. In Class 4 (balance between mobility and access in areas with maximum buildout), the spacing is 250 ft. Finally, in Class 5 (access needs may have priority over mobility), the spacing is 125 ft.

West Virginia *(69)* states that frontages of 50 ft. or less should be limited to one driveway. Normally, not more than two driveways are permitted on any single property tract or business establishment. The recommended spacing is 230 ft.

Wyoming *(70)* recommends a spacing of 330 ft.

A summary comparison table of the findings from the review of state practices is shown on the following page in Table 3. Recommended spacings (for developed arterials with 45 mph design speed) varied for each state, however a common recommended spacing is ~350 ft. Several states also made a distinction in spacing between full-access driveways and restricted-access driveways. In cases where this distinction was made, the spacing between restricted-access driveways is less than that for full-access driveways.

***Table 3**: Comparison Summary of State Agency Driveway Spacing Guidelines (continued on next page)*

	Full Access Spacing (ft.)
Alabama	660 (without median) | 1,320 (with median)
Colorado	325
Connecticut	2 entrances on frontage between 50 and 100 ft.
Delaware	350
Florida	245
Georgia	230 (without right-turn lane) | 369 (with right-turn lane)
Idaho	150
Illinois	2 entrances for average commercial property | 440-660 (high-traffic generators)
Indiana	350
Iowa	300-600
Kansas	300
Kentucky	1,200
Louisiana	550
Maine	265
Maryland	20 (tangent between adjacent entrances)
Michigan	350
Minnesota	360
Mississippi	350 (> 50 peak hour trips) | 100 (< 50 peak hour trips)
Missouri	Principal Arterial: 220-330 (w/ RM) / 440-660 (w/ TWLTL) | Minor Arterial: 165 (w/ RM) / 330 (w/ TWLTL)
Montana	325-375 (undivided) | 150 (divided)
Nevada	350 (principal arterials) | 250 (minor arterials)
New Mexico	Principal Arterial: 1,320 (w/ RM) / 450 (w/ TWLTL) | Minor Arterial: 660 (w/ RM) / 400 w/ (TWLTL)
New York	300-500
North Carolina	One access per 100 ft. frontage | 600 (high-traffic generators)
Ohio	425
Oregon	500-860
Pennsylvania	600 (principal arterials) | 400 (minor arterials)

South Carolina	325
South Dakota	100-660
Texas	360
Utah	350
Vermont	360
Virginia	565 (principal arterials) | 470 (minor arterials)
Washington	125-1,320 (depending on mobility vs. access needs)
West Virginia	230
Wyoming	330

2.3.4. Corner Clearance

As before, for the sake of comparability and brevity, only corner clearances for the 45 mph posted speed are presented here, with other qualifiers noted for each state as they pertain.

Alabama *(42)* provides corner clearances in terms of median treatment and connection type as shown in the tables below.

Without Median

Connection Type	Corner Clearance (Without median)
Right-in (upstream only)	250 ft.
Right-out (downstream only)	250 ft.
Right-in/Right-out	275 ft.
Full Access (unsignalized)	660 ft.
Full access signalized	1320 ft.

With Median

Connection Type	Corner Clearance (With median)
Right-in (upstream only)	125 ft.
Right-out (downstream only)	125 ft.
Right-in/Right-out	250 ft.
Full Access (unsignalized)	660 ft.
Full access signalized	1320 ft.

Connecticut *(27)* permits corner clearances of 10 ft. for commercial driveways.

Florida *(57)* recommends a corner clearance of 245 ft.

Idaho *(30)* provides both upstream and downstream corner clearances based on the median treatment and type of intersection (signalized vs. non-signalized). For signalized intersections, the downstream corner clearance allowed, for both traversable and non-traversable median roadways is 200 ft. For non-traversable median roadways, the upstream corner clearance allowed is 100 ft. while for traversable median roadways the upstream corner clearance is 200 ft. The allowable corner clearance to a median opening is 25 ft. For non-signalized intersections, the downstream corner clearance for traversable and non-traversable medians are both 95 ft. For non-traversable median roadways, the upstream corner clearance allowed is 100 ft. while for traversable median roadways the upstream corner clearance is 200 ft. The allowable corner clearance to a median opening is 25 ft.

Kentucky *(32)* permits a corner clearance of 1,200 ft. for commercial, industrial, and recreational driveways.

Maine *(33)* permits a corner clearance of 75 ft. for unsignalized driveways and 125 ft. for signalized driveways.

Maryland *(48)* recommends a minimum corner clearance of 200 ft. on primary arterials, and 100 ft. on secondary arterials.

Michigan *(34)* permits upstream and downstream corner clearances for signalized intersections of 230 ft. and 460 ft. respectively; and upstream and downstream corner clearances for non-signalized intersections of 170 ft. and 230 ft. respectively

Minnesota *(62)* recommends an upstream corner clearance of 650 ft. and downstream corner clearance of the greater distance between the length of an acceleration lane or stopping sight distance.

Mississippi *(35)* recommends a 125 ft. corner clearance, with an exception to use as low as 50 ft. for right-in/right-out drives.

Missouri *(36)* recommends a minimum corner clearance of 440 ft. for principal arterials and 330 ft. for minor arterials.

Nevada *(65)* specifies corner clearances by driveway type. For residential drives, the allowable corner clearance is 150 ft. For commercial drives, the allowable corner clearance is 350 ft. And for public or private roads the corner clearance allowed is 660 ft.

North Carolina *(67)* specifies a corner clearance of at least 100 ft., where property frontage allows and at no time less than 50 ft.

Ohio *(68)* stipulates that corner clearance shall be the same as the state driveway spacing, 425 ft.

Pennsylvania *(39)* recommends that for principal arterials, the corner clearance be 600 ft., and for minor arterials, 400 ft.

South Carolina *(51)* recommends a corner clearance of 325 ft. for full access drives and 150 ft. for right-in/right-out driveways.

Texas *(40, 53)*, like Ohio stipulates that corner clearance shall be the same as the state driveway spacing, 360ft.

Vermont *(63)* and ***Washington*** *(41)*, like both Texas and Ohio uses spacing standards to stipulate corner clearance, 360 ft. If this value cannot be met, the following provisions are made. With a restrictive median, if the approaching intersection is right-in/right-out or right-in only, the corner clearances may be 115 ft. and 75 ft. respectively. With a restrictive median, if the departing intersection is right-in/right-out or right-in only, the corner clearances may be 230 ft. and 100 ft. respectively. Without a restrictive median, if the approaching intersection is full access or right-in only, the corner clearances may be 230 ft. and 100 ft. respectively. Without a restrictive median, if the departing intersection is full access or right-out only, the corner clearances may be 230 ft. and 100 ft. respectively.

West Virginia *(69)* allows a minimum of 15 feet at the near and far sides of intersection, but 30 to 50 ft. is desirable. If the intersection is signalized, the near side clearance should be two or more times the far side distance.

A summary comparison table of the findings from the review of state practices is shown on the following page in Table 4. Several states distinguished between upstream (approaching) and downstream (departing) corner clearances, while a majority cite one value. Recommended corner clearances (for developed arterials with 45 mph design speed) varied for each state, ranging from 10 ft. to 1,320 ft. However, most corner clearance standards were in the roughly 200-400 ft. range.

***Table 4**: Comparison Summary of State Agency Corner Clearance Guidelines*

	To Signalized		To Unsignalized
	Full Access	**Right-In/Right-Out**	**Full Access**
Alabama	1,320	275 (w/out RM); 250 (with RM)	660
Connecticut	10	--------	--------
Florida	245	--------	--------
Idaho	200 (downstream) | 200 (upstream w/ RM); 100 (up w/out RM)	--------	95 (downstream) | 100 (upstream w/ RM); 200 (upstream w/out RM)
Kentucky	1,200	--------	--------
Maine	150	75	--------
Maryland	200 (primary arterials) | 100 (minor arterials)	--------	--------
Michigan	460 (downstream) | 230 (upstream)	--------	230 (downstream) | 170 (upstream)
Minnesota	Greater of acceleration lane or SSD (downstream) | 650 (upstream)	--------	--------
Mississippi	120	50	--------
Missouri	440 (principal arterials); 330 (minor arterials)	--------	--------
Nevada	350	--------	--------
North Carolina	100 (no less than 50 in limited frontage situations)	--------	--------
Ohio	425	--------	--------
Pennsylvania	600 (principal arterials); 400 (minor arterials)	--------	--------
South Carolina	325	150	Same as signalized
Texas	360	--------	--------
Vermont	360	230 (downstream); 115 (upstream)	--------
Washington	360	230 (downstream); 115 (upstream)	--------
West Virginia	15 (30-50 desirable)	--------	--------

2.3.5. Restricted Access Recommendations

Florida *(57)* stipulates that where minimum corner clearance cannot be met according to the FDOT rules, 125 to 230 feet should become the new minimum corner clearance goal. In these cases of less than minimum corner clearance, left-turns from these driveways should be prohibited (or limited).

Illinois *(45)* stipulates 3/4 access (no left out) on high-volume divided arterials where prevented left-turn volume from the entrance is relatively low, and recommends consolidating access on adjacent properties with continuous parking lots and separate parcels assembled under one entity/usage.

Kansas *(31)* states that right-in/right-out access is typically used on highways in developed areas where the influence areas of adjacent access points provide a window for right-turns but not left-turns.

Maryland *(48)* recommends that commercial right-in/right-out be used on all divided highways with posted speeds above 40 mph.

Minnesota *(62)* recommends the following: when high traffic volumes result in a lack of gaps for entering and exiting traffic to safely cross, left turn movements and crossing movements may be restricted; when a driveway and an intersection are closely spaced such that a vehicle following a turning vehicle cannot anticipate where the lead vehicle will turn, right-in movements may be restricted; when an access is located where it may be blocked by queuing traffic from a nearby intersection, left-turn movements, crossing movements and right-out movements may be restricted; where an access is needed for a specific movement such as a one-way driveway, the driveway may be limited to right-in-only or right-out-only; on a divided highway where a lack of gaps prevent entering traffic from safely weaving across multiple lanes to make a left-turn or U-turn, and a reasonably convenient and suitable alternative route is available, right-out movements may be restricted; or where adequate sight distance does not exist for a specific movement, that movement may be restricted.

New Jersey *(71)* stipulates that if future traffic volumes could warrant installing a traffic signal and signalized spacing requirements cannot be met, as a condition of the access permit, the Commissioner may, at such time as future traffic volumes are reached, close the left-turn access in accordance with New Jersey Code; If an undivided highway becomes divided, as a condition of the access permit, the Commissioner may at such time close the left-turn access in accordance with New Jersey Code.

New Mexico *(66)* states that restrictions to full left-turn access may be required due to safety or operational deficiencies that would be expected if a full access median were implemented. Restricted movements should be prohibited through geometric design and channelization supplemented by signing in accordance with the MUTCD.

North Carolina *(67)* stipulates that if access connections have to be located within the functional area due to limited property frontage, the

NCDOT may restrict access to “right-in/right-out” or other limited movement treatments. Such driveways must still meet all location and minimum distance requirements; In locations where the sight distance cannot be met on both sides of the driveway location, the driveway may be denied. In some cases, the left turn movements into or out of the driveway may be prohibited; thus, restricting the driveway operation to right turns only.

Pennsylvania *(39)* states that the municipality shall restrict access to right turn only ingress and egress or to another state maintained road or local road if safe and efficient movements cannot be accommodated.

Texas *(40)* stipulates that where adequate access connection spacing cannot be achieved, the permitting authority may allow for a lesser spacing when shared access is established with an abutting property. Where no other alternatives exist, construction of an access connection may be allowed along the property line farthest from the intersection. To provide reasonable access under these conditions but also provide the safest operation, consideration should be given to designing the driveway connection to allow only the right-in turning movement or only the right-in/right out turning movements if feasible.

Utah *(72)* recommends that roadway approaches and driveways that are located too close to an intersection can affect signal operation. Consider restricting access to “Right In/ Right Out” operation.

Virginia *(55)* states that on small corner parcels, left turn accessibility may be a problem and access to parcels may be limited to right-in/right-out or similarly restricted movements.

A summary comparison table of the findings from the review of state practices is shown on the following page in Table 5. A common recommendation was where gaps in traffic did not adequately allow for left-turn access. Another common recommendation was for driveways in influence areas of intersections (and/or where inadequate corner clearance was provided).

***Table 5**: Comparison Summary of State Agency Restricted Access Recommendations*

	Restrict to Right-In/Right-Out:
Florida	When minimum acceptable corner clearance is not met
Illinois	On high-volume divided arterials where prevented left-turn volume from entrance is relatively low
Kansas	On highways in developed areas where the influence areas of adjacent access points do not provide window for left-turns
Maryland	On all divided highways with posted speeds above 40 mph
Minnesota	When high traffic results in a lack of gaps for entering/exiting traffic and/or when blocked by intersection queue
New Jersey	If signalized spacing cannot be met or undivided highway becomes divided
New Mexico	If safety or operational deficiencies are expected
North Carolina	If driveway is in influence area of the intersection
Pennsylvania	If safe and efficient movements cannot be accommodated
Texas	Where adequate access connection spacing cannot be achieved
Utah	For roadway approaches and driveways that are located too close to an intersection
Virginia	In situations with limited corner clearance

CHAPTER THREE

RESEARCH METHODOLGY

Recall that there were two objectives of this thesis: **(1)** quantify and compare the operational impacts of four access management strategies – (i) Access Spacing, (ii) Corner Clearances, (iii) Access Restriction of Selected Driveways, and (iv) Non-Traversable Medians – during peak-hour traffic conditions on urban/suburban arterials in South Carolina, and **(2)** quantify and compare the operational impacts of three access control alternatives – (i) full access at all driveways, (ii) right-in/right-out access at all driveways with RTUT movements at nearest feasible intersections, and (iii) alternating access (between full access and right-in/right-out) depending on prevailing traffic conditions for a longer study time indicative of both off-peak and peak hours.

Traffic microsimulation tools have been used in numerous past research efforts to evaluate existing and alternative traffic scenarios because they are a cost-effective means of measuring the impacts of changes in traffic conditions, roadway geometry, and vehicle routing *(9, 10, 77, 78)*. In order to satisfy each objective, the microscopic simulation software, VISSIM, was used to establish base models of existing corridors in South Carolina from which alternative scenarios could be developed to test each of the strategies/scenarios for each of the two objectives. The subsequent sections of this chapter describe the development of said base models (including their site selection, data collection, and calibration) as well as the development of the simulation models used to test each alternative. The chapter concludes with a graphic highlighting the process and different alternative scenarios tested.

3.1. Base Model(s) Development

3.1.1. Corridor Selection and Description

Two corridors were desired to perform the analysis – a 5-lane corridor (2-lanes each direction with a TWLTL), and a 7-lane corridor (3-lanes each direction with a TWLTL) in order to compare the operational functionality of the alternatives between roads with different numbers of lanes. The selection of the corridors was based on a recently completed SCDOT study *(79)* which conducted an in-depth investigation of access-related incidents along US and SC routes in South Carolina and identified 11 top-ranked routes based on the frequency of driveway related crashes per year. These 11 routes were scanned for roadway segments (of 2-lanes and 3-lanes in each direction) with existing TWLTLs, and high AADT *(73)* (greater than 20,000 vph), high commercial land use, and high driveway densities. Under these criterion, 14 segments were identified, shown in Table 6 on the following page. The two selected corridors were chosen for their proximity to the researchers as well as their high AADT's (both have AADT greater than 30,000). Among the 5-lane segments identified, a 1.5 mile stretch on SC 146 (Woodruff road) in Greenville County was chosen as it is on the corridor with the highest crash rate (0.7) and is known to SCDOT for excessive, recurrent peak hour congestion. Of the 7-lane segments identified, all three were on HWY US29, which has an overall corridor crash rate of 0.22, removing this variable as a distinguishing one for making a selection. The segment chosen then, was the one with highest AADT of the three. These selected corridors are also shown in Figures 1 and 2 in the following pages.

Table 6: Corridor Segments Identified as Potential Sites for Base Model Simulation Development

Operational Analysis Corridors										
Corridor Segment	**Length (miles)**	**AADT (veh/day)**	**No. of Lanes (in 1 direction)**	**Median Treatment**	**No. of Signalized Intersections**	**Signals / Mile**	**No. of Non-Signalized Intersections**	**No. of Driveways**	**Driveways / Mile**	**Crash Rate (over entire corridor)**
SC9_Spartanburg_O1	2.45	26600	2	TWLTL	6	2.4	19	82	33	0.25
US29_Greenville_O3	1.79	26600	3	TWLTL	7	3.9	8	71	40	0.22
US1_Richland_O2	1.58	21600	2	TWLTL	5	3.2	7	90	57	0.34
SC146_Greenville_O1	**1.5**	**34600**	**2**	**TWLTL**	**6**	**4.0**	**5**	**62**	**41**	**0.7**
US25_Greenwood_O2	1.45	22700	2	TWLTL	4	2.8	3	71	49	0.43
US1_Lexington_O3	1.22	42200	2	TWLTL	5	4.1	9	30	25	0.13
US52_Florence_O2	1.18	25200	2	TWLTL	3	2.5	15	45	38	0.06
US52_Florence_O3	1.17	20800	2	TWLTL	7	6.0	5	43	37	0.06
US29_Greenville_O2	**1.1**	**31400**	**3**	**TWLTL**	**5**	**4.5**	**2**	**66**	**61**	**0.22**
US1_Lexington_O2	1.1	33200	2	TWLTL	5	4.5	8	39	35	0.13
US176_Richland_O1	0.94	36500	2	TWLTL	6	6.4	4	55	59	0.53
US17_Horry_O1	0.85	43000	2	TWLTL	2	2.4	9	32	38	0.11
US29_Greenville_O1	0.79	22000	3	TWLTL	4	5.1	4	36	46	0.22
US176_Richland_O2	0.68	36500	2	TWLTL	5	7.4	0	49	72	0.53

***Selected corridors highlighted in bold-red boxes*

3.1.1.1. 5-lane (SC146_Greenville_O1: Woodruff Road, Greenville, SC)

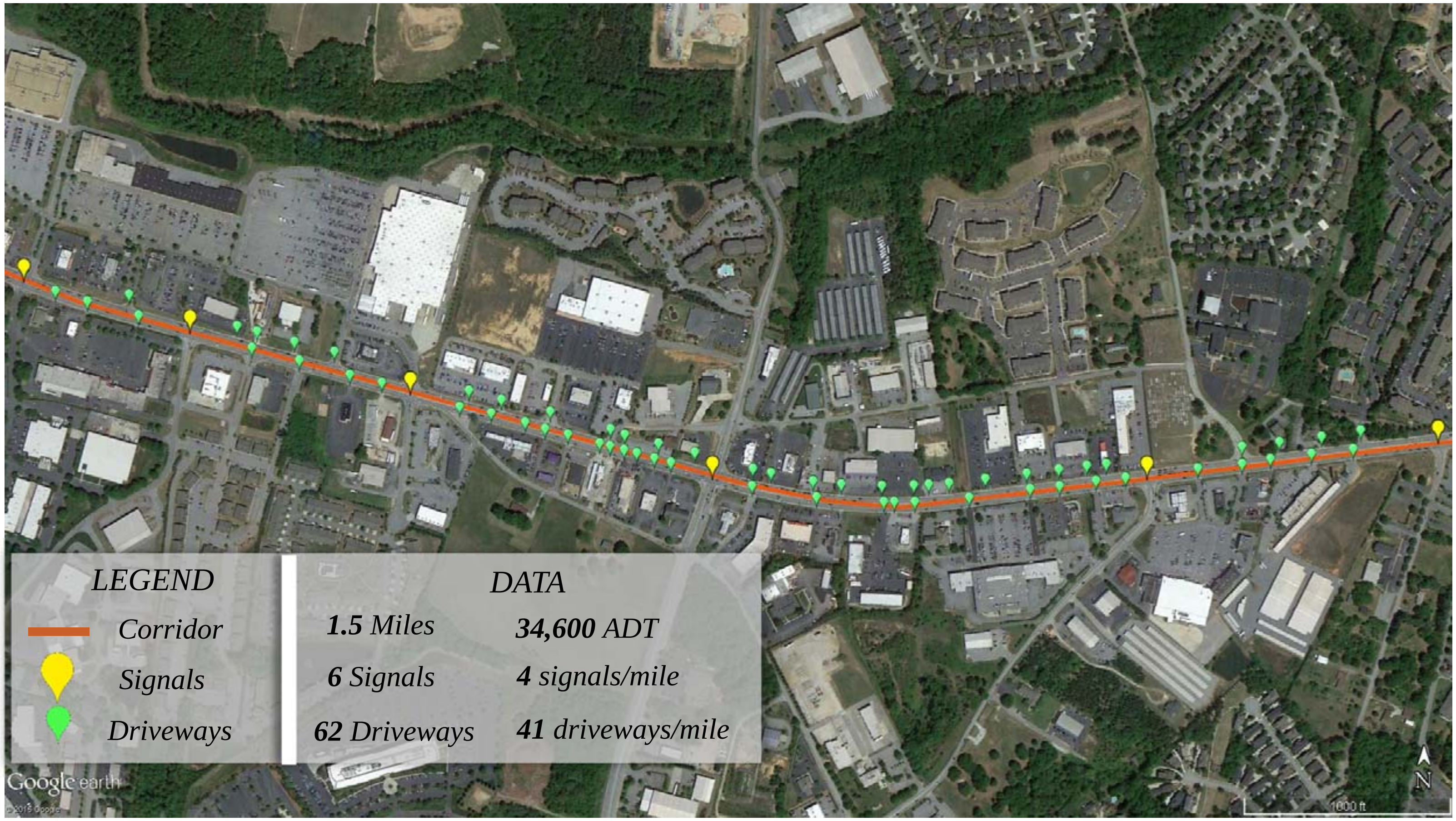


***Figure 1**: Woodruff Road*

3.1.1.2. 7-lane (US29_Greenville_O2: Wade Hampton Road, Greenville, SC)

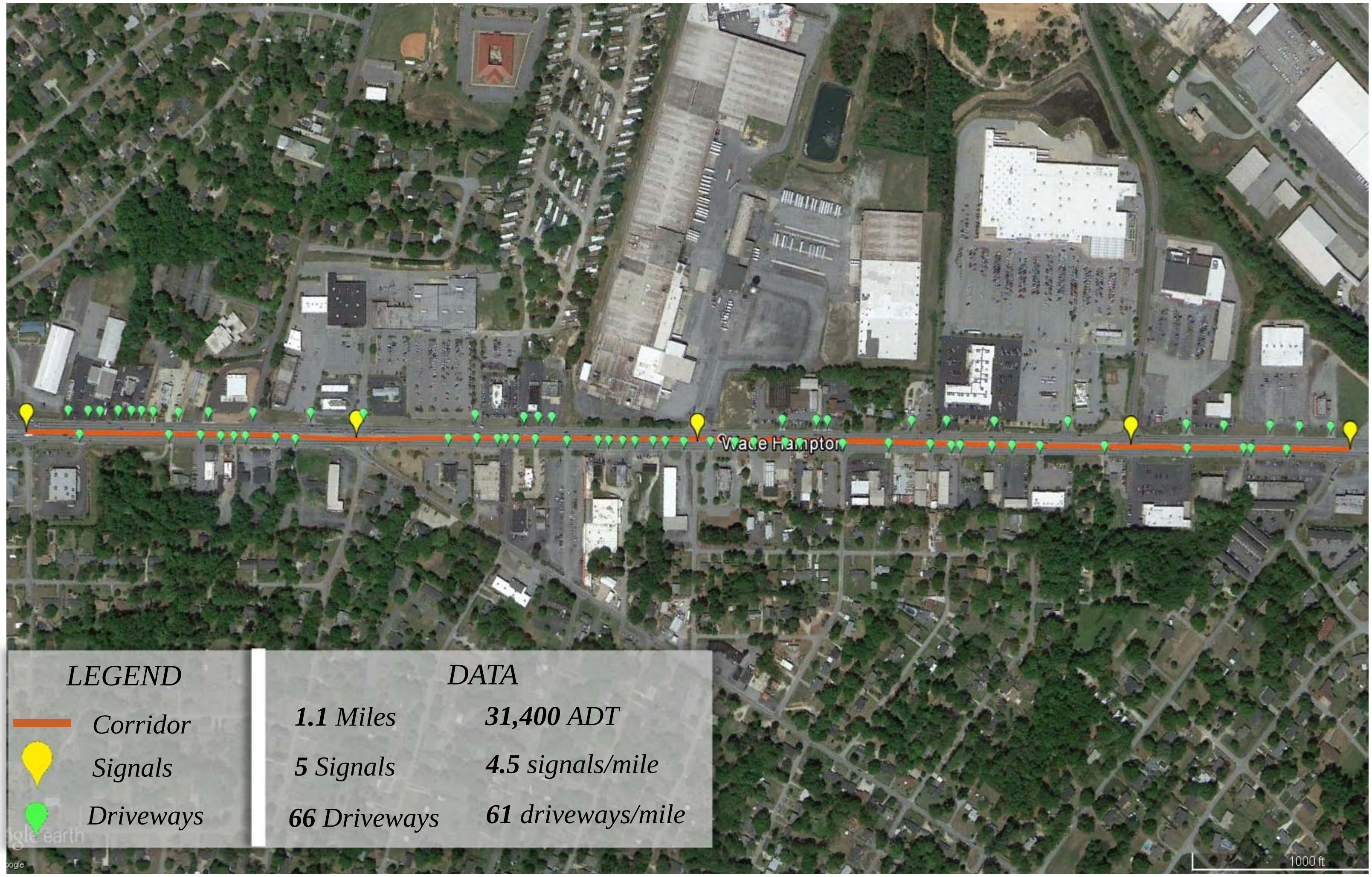


***Figure 2**: Wade Hampton Blvd.*

3.1.2. *Data Collection*

In addition to the descriptive data (obtained using measurements and imagery from Google Earth) given in Table 6 and Figures 1 and 2 on the preceding pages, signal plan, timing, and turning count data, driveway volume data, as well as Eastbound and Westbound travel times needed to be obtained, collected, and/or estimated in order to calibrate the base model. The process and results from this data collection are discussed in the following sections for each roadway segment.

3.1.2.1. *5-lane (Woodruff Road, Greenville, SC)*

Historic signal counts for Woodruff Road were obtained from SCDOT, indicating that for the majority of the signals along the corridor, the peak hour is between 5:00-6:00 PM. Mid-week traffic counts were therefore collected during this interval for each signal. Signal timing plans were obtained from SCDOT and used to design signal splits, network cycle length, and coordination patterns for signal controllers in VISSIM. No optimization was performed on signal splits, cycle lengths, or coordination patterns for the base scenario. Driveway ingress and egress volumes were estimated and assigned using field counts and trip rates from the ITE Trip Generation Manual. Travel times along the corridor were measured during the peak hour for both the Eastbound and Westbound directions using the floating car method. The results of the turning volume counts for Woodruff Road are shown in Table 7, and the travel time results from the floating car method are shown in Table 8. For this corridor, as with the other, the direction from- and to-which traffic and each driveway turned was determined based on the signal volumes at either end of a particular section along the roadway segment. In other words, the ITE Trip Generation Manual provided information of how many trips in and out of a land use to expect, but not from which direction they would come or leave. These ratios of the Trip Gen volumes were determined using engineering judgement as well as a matrix so as to ensure that the entering and exiting volumes at the signals at the East and West end of the section were consistent with the volume counts conducted in the field.

3.1.2.2. 7-lane (Wade Hampton Road, Greenville, SC)

Historic signal counts for Wade Hampton Road were not as conclusive in indicating the peak hour, because only one historic signal count was available from SCDOT, but it did suggest that the peak volumes along the mainline of this stretch of Wade Hampton road occurred between 4:45 and 5:45 PM. Similar to the 5-lane corridor, mid-week traffic counts were collected during this interval for each signal with timing plans obtained from SCDOT and no optimization performed. Driveway volumes were estimated using field counts and the ITE Trip Generation Manual and travel times along the corridor were measured during the peak hour for both the Eastbound and Westbound directions using the floating car method. The results of the turning volume counts for are shown in Table 9, and the travel time results from the floating car method are shown in Table 10.

Table 7: *SC146 (Woodruff Road) Signalized Intersection Turning Volumes during PM Peak Hour (5:00PM – 6:00PM)*

	Southbound			Westbound			Northbound			Eastbound			Total
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
Merovan	99	13	143	0	1435	25	241	22	0	188	1776	0	*3942*
Smith Hines	5	1	12	63	1278	0	186	2	149	24	1717	49	*3486*
Hendrix/Walmart	84	13	104	20	1738	34	239	23	3	67	1287	121	*3733*
Feaster/Verdin	149	164	79	93	1133	47	239	279	149	193	1435	46	*4006*
East Butler	48	78	25	300	1091	25	139	39	357	18	1428	233	*3781*
Bell/Rocky Creek	10	1	48	26	1311	13	82	2	35	49	1932	64	*3573*

Table 8: *SC146 (Woodruff Road) Existing Condition Travel Times During Peak Hour*

	Travel Time (s)					
	Measurement No.				Average	St. Dev
	1	2	3	4		
Eastbound	316	301	366	245	**307**	43.1
Westbound	286	272	294	220	**268**	28.8

Table 9: *<u>US29 (Wade Hampton Blvd.)</u> Signalized Intersection Turning Volumes during PM Peak Hour (4:45PM – 5:45PM)*

	Southbound			Westbound			Northbound			Eastbound			Total
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	
W Lee/Cherokee	220	53	3	92	1401	182	45	77	77	11	1891	30	*4082*
S-23-166	47	48	29	58	1191	31	326	30	24	60	1562	474	*3880*
Vance	2	2	8	13	1302	0	11	0	24	4	1685	6	*3057*
Tappan	183	16	61	10	1175	126	35	25	16	54	1518	55	*3274*
S Watson	32	43	41	30	1206	2	70	71	41	31	1573	67	*3207*

Table 10: *US29 (Wade Hampton Blvd.) Existing Condition Travel Times During Peak Hour*

	Travel Time (s)											
	Measurement No.										Average	St. Dev
	1	2	3	4	5	6	7	8	9	10		
Eastbound	96	93	97	103	104	104	116	144	146	174	**118**	26.0
Westbound	96	98	124	128	129	133	135	140	141	158	**128**	18.0

3.1.3. *Base Model Calibration*

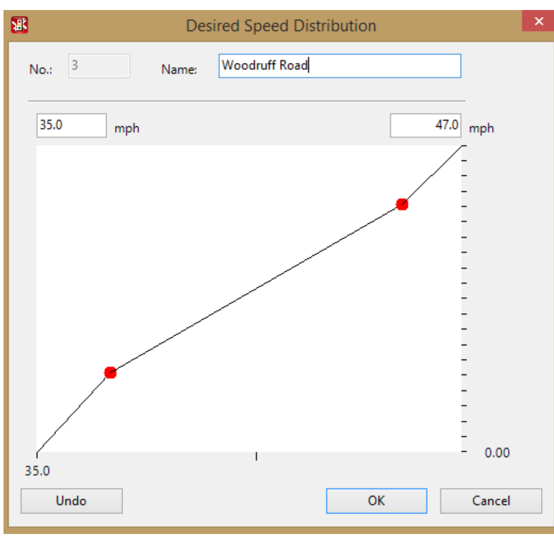
After developing the base geometry, signal controllers, and gateway and driveway volumes, each model (5-lane and 7-lane) had to be calibrated to match the Eastbound and Westbound travel times collected in the field. The base model(s) were considered calibrated when they produced average travel times during the peak hour within 10% of the travel times measured in the field. To reach this calibration threshold, principles from Park and Schneeberger's discussion of microscopic simulation model calibration and validation were used *(25)*. Their study identified emergency stopping distance, lane-change distance, desired speed distribution, number of observed preceding vehicles, average standstill distance, waiting time before diffusion, and minimum headway as controllable parameters which may be reasonably adjusted to calibrate the model. These parameters were manipulated within the acceptable ranges given in Park and Schneeberger's study in order to calibrate the model. The finalized values of these parameters for each corridor for the Traditional Strategies are shown in Table 11 below. Table 12 below it shows the finalized values of these parameters for each corridor for the Demand Responsive Strategies. The base models for the traditional access management strategy scenarios and those for the demand-responsive scenarios were calibrated separately because the loading patterns differ for each. For the traditional strategies, only the peak hour is tested (4,200 sec run time including 600 sec warm up).

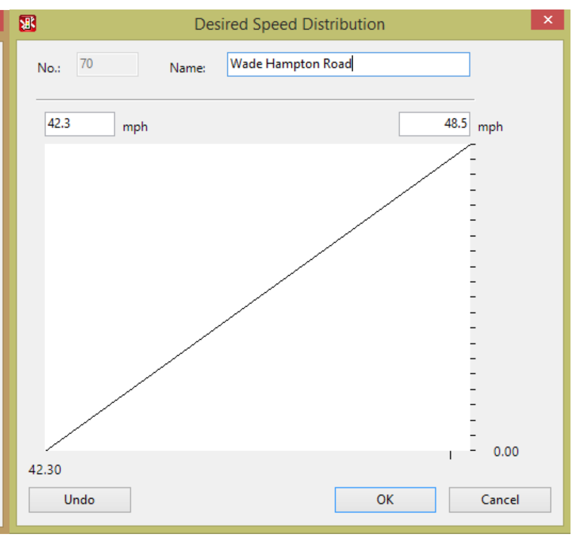
For the demand responsive strategies, a 5-hour run is tested in order to analyze both peak and off-peak conditions.

Table 11*: Calibration Parameters Used in Base Model Calibration (for Traditional Access Management Strategies)*

Parameter	VISSIM Default	Acceptable Range *(25)*	Selected Value	
			Woodruff	Wade Hampton
Emergency Stopping Distance (ft.)	16.4	6.6 to 23	16.4	16.4
Lane-Change Distance (ft.)	656	492 to 984	656	656
Desired Speed Distribution (mph)*	N/A	35 to 55	35.0 - 47.0	42.3 - 48.5
Number of Observed Preceding Vehicles	2	1 to 4	3	4
Average Standstill Distance (ft.)	6.56	3.28 - 9.84	7.51	6.56
Waiting Time Before Diffusion (s)	60	20 to 60	20	60
Minimum Headway (ft.)	1.64	1.64 to 23	6.99	1.64

** More than simply a range, this is also a curve, these are shown below.*



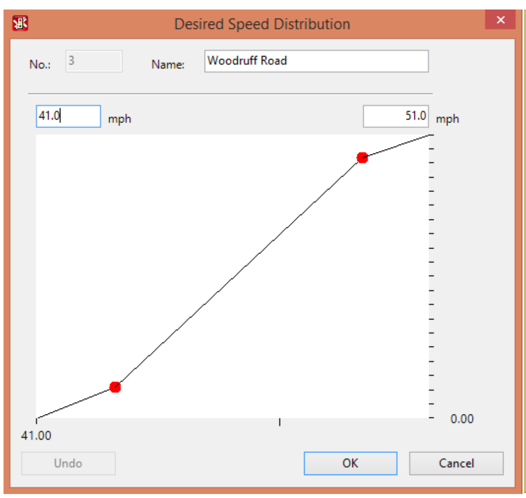


Speed Distribution for Woodruff Road *Speed Distribution for Wade Hampton Blvd.*

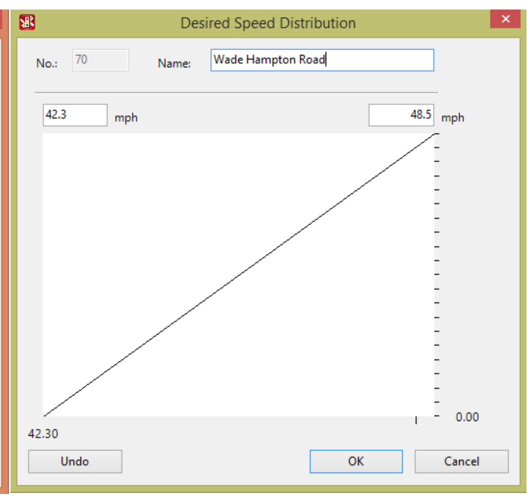
***Table 12**: Calibration Parameters Used in Base Model Calibration (for Demand Responsive Access Management Strategies)*

Parameter	VISSIM Default	Acceptable Range *(25)*	Selected Value	
			Woodruff	Wade Hampton
Emergency Stopping Distance (ft.)	16.4	6.6 to 23	16.4	16.4
Lane-Change Distance (ft.)	656	492 to 984	656	656
Desired Speed Distribution (mph)*	N/A	35 to 55	41.0 – 51.0	42.3 - 48.5
Number of Observed Preceding Vehicles	2	1 to 4	4	4
Average Standstill Distance (ft.)	6.56	3.28 - 9.84	3.31	6.56
Waiting Time Before Diffusion (s)	60	20 to 60	20	60
Minimum Headway (ft.)	1.64	1.64 to 23	1.70	1.64

** More than simply a range, this is also a curve, these are shown below.*



Speed Distribution for Woodruff Road



Speed Distribution for Wade Hampton Blvd.

Additionally, an important calibration parameter is acceptable gap time for median and driveway turning movements. Two sources for acceptable minimum gap times were found in the literature *(19, 26)*, one addressing left and right turns and the other addressing U-turns. Table 13 below shows the suggested gap times for each of these sources. These values were adopted for use in the base models for both corridors.

***Table 13**: Minimum Gap Acceptance Times for Turning Movements*

Turning Movement	Minimum Suggested Gap Acceptance Time (s)	
	Liu et al. *(26)*	Siddiqui *(19)*
U-turns	6.3 (2-lanes) | 5.1 (3-lanes)	N/A
Left-turns in	N/A	3.6
Left-turns out	N/A	3.1
Right-turns	N/A	3.0

Another important factor is turning speed of right-turners as this has the potential to impact following right-lane mainline traffic and thus mainline travel times. One typical right-turn speed cited in the literature is 15 mph *(74)*. Another study observed right-turning speeds between 10 and 18 mph *(75)*. Given these values, a right-turning speed of 14 mph was used in this study. This speed was also used as the speed for TWLTL traffic.

The TWLWL was modeled using overlapping links and connectors, controlling TWLTL traffic through priority rules and conflict areas with the aforementioned minimum gap times. An example of the TWLTL modeling approach is shown below in Figure 3:

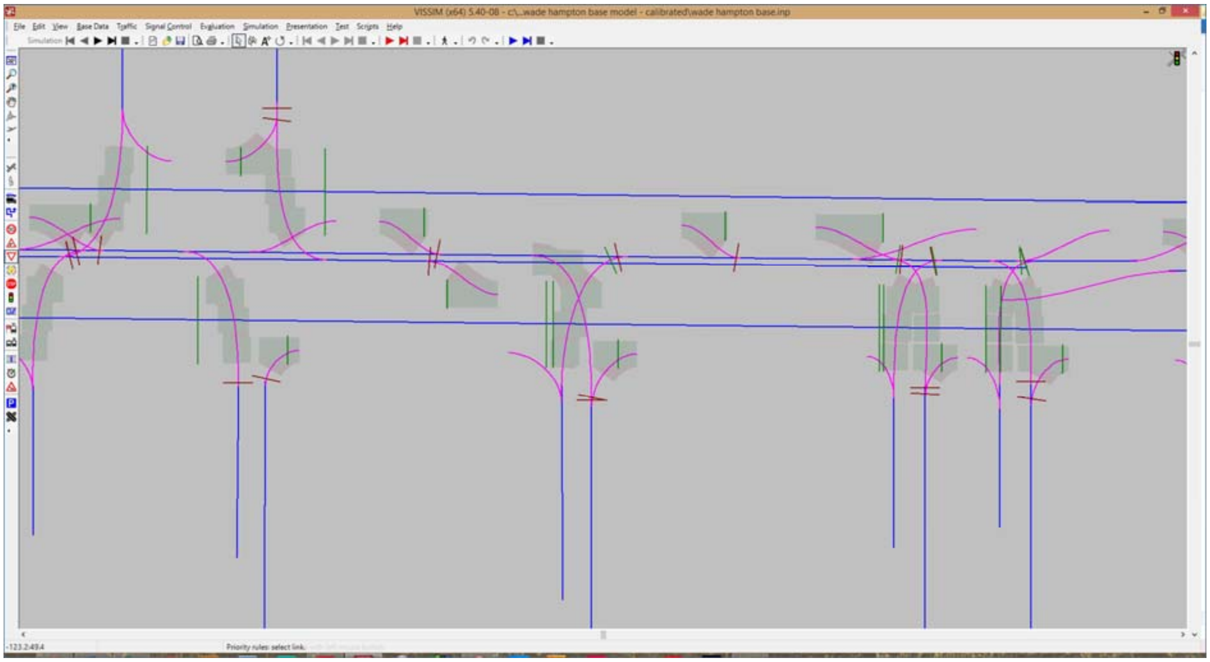


Figure 3: *TWLTL Modeling using Priority Rules and Conflict Areas*

The model was run 10 times, each time with a different random seed. The average travel time results for Woodruff Road and Wade Hampton Blvd. for the Traditional Access Management Strategies are shown in Table 14 and Table 15 respectively. The average of the travel times had less than a 10% difference, and thus, the models were considered calibrated. Tables 16 and 17, show the travel time results for the 5-hour base model calibration runs for Woodruff Road and Wade Hampton Blvd., respectively These results represent the average travel time during the peak hour of that 5-hour run. The calibrated models represent the “Existing Conditions” scenarios to which all alternative scenarios (discussed in the subsequent sections) will be compared.

***Table 14**: Travel Time Model Calibration Results for 1-Hour Simulation Run* *(Woodruff)*

	East-Bound			*West-Bound*		
	Field (s)	*VISSIM (s)*	*% Difference*	*Field (s)*	*VISSIM (s)*	*% Difference*
Average	307	295	**4%**	268	259	**3.5%**
St. Dev.	43	35		29	2.5	

***Table 15**: Travel Time Model Calibration Results for 1-Hour Simulation Run* *(Wade Hampton)*

	East-Bound			*West-Bound*		
	Field (s)	*VISSIM (s)*	*% Difference*	*Field (s)*	*VISSIM (s)*	*% Difference*
Average	118	118	**0%**	128	122	**5%**
St. Dev.	26	0.87		18	1.3	

***Table 16**: Travel Time Model Calibration Results for 5-Hour Simulation Run* *(Woodruff)*

	East-Bound			*West-Bound*		
	Field (s)	*VISSIM (s)*	*% Difference*	*Field (s)*	*VISSIM (s)*	*% Difference*
Average	307	338	**9.6%**	268	256	**4.6%**
St. Dev.	43	41		29	3.5	

***Table 17**: Travel Time Model Calibration Results for 5-Hour Simulation Run* *(Wade Hampton)*

	East-Bound			*West-Bound*		
	Field (s)	*VISSIM (s)*	*% Difference*	*Field (s)*	*VISSIM (s)*	*% Difference*
Average	118	117	**1%**	128	122	**5%**
St. Dev.	26	0.87		18	1.3	

3.2. Traditional Access Management Strategy Scenarios

Recall that the four access management strategies of interest for this objective are: (i) Access Spacing, (ii) Corner Clearances, (iii) Access Restriction of Selected Driveways, and (iv) Non-Traversable Medians. To test the operational impacts of each of these strategies, four alternative scenarios were developed. Each alternative scenario was tested both on the 5-lane Woodruff Road segment and the 7-lane Wade Hampton Road segment. The simulation run time was 70 minutes, which included 10 minutes of 'warm up' time and 60 minutes of data collection. This 60 minutes represented peak hour volumes, as collected in the field. The calibrated base models for both corridors were run for this simulation time and mainline travel time across the corridor as well as travel times from driveways to destinations were collected as measures of effectiveness. These same measures of effectiveness were analyzed for the four alternative scenarios to test each access management strategy, described below.

3.2.1. Access Spacing

In order to test access spacing, a criteria for determining acceptable spacing needed to be established. The literature review in Chapter 2 referenced different spacing criteria of 36 states. Many of these values were between 300 to 400 feet. South Carolina DOT's spacing criteria, 325 ft. was also in this range. After review of both the corridors, it was evident that they were not consistent with this spacing. Therefore, 325 feet was chosen as the spacing to test. In order to alter the existing corridors to have at this minimum spacing, driveways were consolidated along the corridor – in other words, certain driveways were closed and their ingress and egress traffic added to nearby driveways to achieve the desired spacing of 325 ft. Driveways within the minimum corner clearance were not closed so long as there was adequate spacing to the next driveway. Consideration was given to whether there were side-streets and/or alternate routes from the remaining driveways to the land-uses serviced by the closed driveways. Non-signalized intersections were not closed and major-traffic generators were given priority to remain 'open.' Signals were not optimized as no turning volumes were altered in this scenario. Figures 4, 5, and 6 for Woodruff Road (and Figures 7, 8, and 9 for Wade Hampton Blvd.) on the following pages (split into segments for viewing) show the driveways that were consolidated for each corridor. The pink markers represent the location of the remaining driveway whereas the green markers represent the driveways that are being consolidated (in the yellow boxes) to form the new driveway. Along Woodruff road, the number of driveways in resulting alternative scenario was reduced from 62 to 28 and the driveway density from 41 driveways/mile to 19 driveways/mile. Along Wade Hampton Blvd., the number of driveways in the resulting alternative scenario was reduced from 66 to 24 and the driveway density from 61 driveways/mile to 22 driveways/mile.

Woodruff Road Driveway Consolidation

Figure 4 – *Consolidation of Driveways along Woodruff Road (continued on next page)*

Woodruff Road Driveway Consolidation (cont.)

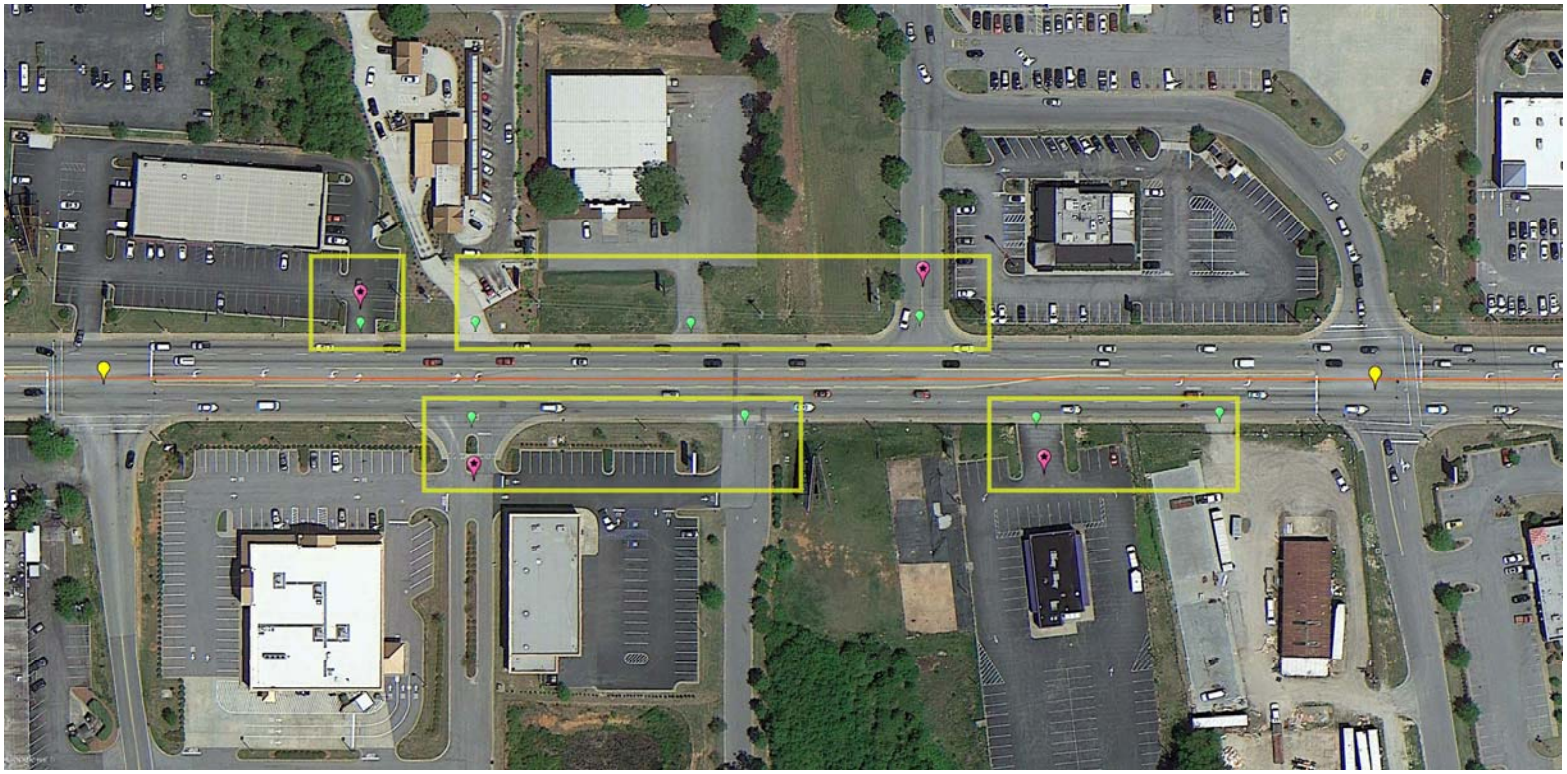


***Figure 4b** – Consolidation of Driveways along Woodruff Road* *(continued on next page)*

Woodruff Road Driveway Consolidation (cont.)

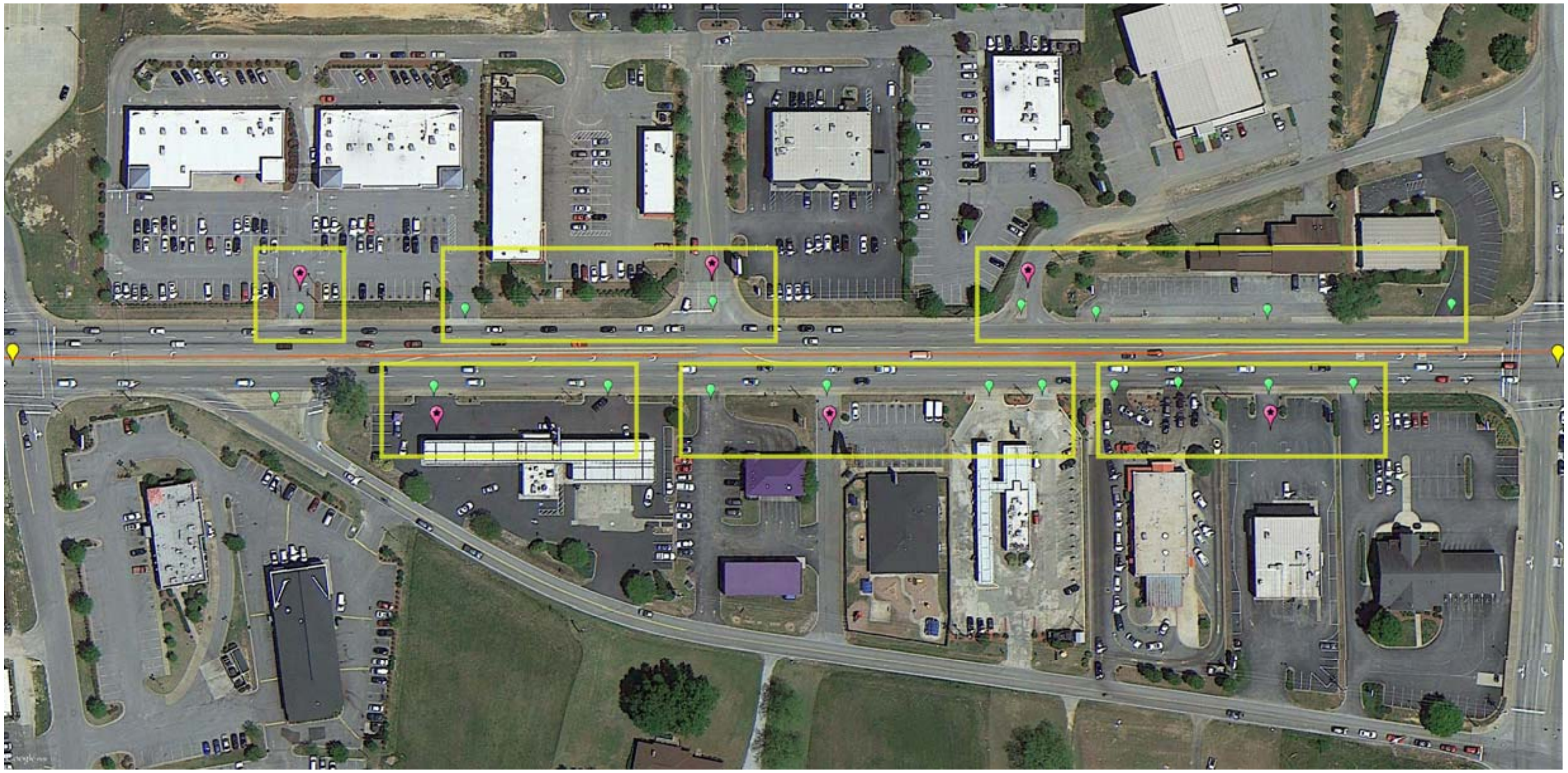


Figure 4c *– Consolidation of Driveways along Woodruff Road (continued on next page)*

Woodruff Road Driveway Consolidation (cont.)

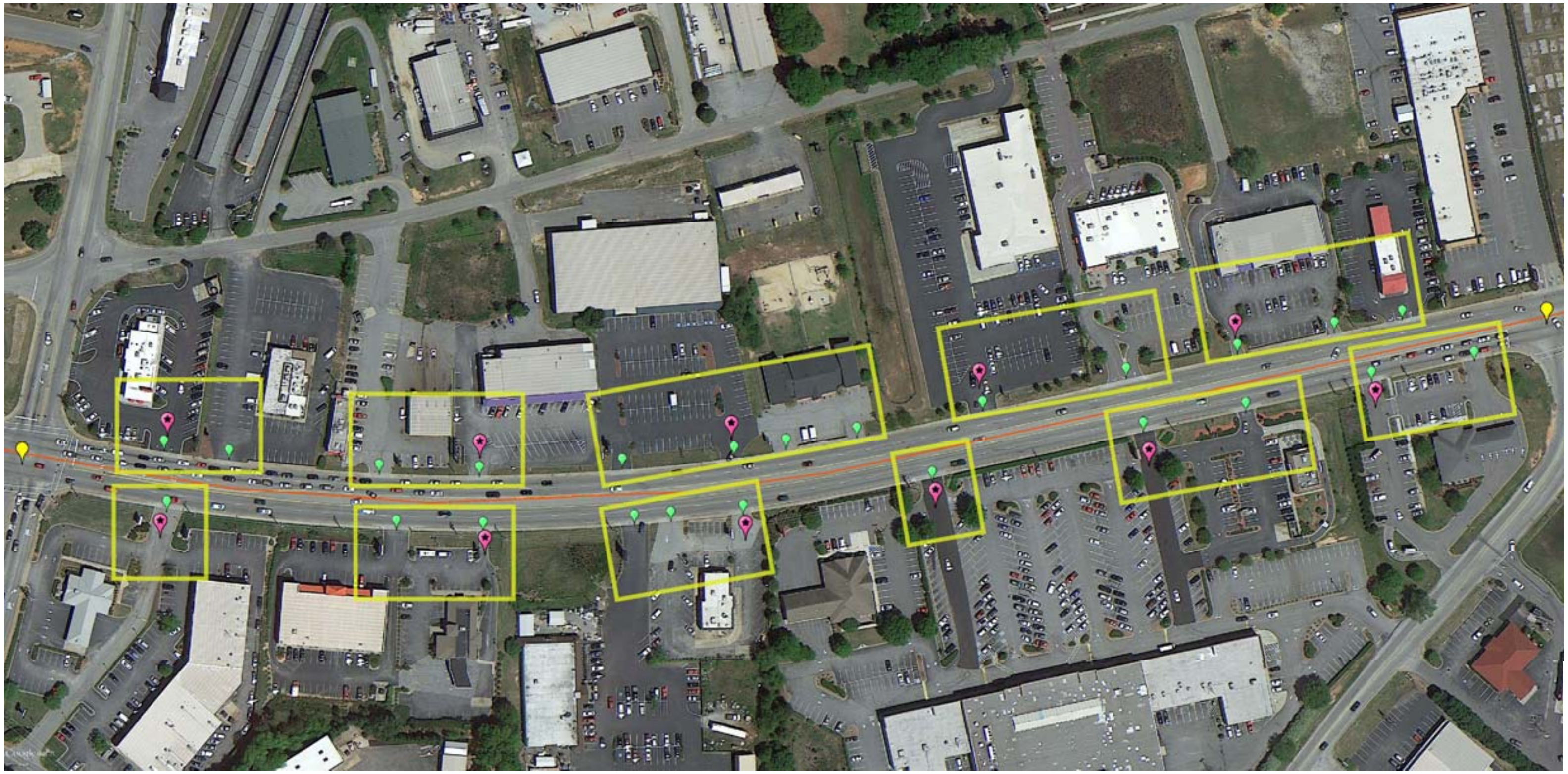


***Figure 4d** – Consolidation of Driveways along Woodruff Road (continued on next page)*

Woodruff Road Driveway Consolidation (cont.)

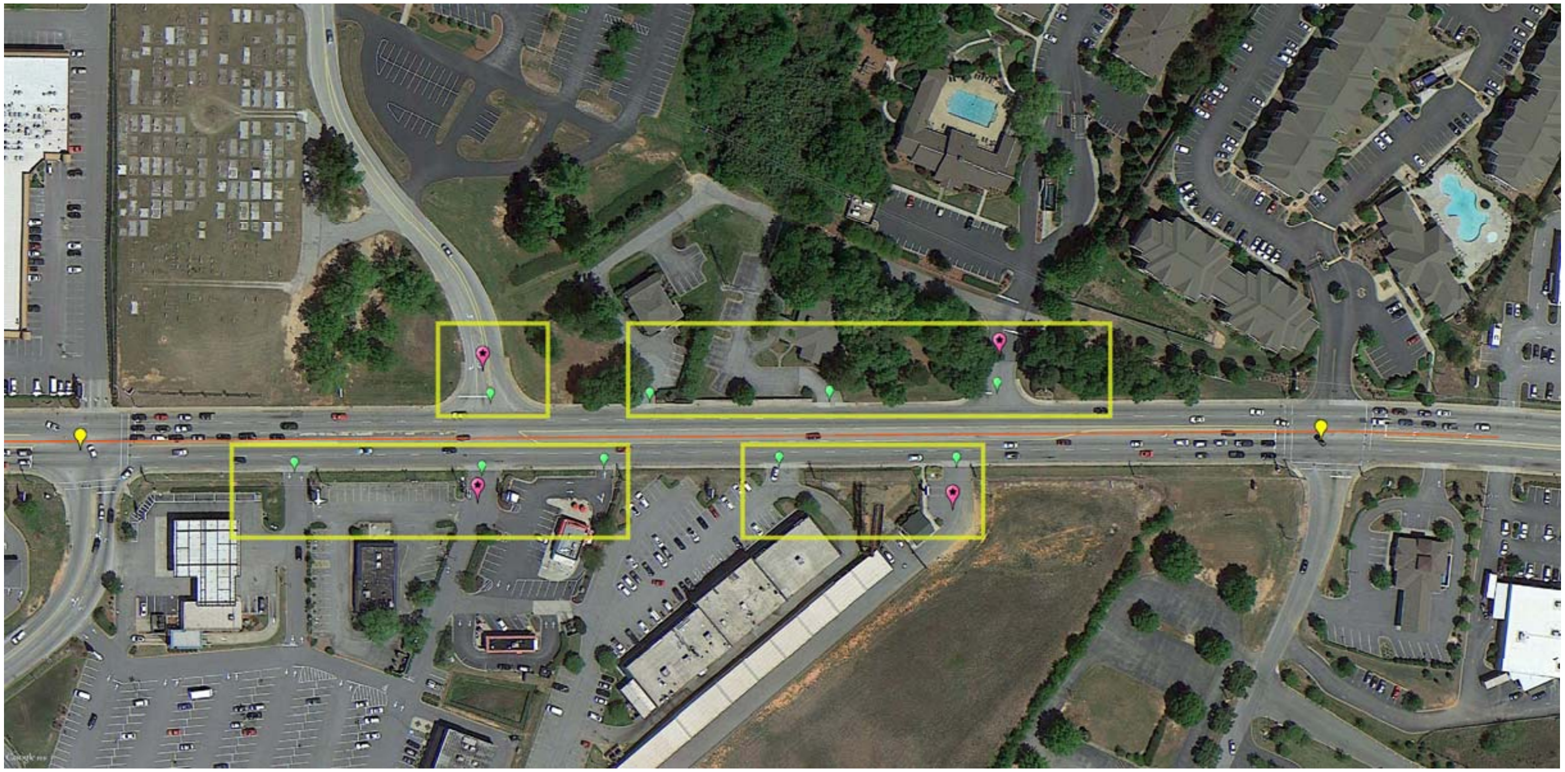
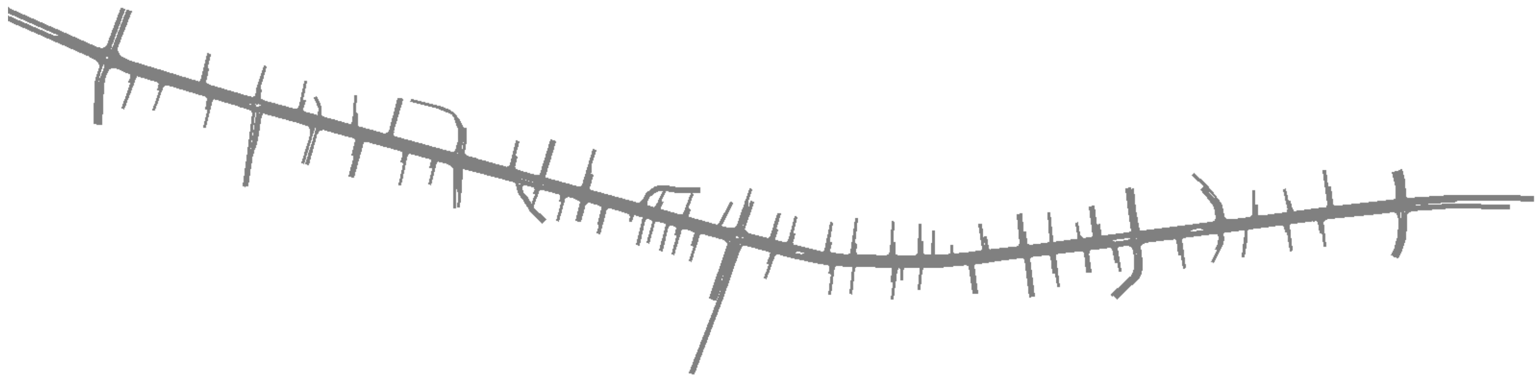


***Figure 4e** – Consolidation of Driveways along Woodruff Road*

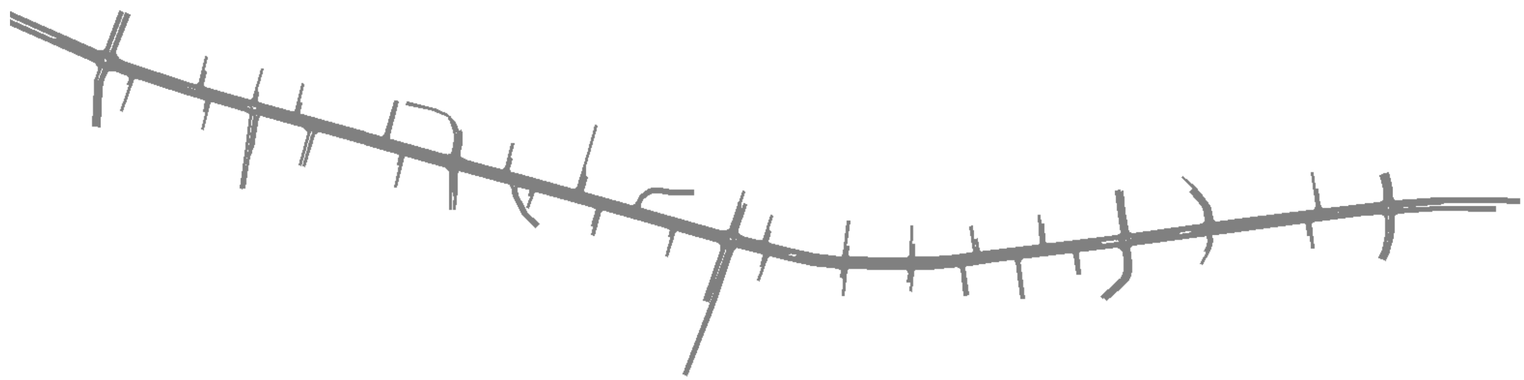
Woodruff Road Driveway Consolidation (cont.)

Figure 5 *–Resulting Driveways along Entire Woodruff Road Corridor*

Woodruff Road Driveway Consolidation (cont.)



Base Model (Before Consolidation)



Consolidated Driveways to Achieve 325' Driveway Spacing SCDOT AMRS Criteria (After Consolidation)

Figure 6 *–Woodruff Road Vissim Models Before and After Driveway Consolidation*

Wade Hampton Blvd. Driveway Consolidation

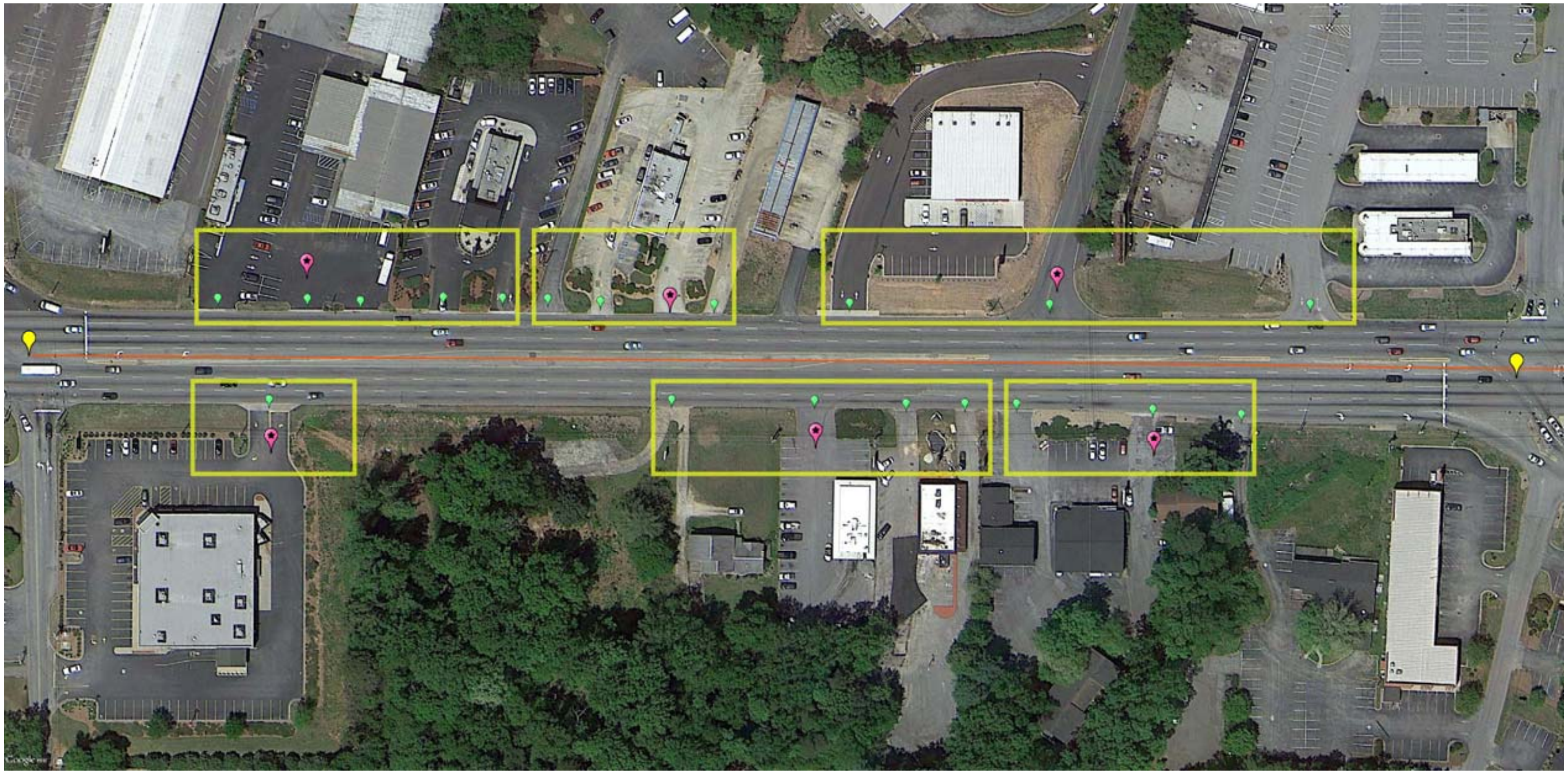


Figure 7a – *Consolidation of Driveways along Wade Hampton Blvd. (continued on next page)*

Wade Hampton Blvd. Driveway Consolidation (cont.)

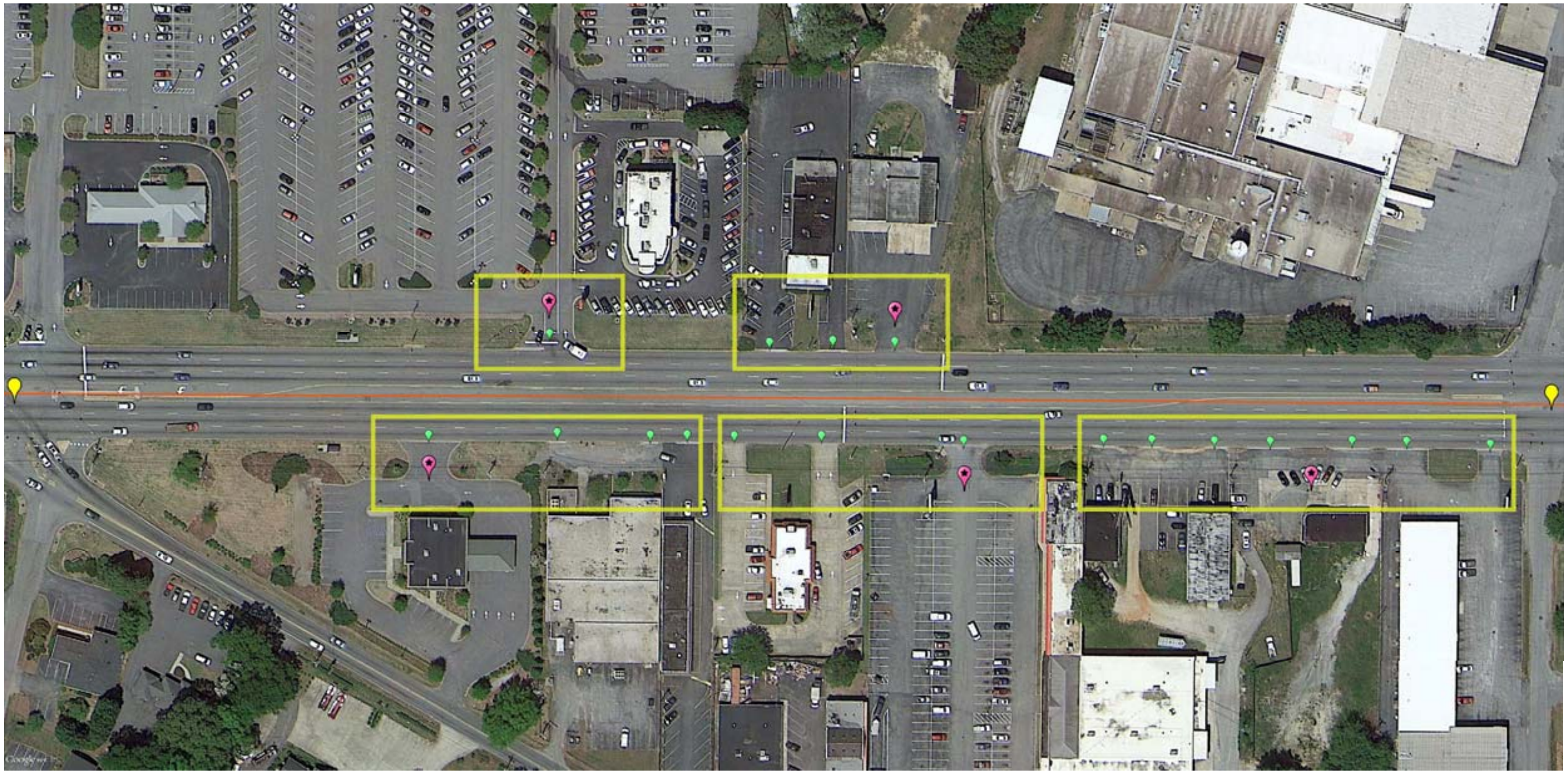


***Figure 7b** – Consolidation of Driveways along Wade Hampton Blvd. (continued on next page)*

Wade Hampton Blvd. Driveway Consolidation (cont.)

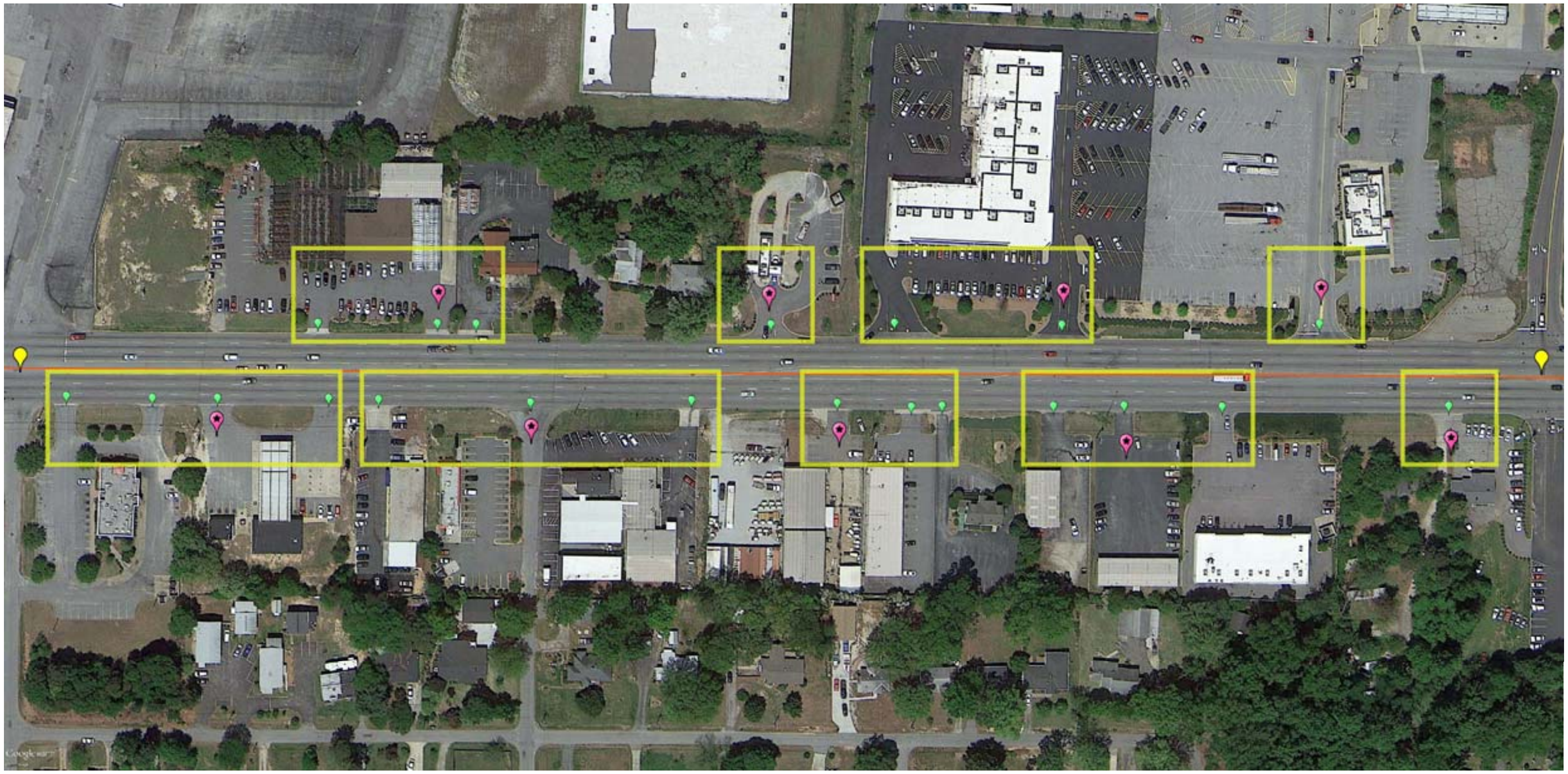


***Figure** 7c – Consolidation of Driveways along Wade Hampton Blvd. (continued on next page)*

Wade Hampton Blvd. Driveway Consolidation (cont.)

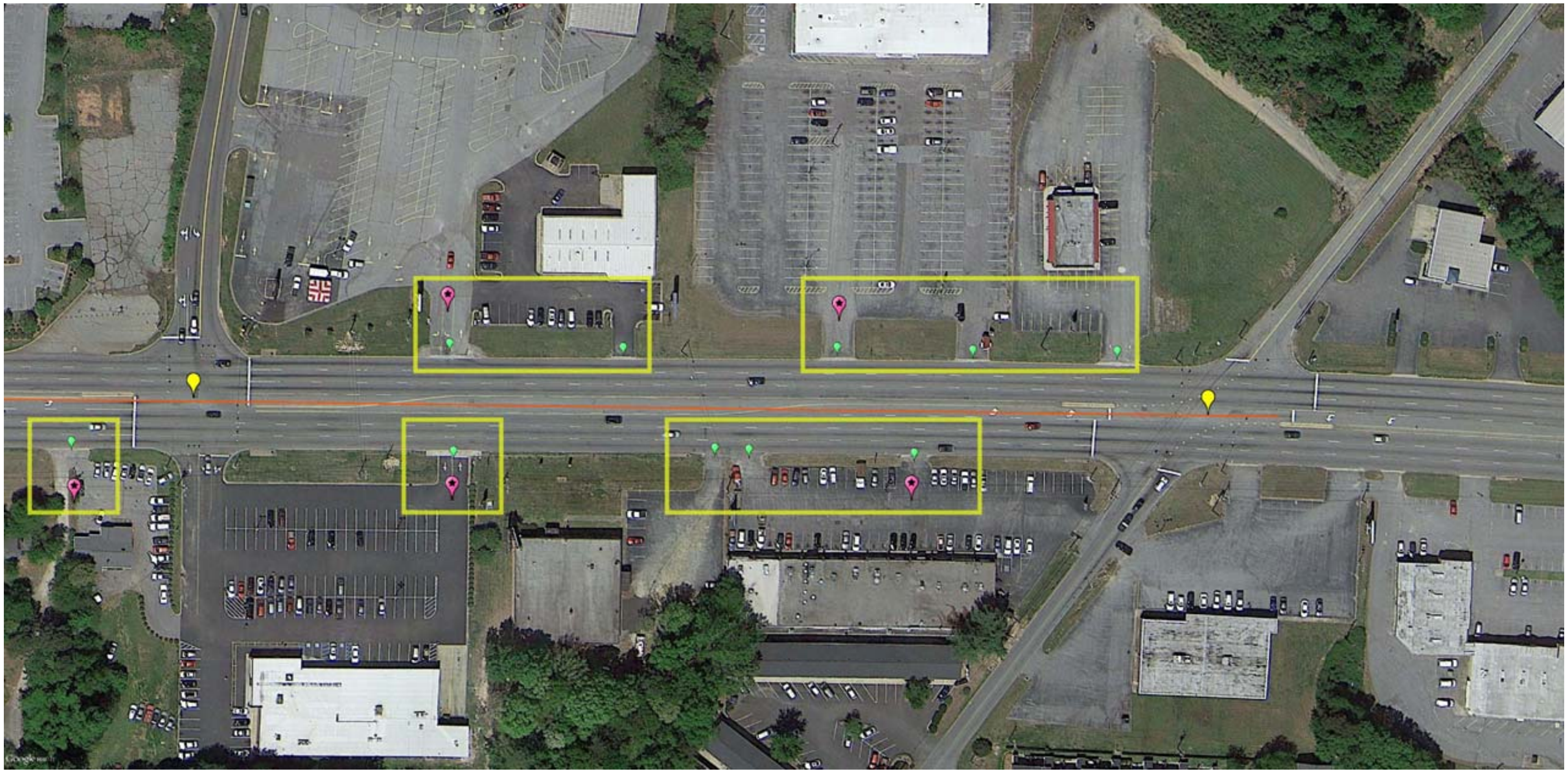
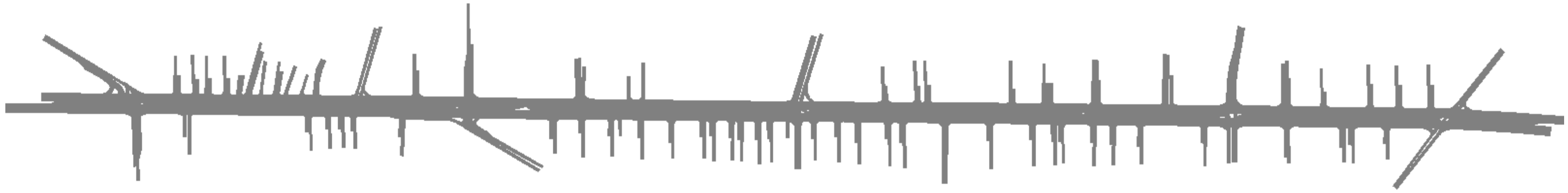


***Figure 7d** – Consolidation of Driveways along Wade Hampton Blvd.*

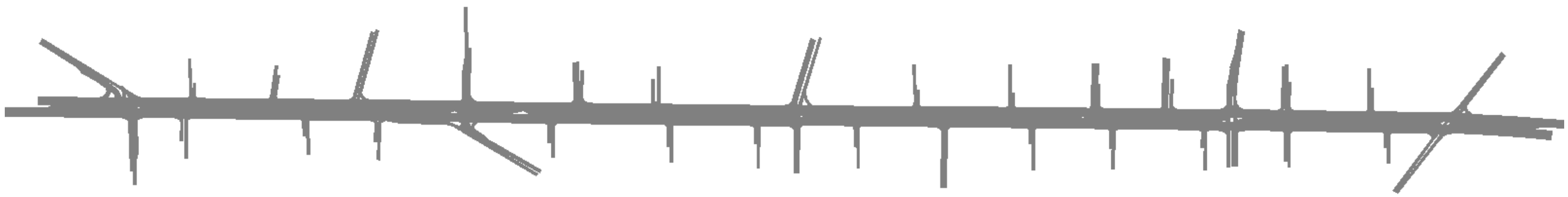
Wade Hampton Blvd. Driveway Consolidation (cont.)

Figure 8 *–Resulting Driveways along Entire Wade Hampton Blvd. Corridor*

Wade Hampton Blvd. Driveway Consolidation (cont.)



Base Model (Before Consolidation)



Consolidated Driveways to Achieve 325' Driveway Spacing SCDOT AMRS Criteria (After Consolidation)

***Figure 9**: Wade Hampton Blvd. Vissim Models Before and After Driveway Consolidation*

3.2.2. Corner Clearance

Similarly to the access spacing scenario, in order to test the impact of corner clearance, a criteria for determining acceptable corner clearance needed to be established. Most state corner clearance standards cited values in the 200-400 ft. range. South Carolina's standard, 325 ft., is also in this range. For the sake of consistency, South Carolina's values were chosen for testing in this scenario as well. Similar to the access spacing test scenario, driveways that were within the minimum of 325 were closed and their ingress and egress traffic added to nearby driveways that were located beyond the minimum acceptable corner clearance. In many cases, however, the traffic from closed driveways had to be routed to the nearest signal as no other driveways were available. In view of this, the signal splits, cycle length, and coordination were optimized in this scenario for both corridors. As was similarly displayed for the previous *Access Spacing* alternative scenario, Figures 10, 11 and 12 for Woodruff Road (and Figures 13, 14, and 15 for Wade Hampton Blvd) below show the driveways which were closed to achieve 325' corner clearance as well as the corresponding driveway or signal to which the traffic was routed.

Woodruff Road. Driveway Corner Clearance Closures

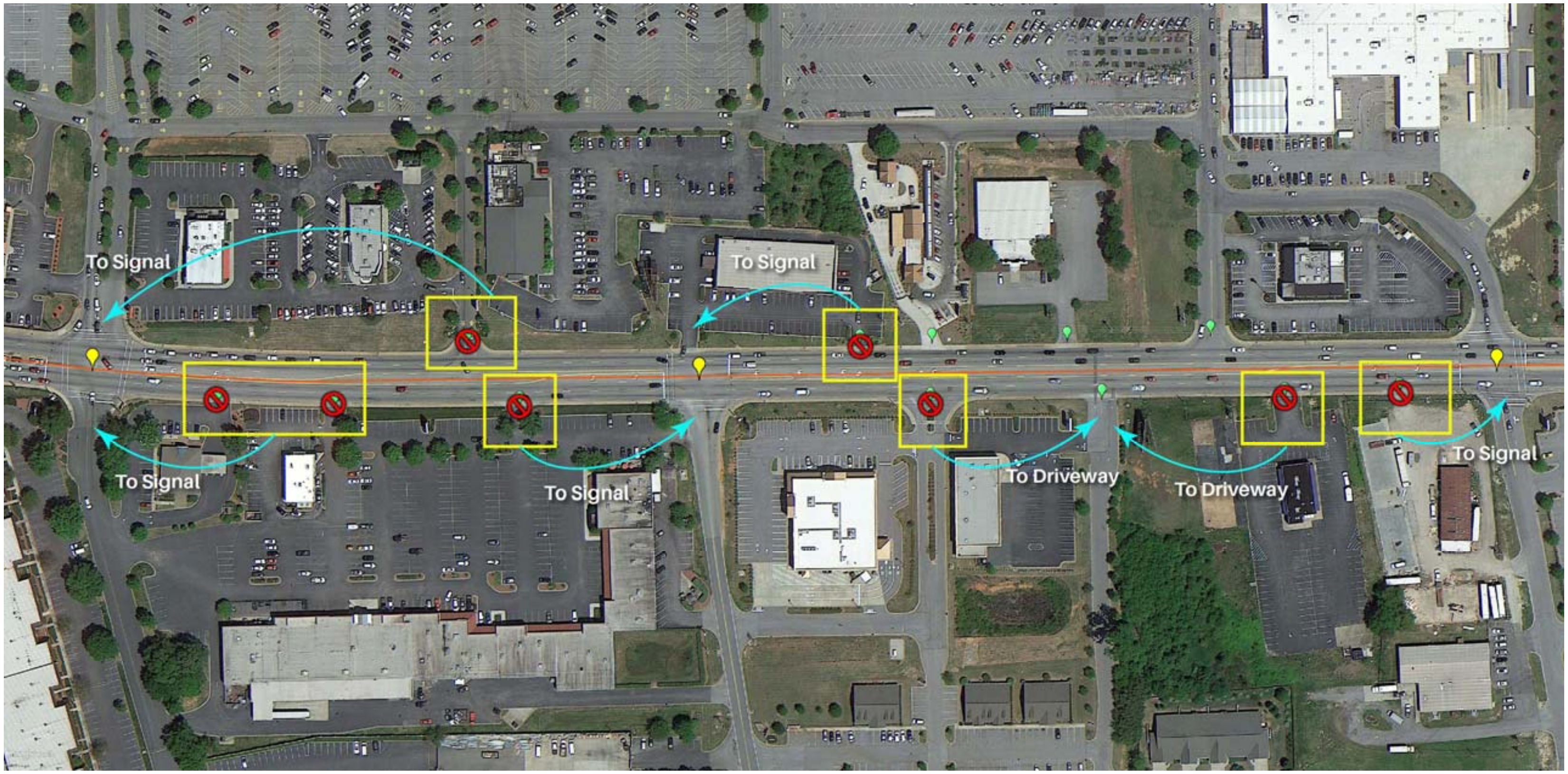


Figure 10a – *Closing of Driveways within Minimum Acceptable Corner Clearance along Woodruff Road (cont. on following pages).*

Woodruff Road. Driveway Corner Clearance Closures (cont.)

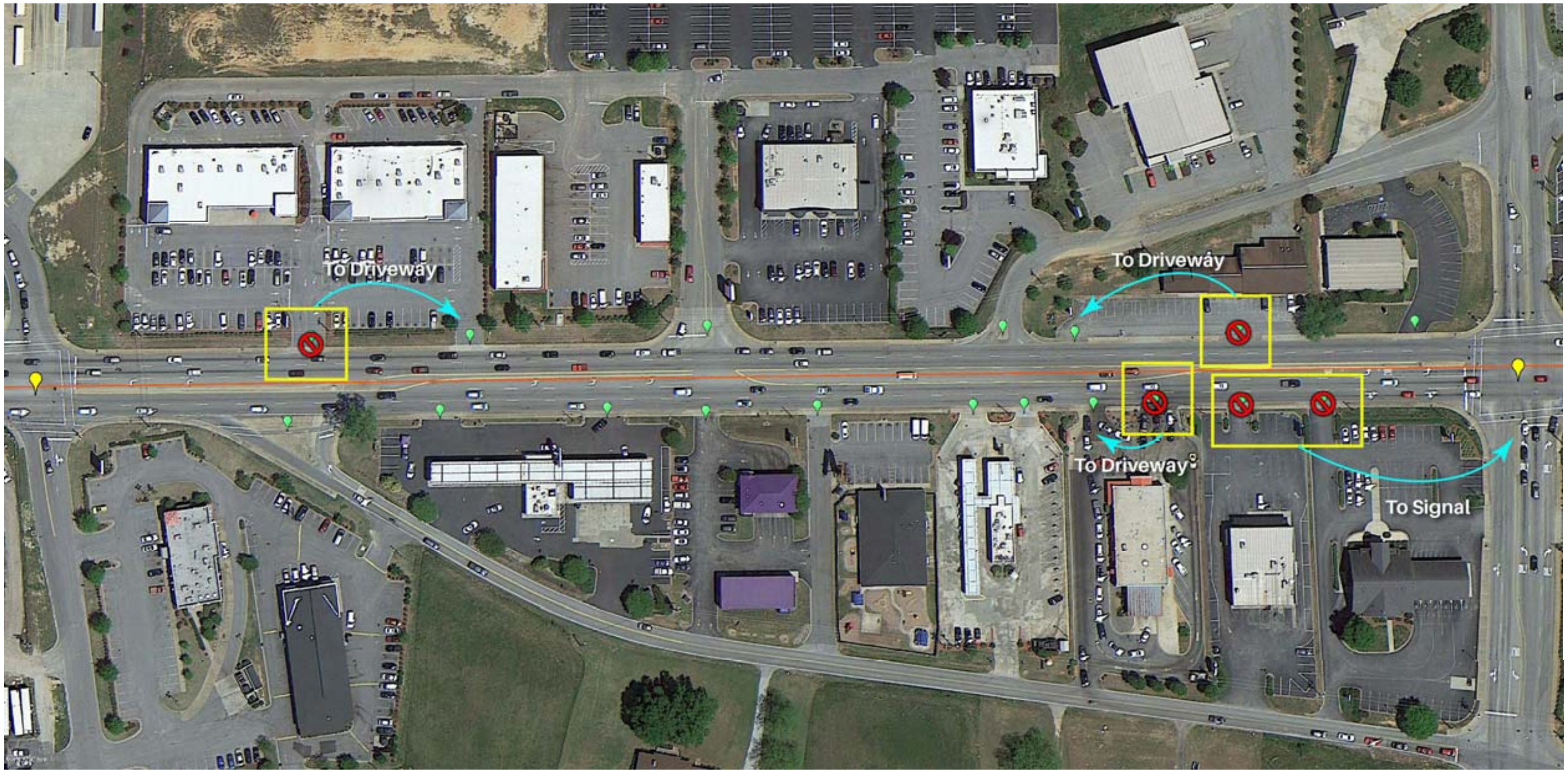


Figure 10b – *Closing of Driveways within Minimum Acceptable Corner Clearance along Woodruff Road (cont. on following pages).*

Woodruff Road. Driveway Corner Clearance Closures (cont.)

Figure 10c – *Closing of Driveways within Minimum Acceptable Corner Clearance along Woodruff Road (cont. on following pages).*

Woodruff Road. Driveway Corner Clearance Closures (cont.)

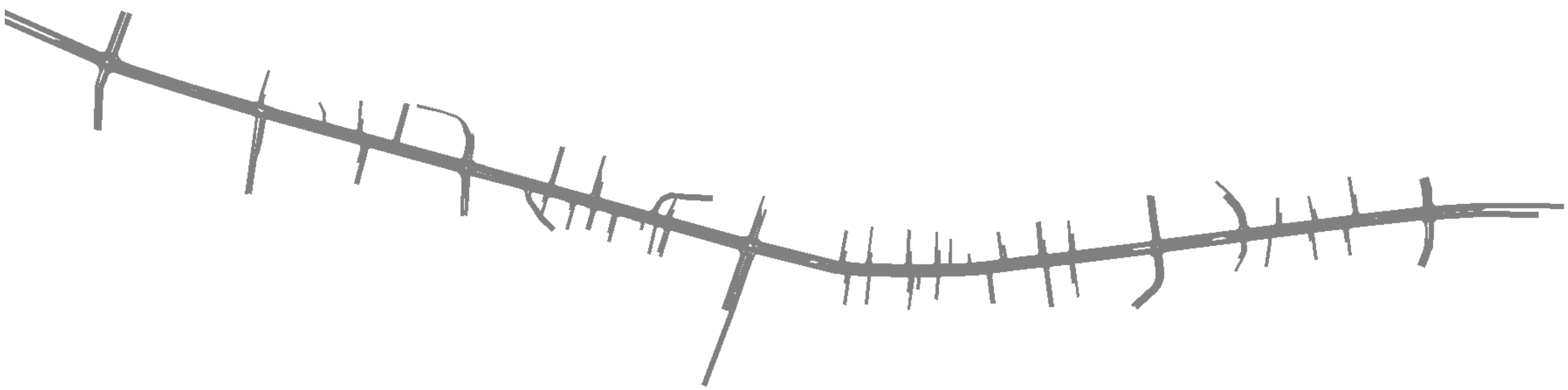
***Figure 10d –** Closing of Driveways within Minimum Acceptable Corner Clearance along Woodruff Road.*

Woodruff Road Driveway Corner Clearance Closures (cont.)

***Figure 11** – Closing of Driveways within Minimum Acceptable Corner Clearance along Entire Woodruff Road.*

Woodruff Road Driveway Corner Clearance Closures (cont.)

Base Model (Before Closures for Corner Clearance)



Consolidated Driveways to Achieve 325' Corner Clearance SCDOT AMRS Criteria (After Closures)

***Figure 12**: Woodruff Road Vissim Models Before and After Corner Clearance Driveway Closures*

Wade Hampton Blvd. Driveway Corner Clearance Closures

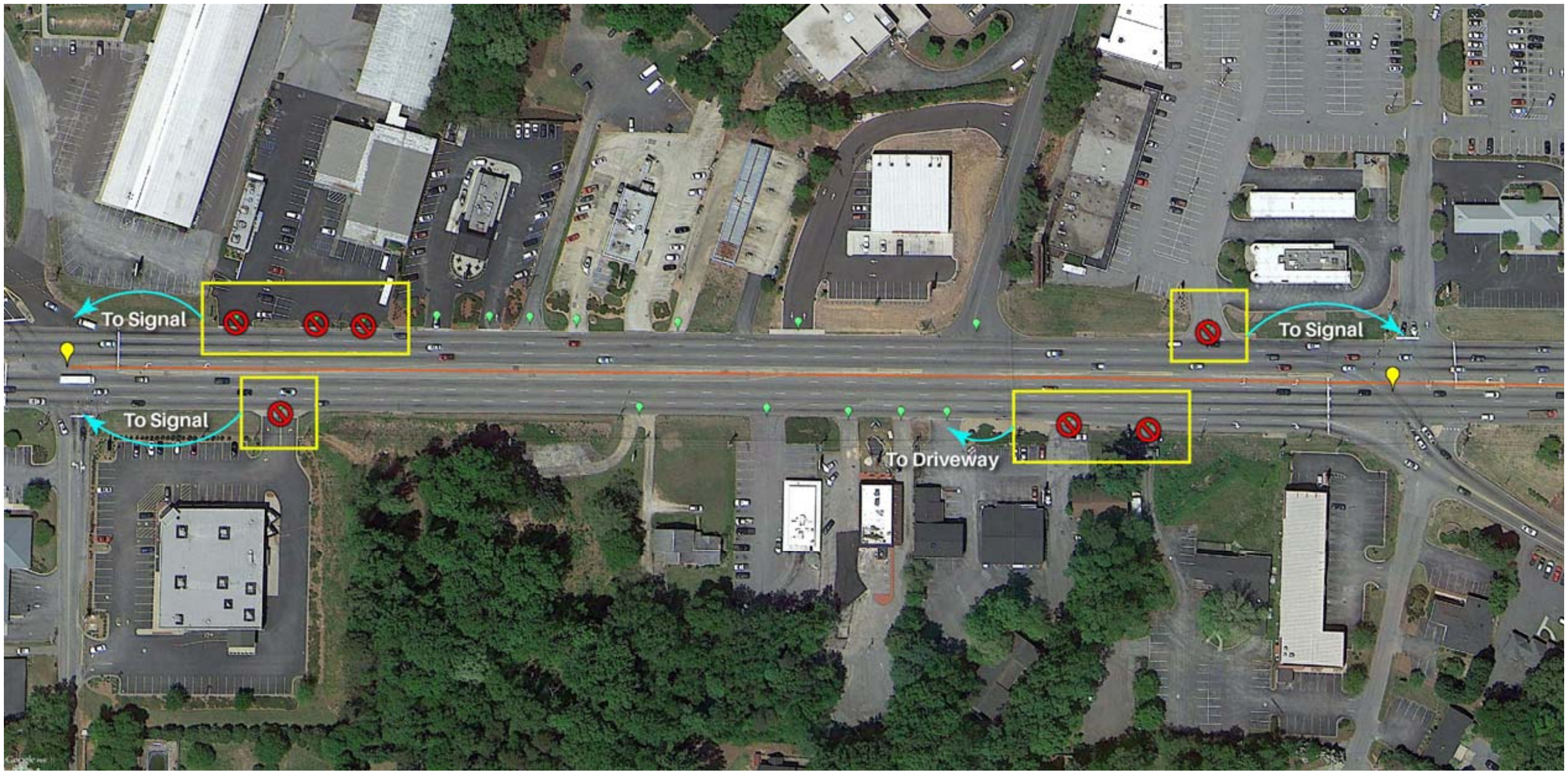


Figure 13a *– Closing of Driveways within Minimum Acceptable Corner Clearance along Wade Hampton Blvd (cont. on following pages).*

Wade Hampton Blvd. Driveway Corner Clearance Closures (cont.)

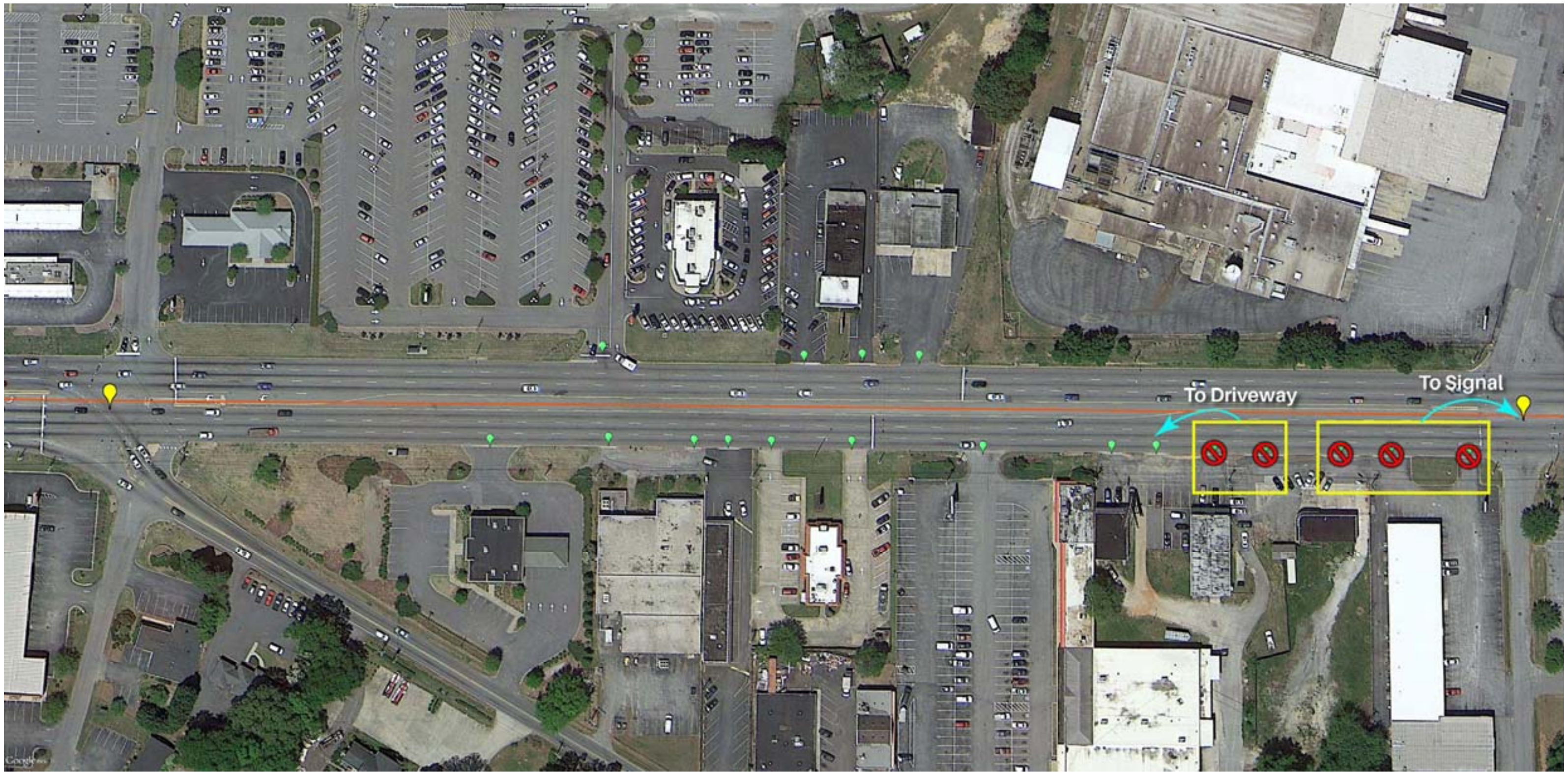


Figure 13b *– Closing of Driveways within Minimum Acceptable Corner Clearance along Wade Hampton Blvd (cont. on following pages).*

Wade Hampton Blvd. Driveway Corner Clearance Closures (cont.)

Figure 13c – *Closing of Driveways within Minimum Acceptable Corner Clearance along Wade Hampton Blvd (cont. on following pages).*

Wade Hampton Blvd. Driveway Corner Clearance Closures (cont.)

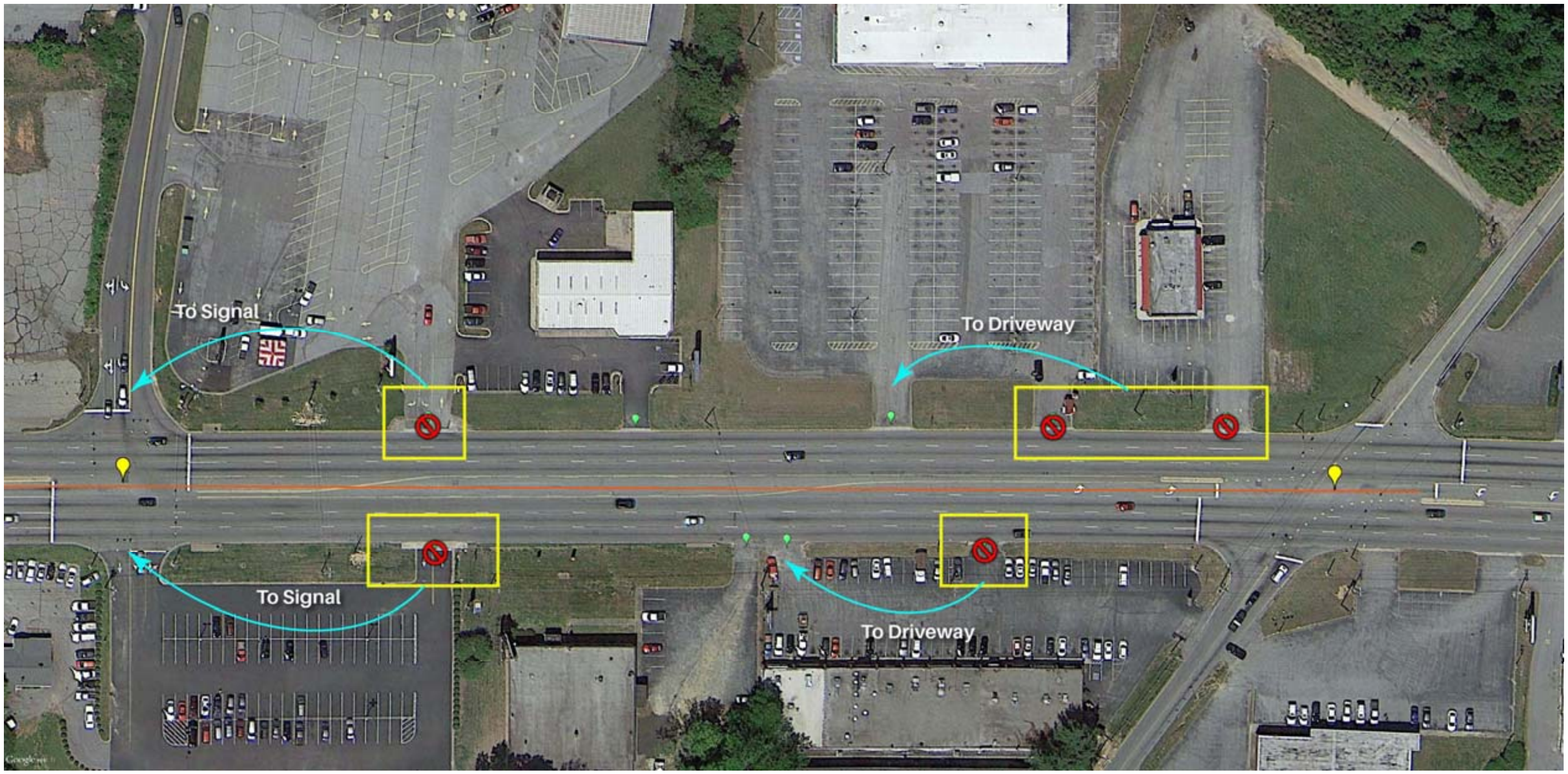


***Figure 13d –** Closing of Driveways within Minimum Acceptable Corner Clearance along Wade Hampton Blvd.*

Wade Hampton Blvd. Driveway Corner Clearance Closures (cont.)

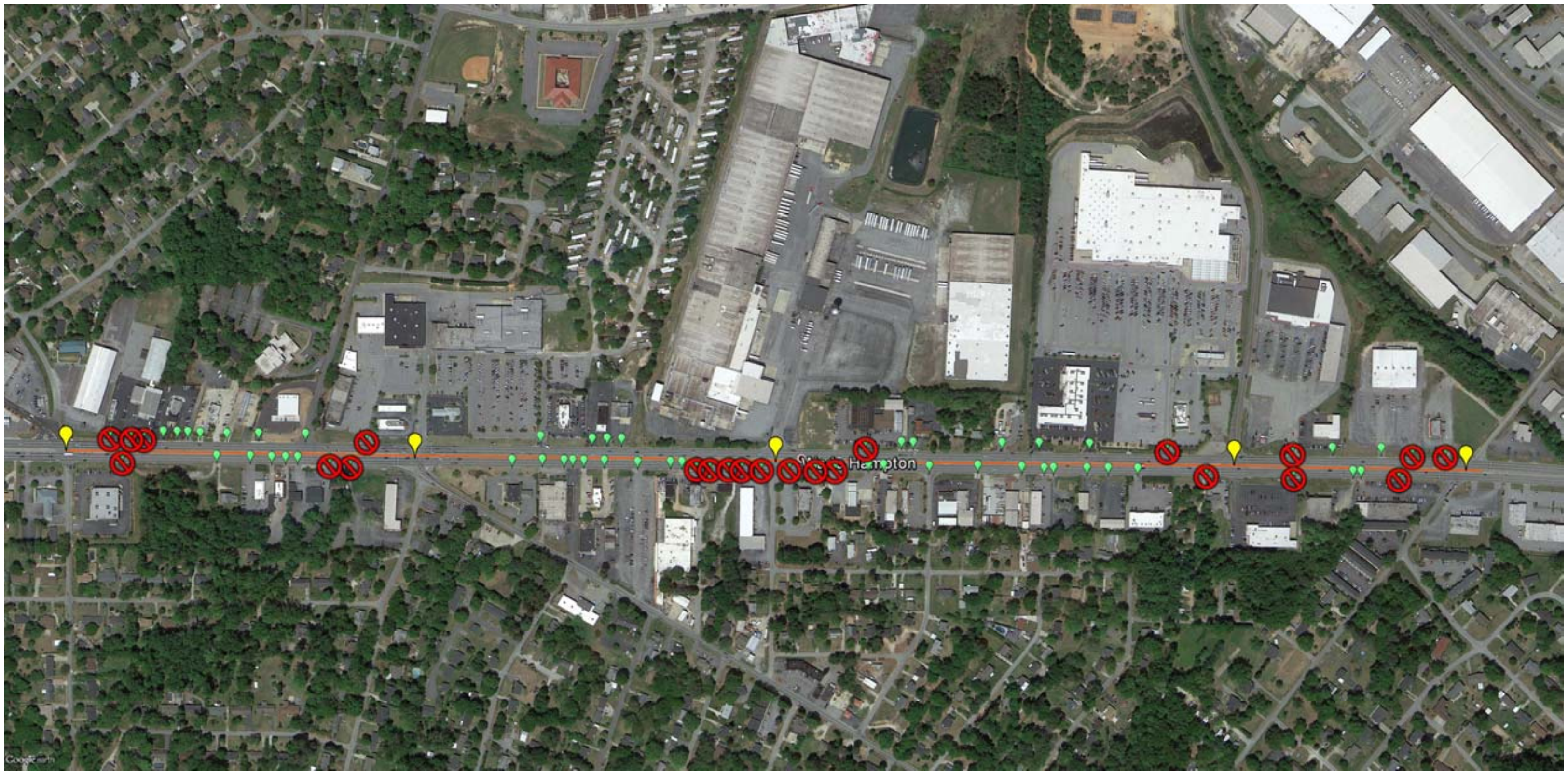
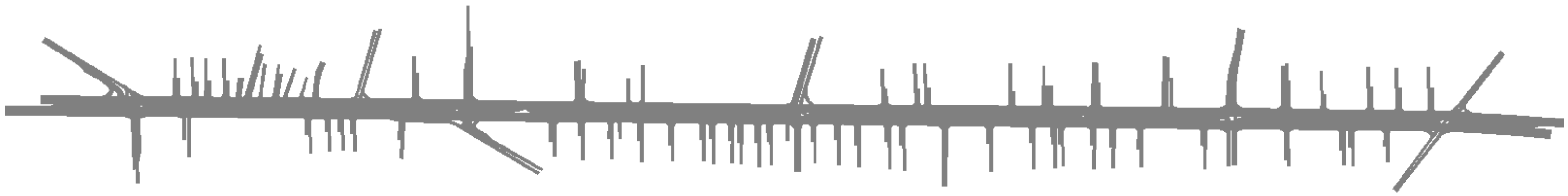
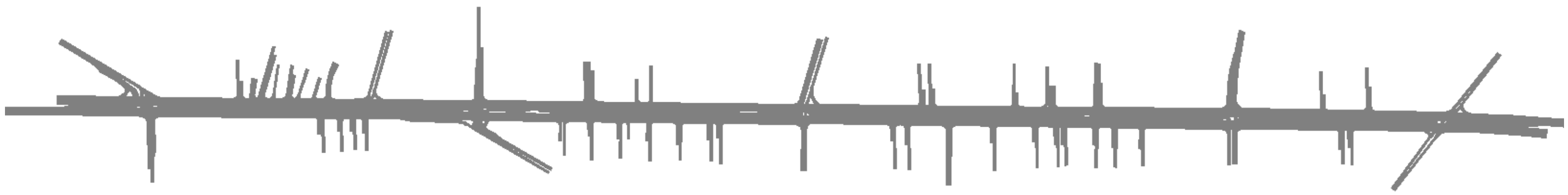


Figure 14 – *Closing of Driveways within Minimum Acceptable Corner Clearance along Entire Wade Hampton Blvd.*

Wade Hampton Blvd. Driveway Corner Clearance Closures (cont.)



Base Model (Before Closures for Corner Clearance)



Consolidated Driveways to Achieve 325' Corner Clearance SCDOT AMRS Criteria (After Closures)

***Figure 15**: Wade Hampton Blvd. Vissim Models Before and After Corner Clearance Driveway Closures*

3.2.3. Access Restriction of Selected Driveways

In order to test the effect of restricting access to only selected driveways, some criteria for which driveways to restrict was needed. In current practice, the most common recommendation for when to restrict access to right-in/right-out is when minimum corner clearance cannot be met and when driveways are within the influence area of an intersection such that they are frequently blocked by queues. Again, for the sake of consistency, South Carolina DOT's corner clearance standard was used to select driveways for access restriction to right-in/right-out based on this common recommendation in current practice. South Carolina stipulates that the minimum corner clearance is 325 ft. for a full access driveway and 150 ft. for a right-in/right-out driveway. However, in order to test the effect of restricting access without closing any access points, in this scenario, all driveways located 325 ft. or closer to an intersection were restricted to right-in/right-out, even those closer than 150 ft. No driveways were removed – only their access was altered. In other words, all the driveways which were closed (and had their traffic rerouted to an adjacent signal or driveway) in the previous scenario, were instead changed to right-in/right-out access. To avoid unnecessary repetition, in order to see which driveways were altered to right-in/right-out, please refer to the figures from the previous section. For the driveways which had their access restricted to right-in/right-out, the left-in and left-out volumes were redirected using RTUT movements at the nearest feasible signalized intersection. 'Nearest feasible' was determined using the suggested offset distances provided by Lu et al. *(16)*: 550 ft. on 4 lane roads and 750 ft. on 6 lane roads. Because signal turning and thru volumes were altered in this scenario, signal optimization of splits, cycle, and coordination was performed.

3.2.4. Non-Traversable Medians

Both corridors analyzed have existing TWLTL median treatment. In order to test the operational impact of non-traversable medians, the TWLTL was converted to a raised median, effectively restricting access at all driveways to right-in/right-out. As with the previous scenario, the left-in and left-out volumes were redirected using RTUT movements at the nearest feasible signalized intersection. 'Nearest feasible' was determined using the suggested offset distances provided by Lu et al. *(16)*: 550 ft. on 4 lane roads and 750 ft. on 6 lane roads. Because signal turning and thru volumes were altered in this scenario, signal optimization of splits, cycle, and coordination was performed. In addition, in order to account for the additional U-turning traffic, left turn storage lanes were lengthened and protected left turn phases were added at signals where they previously did not exist. Another important note for this scenario is the necessary median width – and therefore right-of-way in order to perform U-turns. Figure 16 below from the TRB Access Management Manual gives minimum width of median separators by design vehicle. For the Passenger Car design vehicle (P) the minimum total median width required to perform a U-turn is 30 feet (18 ft. separator + 12 ft. turning lane) for 4-lane roads and 18 feet (6 ft. separator + 12 ft. turning lane) for 6-lane roads. In order to explore the feasibility of this scenario, buffers were drawn along the centerline of each corridor to determine where the extents of the widened road would be.

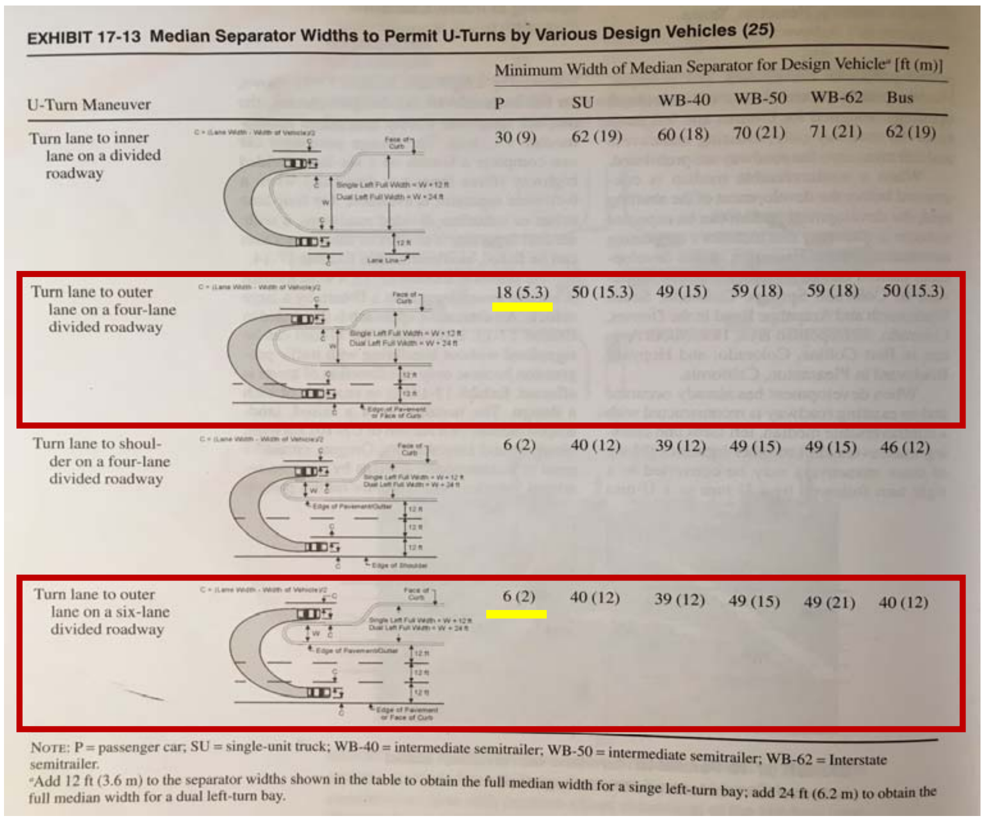


EXHIBIT 17-13 Median Separator Widths to Permit U-Turns by Various Design Vehicles (25)

U-Turn Maneuver	Minimum Width of Median Separator for Design Vehicle* [ft (m)]					
	P	SU	WB-40	WB-50	WB-62	Bus
Turn lane to inner lane on a divided roadway	30 (9)	62 (19)	60 (18)	70 (21)	71 (21)	62 (19)
Turn lane to outer lane on a four-lane divided roadway	18 (5.3)	50 (15.3)	49 (15)	59 (18)	59 (18)	50 (15.3)
Turn lane to shoulder on a four-lane divided roadway	6 (2)	40 (12)	39 (12)	49 (15)	49 (15)	46 (12)
Turn lane to outer lane on a six-lane divided roadway	6 (2)	40 (12)	39 (12)	49 (15)	49 (21)	40 (12)

NOTE: P = passenger car; SU = single-unit truck; WB-40 = intermediate semitrailer; WB-50 = intermediate semitrailer; WB-62 = Interstate semitrailer.

*Add 12 ft (3.6 m) to the separator widths shown in the table to obtain the full median width for a singe left-turn bay; add 24 ft (6.2 m) to obtain the full median width for a dual left-turn bay.

Figure 16: Turning Radii for U-turns for different roadways

For the 4-lane Woodruff Road, the existing width of the road (including sidewalks) is roughly 78 ft. With the additional 18 feet of median width necessary, the required width is 96 ft. For the 6-lane Wade Hampton Blvd., the existing width of the road is roughly 90 ft. With the additional 6 feet of median width necessary, the required width is 96 ft. Figures 17 and 18 below show the 96 ft. buffers for both corridor alignments.

For Woodruff Road, the change in providing the sufficient turning radius would require a fairly significant widening of the road, however, it appears feasible, at least in the sense that the buffer does not intrude on any business fronts. There would be major considerations, of course, concerning parking, driveway throat lengths, etc. For Wade Hampton Blvd., the change is much less significant, and certainly appears feasible, given that the existing three lanes in each direction provide extra turning width for passenger cars.

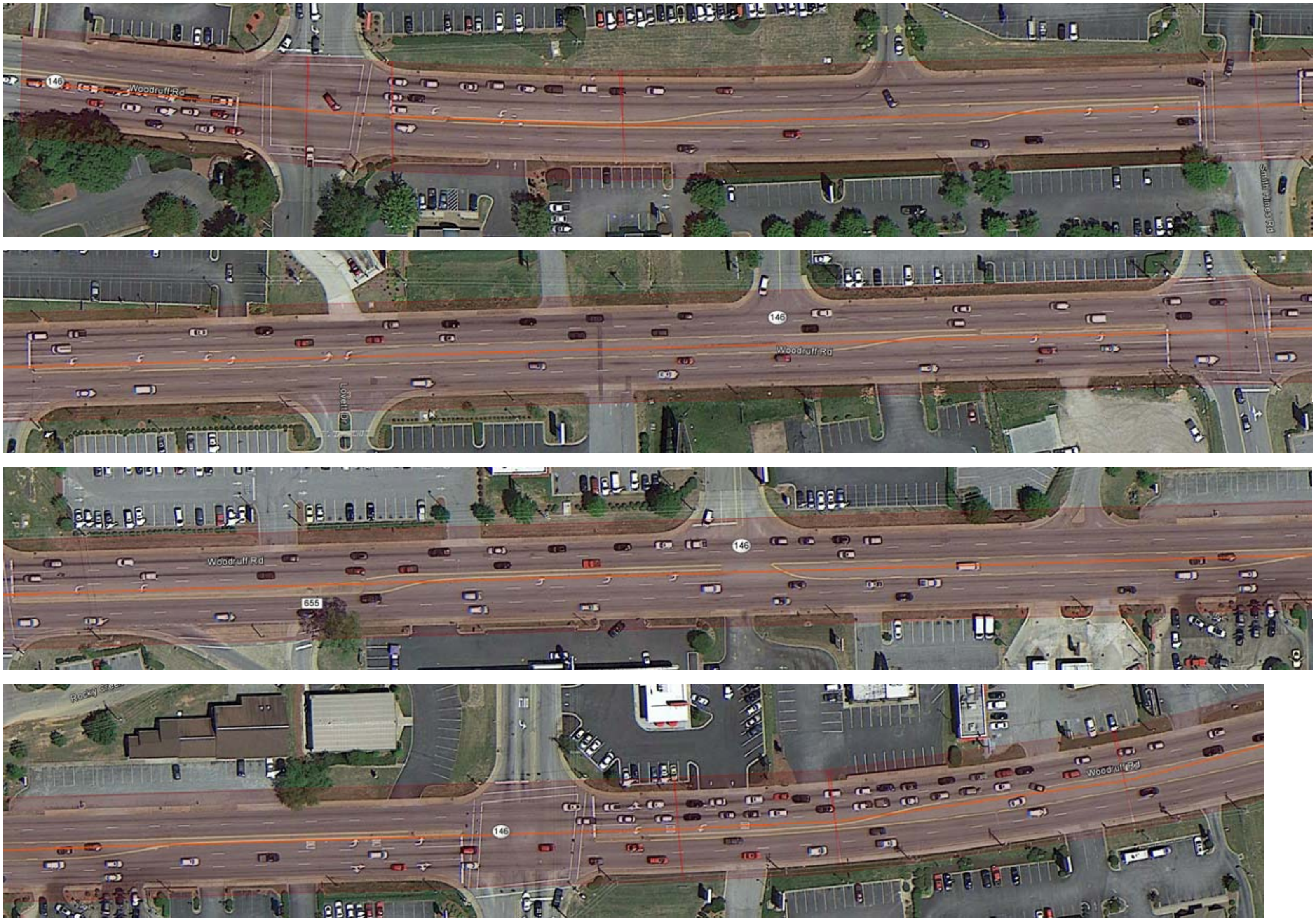


Figure 17a: *Woodruff Road w/ 96 ft. Buffer (continued on next page)*

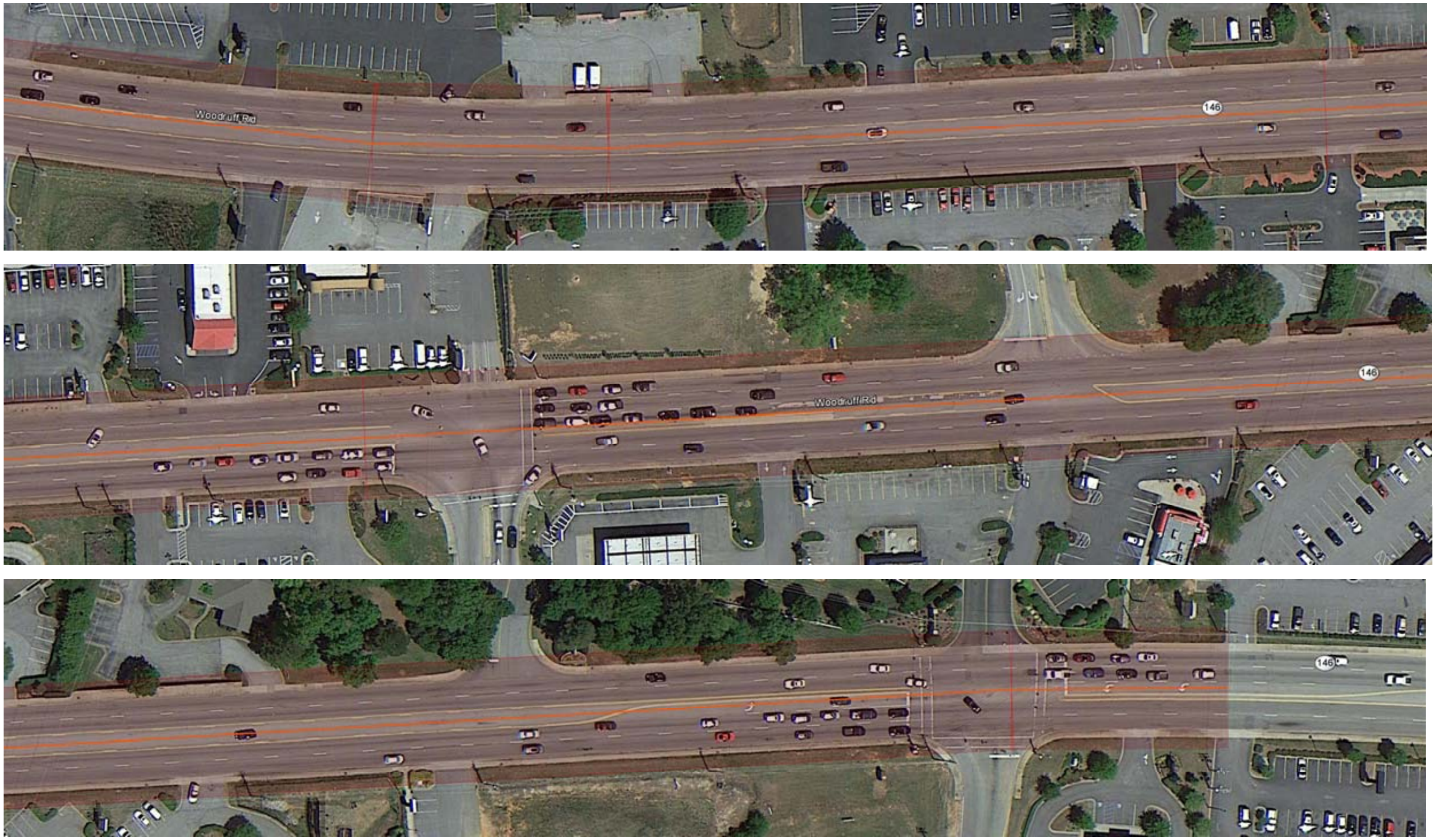


Figure 17b: *Woodruff Road w/ 96 ft. Buffer (continued from previous page)*

Figure 18: *Wade Hampton Blvd. w/ 96 ft. Buffer*

The four scenarios described above were devised to satisfy Objective 1. They have been termed 'traditional' access management tests because they have in some form been tested in similar experiences presented in previous literature. In addition to testing the operational impact of these strategies, two additional alternative scenarios were devised to test the effect of varying access restriction based on prevailing traffic conditions.

3.3. Demand Responsive Access Control Scenarios

There were an additional two alternative scenarios tested in this part of the thesis. While the peak hour was the only hour of interest in the 'traditional' access management tests, to adequately analyze the impacts of these strategies, a 5-hour simulation run time with a trapezoidal vehicle loading input pattern was used to test the impacts in both peak and off-peak hours. This was necessary because in these scenarios, the effect of changing volumes (and thus changing access restriction) was desired. So, running for only a peak hour loading would have no significant change in prevailing traffic conditions. Past SCDOT signal counts were used to determine ratios between peak and off-peak volumes. In other words, as with the previous four scenarios, the peak hour traffic was assigned according to the traffic counts performed in the field. The lowest, "off-peak" volumes, then, were determined by calculating the ratio of the lowest volume hour from historic counts and applying this ratio to the counts specifically performed for this study. So, for example, on the Woodruff Road corridor, which had a peak hour of 5:00-6:00PM, the model ran at off-peak volume from 3:00-4:00pm and increased traffic volume incrementally during the 4:00-5:00 PM interval until reaching peak volume. It then operated at peak hour volumes during the 5:00-6:00pm interval, incrementally decreased back down to off-peak volumes during the 6:00-7:00pm interval, and operated at off-peak hour volume again during the 7:00-8:00pm interval. This was likewise done with the Wade Hampton Road corridor simulation model. The calibrated base models for each corridor were run again during this simulation time and with these trapezoidal loadings, and the same MOEs (mainline travel times across the corridor and travel times from selected driveways to destinations) were analyzed. These same MOEs were analyzed for the two alternative scenarios, described below.

3.3.1. Non-Traversable Medians

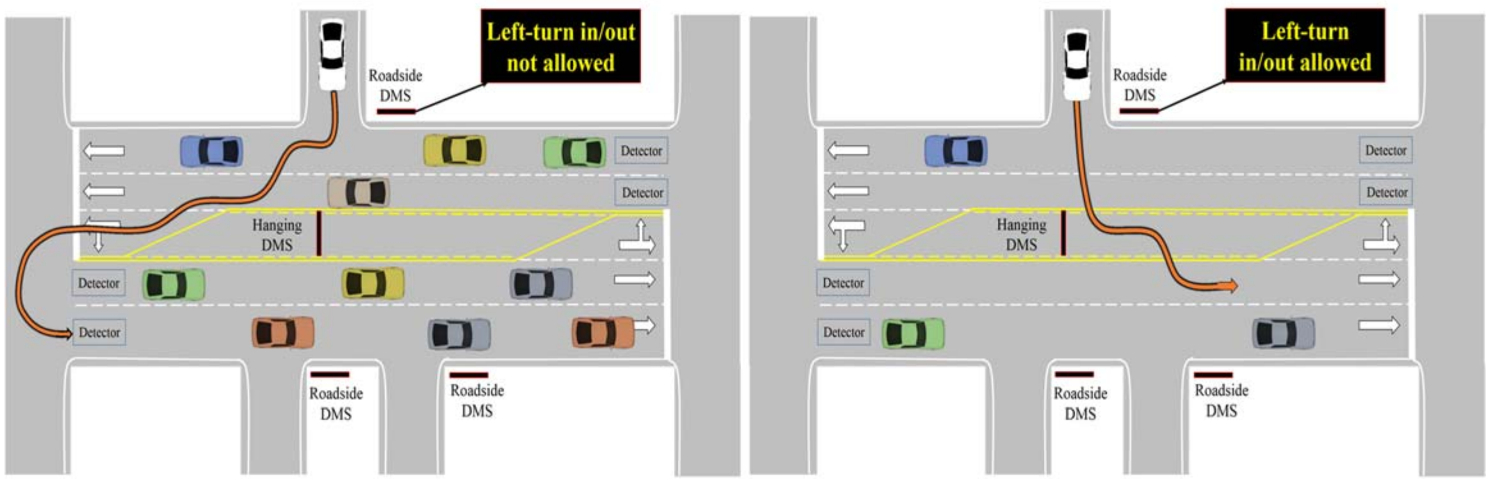
The first test scenario was similar to the non-traversable test from the previous section and included the replacement of the TWLTL with a raised median, restricting all driveways along the corridor to right-in-right-out access only, with U-turns at the nearest, upstream signalized intersection, provided there is sufficient space to accommodate weaving. An alternative option for handling U-turning movements is to allow U-turns at midblock median openings (either fully open or directional). However, the second alternative scenario (demand responsive access restriction) did not allow for this movement, so in order to allow for a closer comparison of alternative scenarios, U-turns at midblock median openings were not considered. The distribution of traffic to and from driveways was unaltered from the base model, however left-turning traffic was re-routed to the nearest and most practical upstream signalized intersection to perform a U-turn. A signal was considered a 'feasible' option if it has a weaving distance of at least 550 feet (for four-lanes) and 750 feet (on six-lanes), per the University of South Florida study of recommended minimum offset distances for RTUT movements *(16)*. Using these new routes, new signal counts were input into Synchro, and the signal splits, network cycle lengths, and coordination patterns and offsets were optimized and re-timed. In addition, in order to account for the additional U-turning traffic, left turn storage lanes were lengthened and protected left turn phases were added at signals where they previously did not exist.

3.3.2. Demand Responsive Access Point Control

The second test scenario for Objective 2 was to keep the TWLTL in place but allow direct left turn egress and left-in movements only when traffic flows on the approaching and opposing main-street movements are under volume thresholds during a defined interval (response time). In other words, a decision is made regarding permitted movements (i.e., whether to allow left turn in and left turn out) at every response time interval and the median functionality changes accordingly. This dynamic functionality occurs on a segment by segment basis – a segment being the stretch of roadway between two signals. Each segment has its own set of detectors which dictate how it operates, independent of the other segments along the corridor. Detectors are placed on each lane at the approach point of each direction of roadway in the segment, and set to calculate the number of vehicle front ends – used to determine the current volume. When the volume threshold is reached, left turning vehicles are permitted only RTUT movements for the duration of the response time, during which the flow rate from the detector is recalculated, and at the end of which the next decision regarding median functionality is made (Figure 19-a). If the volume threshold is not reached, left turning vehicles (both in and out of driveways) are permitted DLT movements for the duration of the response time (Figure 19-b).

Front end vehicle counts from detectors to estimate volume were chosen as the threshold indicators instead of density because it allowed direct left turn movements to occur both in low flow, off-peak intervals, as well as in peak-hour intervals of heavy congestion (if such heavy congestion was in fact encountered), where density is high but flow rate is low. This was done because in heavy-congestion/density conditions, many intersections along the corridor operate at low levels of service. Adding U-turning traffic to the signals exacerbates signal capacity issues, leading to lower travel times. In addition, direct left turns are justifiable during these congestion conditions from a safety point of view because the severity of conflicts is low due to very low travel speeds – and mainline drivers typically leave gaps for left-turners to exit and enter driveways.

Three different flow rate thresholds (in one direction) and three response time intervals were evaluated in this study: 750, 1500, and 3000 vph and 15, 30, and 45 seconds, respectively. 1500 vph was chosen as the middle threshold value because it was a threshold at which restricting driveways to right-in-right-out access was recommended in one of the reviewed studies *(21).* The other two thresholds were chosen to highlight the impact of a doubling or halving of the traffic flow threshold. Low response intervals of 15, 30, and 45 seconds were chosen to simulate a highly responsive system.



(a) *(b)*

Figure 19: *Demand responsive access control.*

This scenario was run using the optimized signal timings from the raised median scenario. The dynamic routing function of the Vehicle Actuated Programming (VAP) module in VISSIM was used to assign routes for left turning vehicles (either DLT or RTUT) at each driveway based on the appropriate segment's detectors' readings and to reset, recalculate, and reassign routes every response time interval throughout the entire simulation run. The VAP code is included in the Appendix at the end of this thesis.

Table 18 below shows all of the scenarios tested, the simulation run times, and the total number of simulation runs. Note that there are 7 different scenarios listed. However, the Demand-Responsive Access Control scenario has 9 sub-parts, for each of the threshold combinations of different volumes (750, 1500, 3000 vph) and response time (15, 30, 45) thresholds. Also note that the Base scenario is run for both the peak hour and the 5-hour simulation run times in order to be able to compare both sets of alternative scenarios. Therefore, there are a total of 16 separate scenarios (including 2 base scenarios for each run time, 4 traditional scenarios, and 10 ITS-based scenarios – 9 for the demand responsive). Each separate simulation scenario is run 10 times, for a total of 320 simulation runs.

Table 18: *Overview of Simulation Scenarios and Study Plan*

Scenarios	Corridor Segment		Simulation Run Time (including warm-up) [s]	No. of Runs (per corridor)	Total No. of Simulation Runs
	5-lane Minor Arterial (Woodruff)	7-lane Major Arterial (Wade Hampton)			
Base					
Existing Conditions	Full Access Driveways, TWLTL Median, Existing Signal Cycle length, splits, & Coordination		4200 & 18600	20	40
Traditional Scenarios					
Access Spacing	Consolidate driveways such that spacing equals SCDOT ARMS Standard (325')		4200	10	20
Corner Clearance	Consolidate driveways such that corner clearances equals SCDOT ARMS Standard (325')		4200	10	20
Access Restriction	Restrict all driveways within corner clearance (SCDOT ARMS 325') to right-in/right-out		4200	10	20
Non-Traversable Medians	Convert TWLTL to RM w/ RTUT at nearest feasible signals		4200	10	20
Demand Responsive Scenarios					
Non-Traversable Median	Convert TWLTL to RM w/ RTUT at nearest feasible signals		18600	10	20
Demand-Responsive	Restrict driveways during volume (750, 1500, 3000vph) and response time (15, 30, 45s) thresholds		18600	90	180
					320

CHAPTER FOUR

ANALYSIS AND RESULTS

The results from the base and alternative models for both traditional and demand-responsive access management scenarios are discussed below, first for the 5-lane Woodruff Road corridor, and then for the 7-lane Wade Hampton Blvd. corridor. On the figures, where a red 'X' indicates the value is not significantly different – at a 95% confidence level according to an independent sample t-test. A green arrow indicates that there was a significant difference.

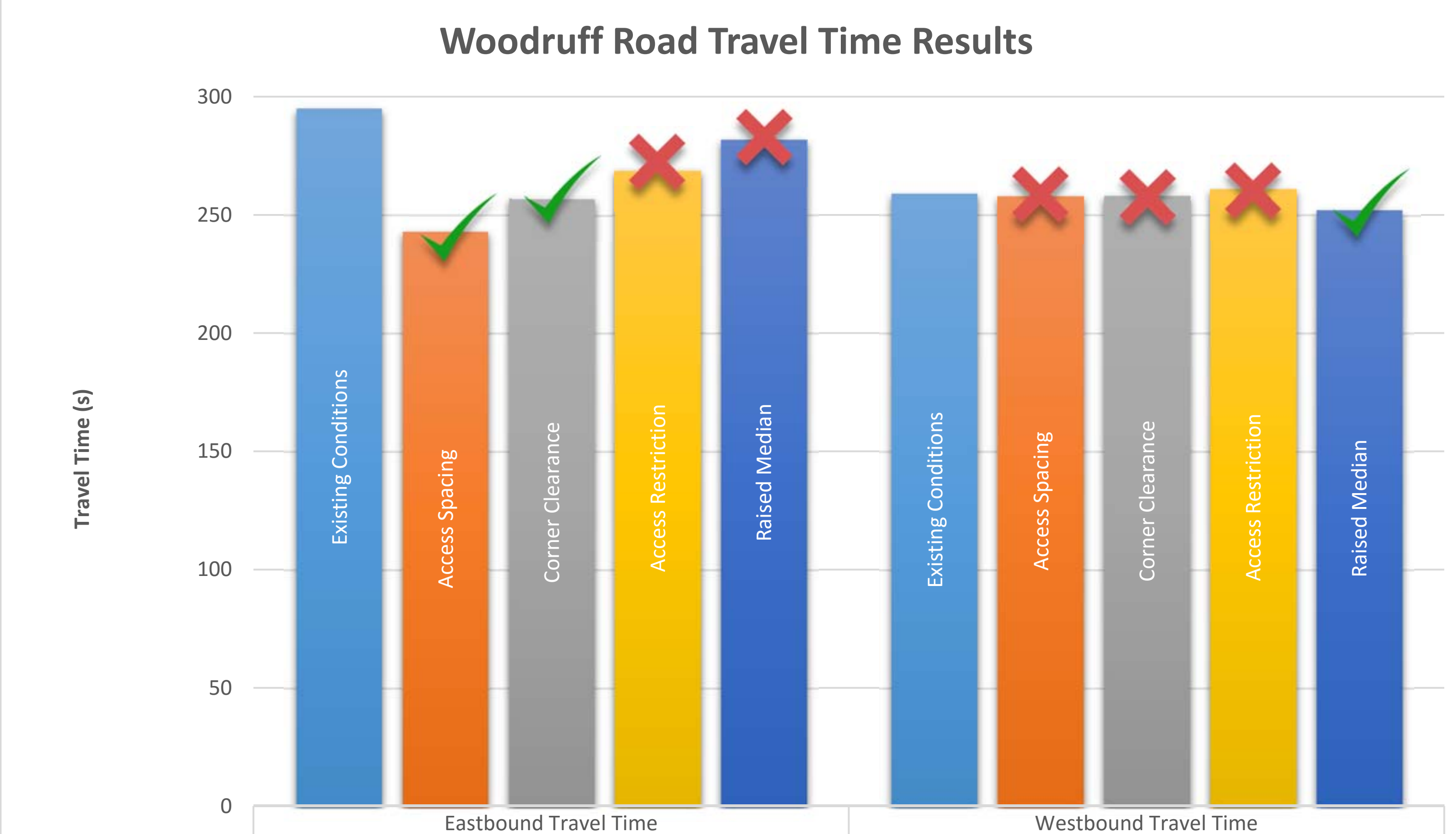
4.1. Woodruff Road

4.1.1. Traditional Access Management Scenarios

Recall that for each traditional access management scenario, the model was run for one peak-hour time period with peak-hour traffic volumes as collected in the field. Average Eastbound and Westbound travel times, as well as the total delay and stopped delay of egress and ingress traffic for each driveway along the corridor were collected for the entire run. The delay measures of effectiveness were collected as average delay per vehicle for the entire run. The results of the travel time and delay MOE's for Woodruff road are presented below in the following sections.

4.1.1.1. Mainline Corridor Segment Travel Times

Figure 20 on the following page displays the mainline corridor travel time results numerically as well as graphically for each of the 5 (1 base + 4-alternative) scenarios.



	Eastbound Travel Time		Westbound Travel Time	
Existing Conditions	295	p-value	259	p-value
Access Spacing	243	0.0008	258	0.4402
Corner Clearance	257	0.0104	258	0.3380
Access Restriction	269	0.0746	261	0.1036
Raised Median	282	0.3658	252	0.0284

***Figure 20**: Woodruff Road Traditional Access Management Strategy Mainline Travel Times*

Travel time results varied by direction of travel. In the *Eastbound* direction, all four alternative scenarios produced travel times lower than the existing conditions. The most favorable scenario from this perspective was that of Access Spacing, which decreased average peak hour travel times by 52 seconds, or 18%. The next-most favorable scenario was that of Corner Clearance which decreased Eastbound travel times by 38 seconds, or 13%. The Access Restriction scenario decreased travel times by 26 seconds, or 9%, and the Raised Median scenario decreased travel times by 13 seconds, or 4%. Recall that the only scenario for which signals were optimized and retimed was the Raised Median scenario. In the *Westbound* direction, there was little, to no change in travel times across the four alternative scenarios.

4.1.1.2. Driveway Traffic Total Delay and Stopped Delay

Figure 21 on the following page displays the total and stopped delay for ingress and egress driveway traffic numerically as well as graphically for each of the 5 (1 base + 4-alternative) scenarios. As with Eastbound travel times, the Access Spacing scenario had the most favorable results from both a total and stopped delay perspective and exhibited decreases of 12% in both types of delay. The Corner Clearance strategy also decreased both total delay and stopped delay by 8% and 12% respectively. The Access

Restriction and Raised Median scenarios, on the other hand, increased total delay (by 2 and 4% respectively), while the Access Restriction strategy increased stopped delay by 4% and the Raised Median strategy decreased stopped delay by 8%.

	Total Delay		Stopped Delay	
Existing Conditions	50	p-value	25	p-value
Access Spacing	44	0.0250	22	0.0240
Corner Clearance	46	0.1416	22	0.0302
Access Restriction	51	0.7316	26	0.4878
Raised Median	52	0.4888	23	0.1190

***Figure 21**: Woodruff Road Traditional Access Management Strategy Driveway Delay*

4.1.1.3. *Summary of Objective 1 Results*

Table 19 below shows the results of percent changes in each MOE for each strategy, compared to the existing conditions scenario for Woodruff Road, and Figure 22 on the following page shows these percent changes graphically such that the total change in MOEs can be compared for each strategy. From the values in the table and the graphical representation of the figure, each strategy improved or kept relatively constant the travel times in both direction., while only the access restriction and raised median strategies increased delay (total and/or stopped). When comparing the strategies however, including taking into consideration the sum of all improvements for MOE, it is clear that the access spacing strategy led to the greatest improvements in operational performance of this corridor.

Table 19: *% Changes from Existing Conditions for Each Alternative Strategy for Woodruff Rd.*

Woodruff Road (5-lane)*				
Strategy	Eastbound Travel Time (s)	Westbound Travel Time (s)	Total Delay (s)	Stopped Delay (s)
Access Spacing	-18%	0%	-12%	-12%
Corner Clearance	-13%	0%	-8%	-12%
Access Restriction	-9%	1%	2%	4%
Raised Median	-4%	-3%	4%	-8%

**Negative % indicates a decrease*

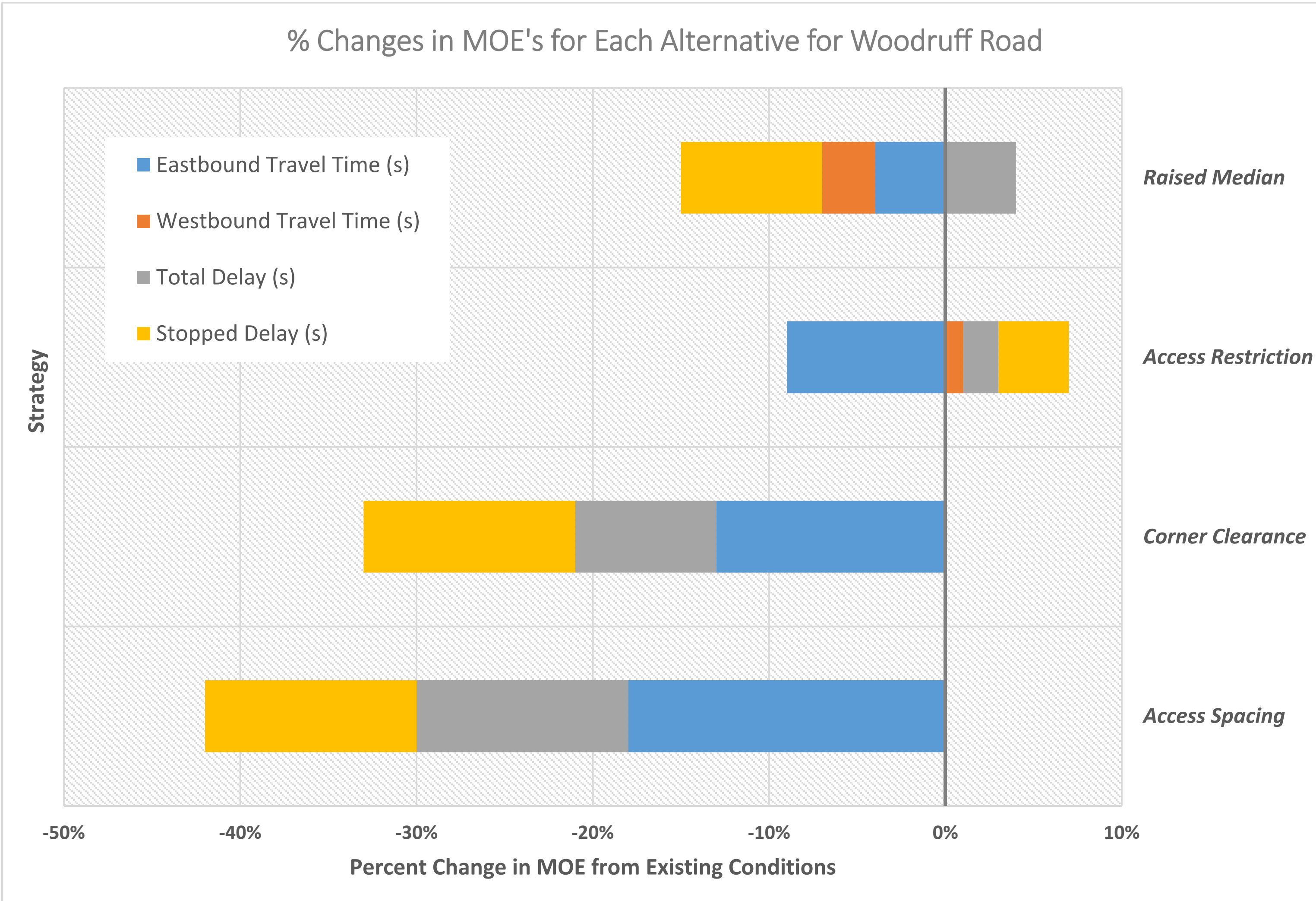


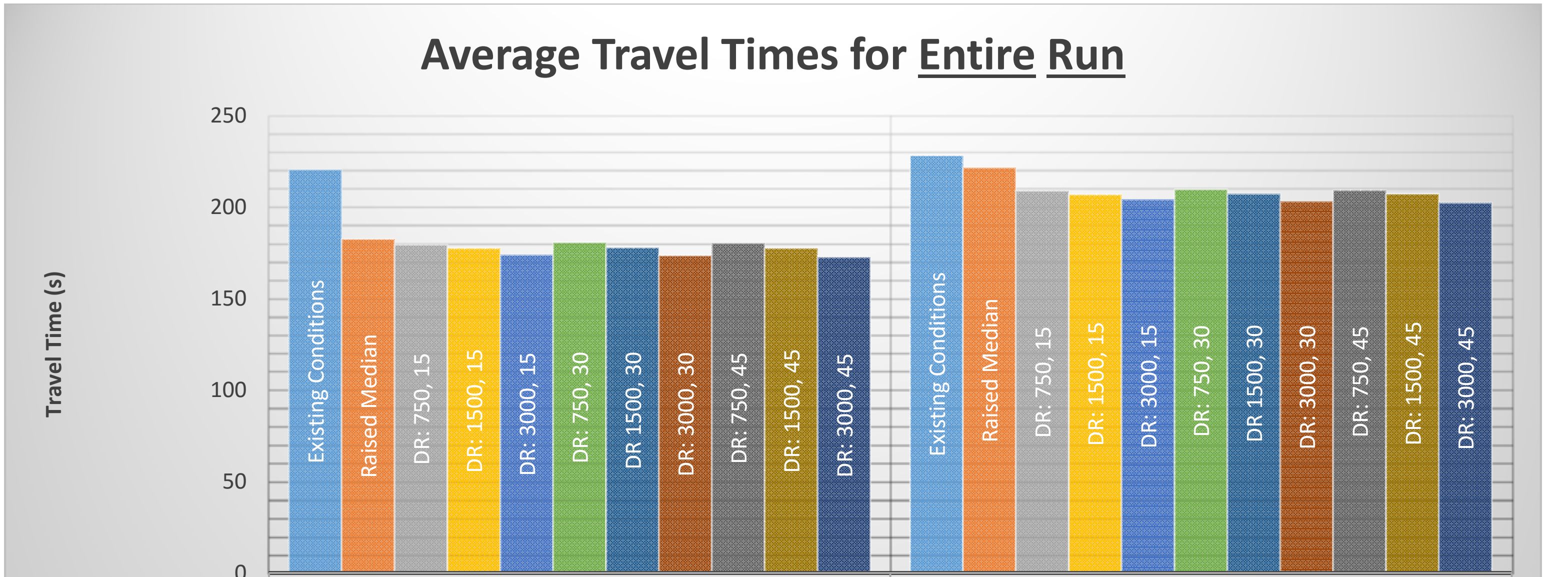
***Figure 22**: Changes in MOE's for Each Alternative Along Woodruff Road*

4.1.2. *Demand-Responsive Access Management Scenarios*

Recall that for each demand-responsive access management scenario, the model was run a 5-hour time period including one hour of off-peak traffic, followed by one hour of linearly increasing traffic loading, followed by one hour at peak traffic, followed by one hour of linearly decreasing traffic loading, concluding with one hour of off-peak traffic. The same MOE's (Eastbound/Westbound travel times and Total and Stopped Delay) were collected for these scenarios as well. However, since the model was run for different loading conditions, the results are presented differently than for the traditional access management scenarios. For each of the MOEs, the average values for the entire simulation time (5-hours) and the average values for the peak-hour will be presented in tabular form for existing condition scenario, the raised median scenario, and for each of the 9 demand-responsive scenarios. Additionally, in order to display the impact of each scenario over the course of the changing volume loadings, graphical representations of the MOE's over the course of the 5-hour simulation time will be presented comparing the existing condition scenario, the raised median scenario, and the most-favorable of the 9 demand-responsive scenarios.

4.1.2.1. *Mainline Corridor Travel Times*

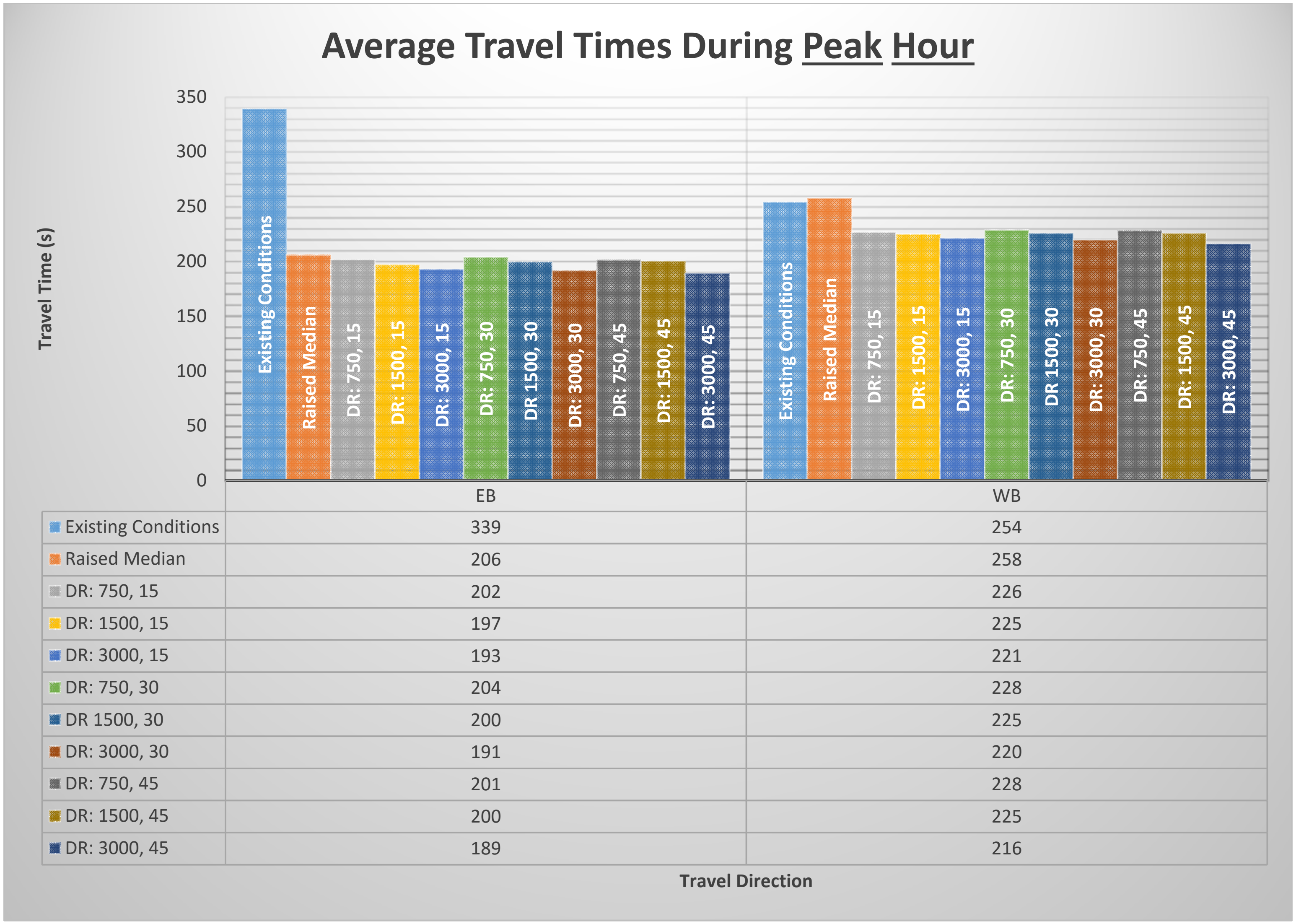
Figures 23 and 24 on the following pages show the Eastbound and Westbound travel times for the entire 5-hour run and during the peak hour for the 11 scenarios (existing, raised median, and 9 demand-responsive).



	EB	WB
Existing Conditions	220	228
Raised Median	182	221
DR: 750, 15	179	209
DR: 1500, 15	177	207
DR: 3000, 15	174	204
DR: 750, 30	181	210
DR 1500, 30	178	207
DR: 3000, 30	174	203
DR: 750, 45	180	209
DR: 1500, 45	178	207
DR: 3000, 45	173	202

Travel Direction

Figure 23*: Woodruff Road Demand-Responsive Travel Times for Entire 5-hour Run*



	EB	WB
Existing Conditions	339	254
Raised Median	206	258
DR: 750, 15	202	226
DR: 1500, 15	197	225
DR: 3000, 15	193	221
DR: 750, 30	204	228
DR 1500, 30	200	225
DR: 3000, 30	191	220
DR: 750, 45	201	228
DR: 1500, 45	200	225
DR: 3000, 45	189	216

***Figure 24**: Woodruff Road Demand-Responsive Travel Times for Peak Hour*

Several trends can be seen from the data. First, in the Eastbound direction of travel, for both the average travel times for the entire simulation run and during the peak hour, the alternative scenarios (raised median and demand-responsive) showed decreased travel times from the existing conditions. This decrease is especially pronounced for the travel times collected during the peak hour (Figure 24). In the Westbound direction of travel, the demand-responsive produced lower travel times, however the difference is less pronounced than in the Eastbound direction.

Another trend can be noted concerning the different demand-responsive scenarios. For each of the response times tested, the highest volume threshold produced the lowest travel times. Additionally, the volume thresholds for each response time were relatively similar. In other words, the changing volume thresholds for the demand-responsive scenarios had a greater effect on the travel times than on the time at which the access control was changed (response time). The 'best' demand-responsive scenario in terms of travel time, in both directions, for both the entire run and during the peak hour was the **DR: 3000, 45** alternative: access changed from fully open to right-in, right-out when the volume reached 3000 vph with the volume recalculated – and control decisions changed – every 45 seconds. Therefore, this demand-responsive scenario will be compared to the existing condition and raised median scenarios. Figures 25 and 26 on the following pages show the Eastbound and Westbound travel times for the entire run as a function of time.

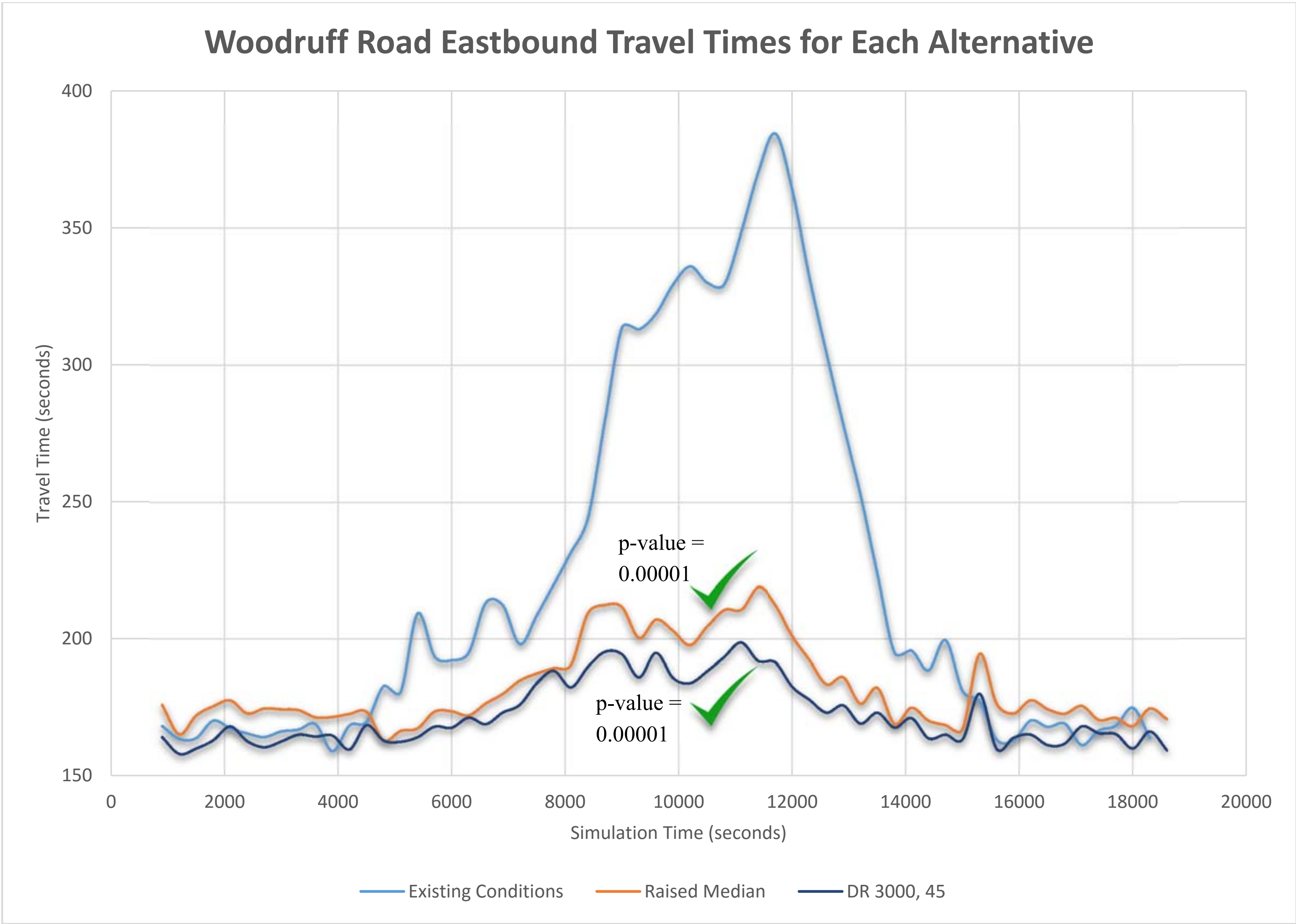


***Figure 25**: Woodruff Road Demand-Responsive Eastbound Travel Times*

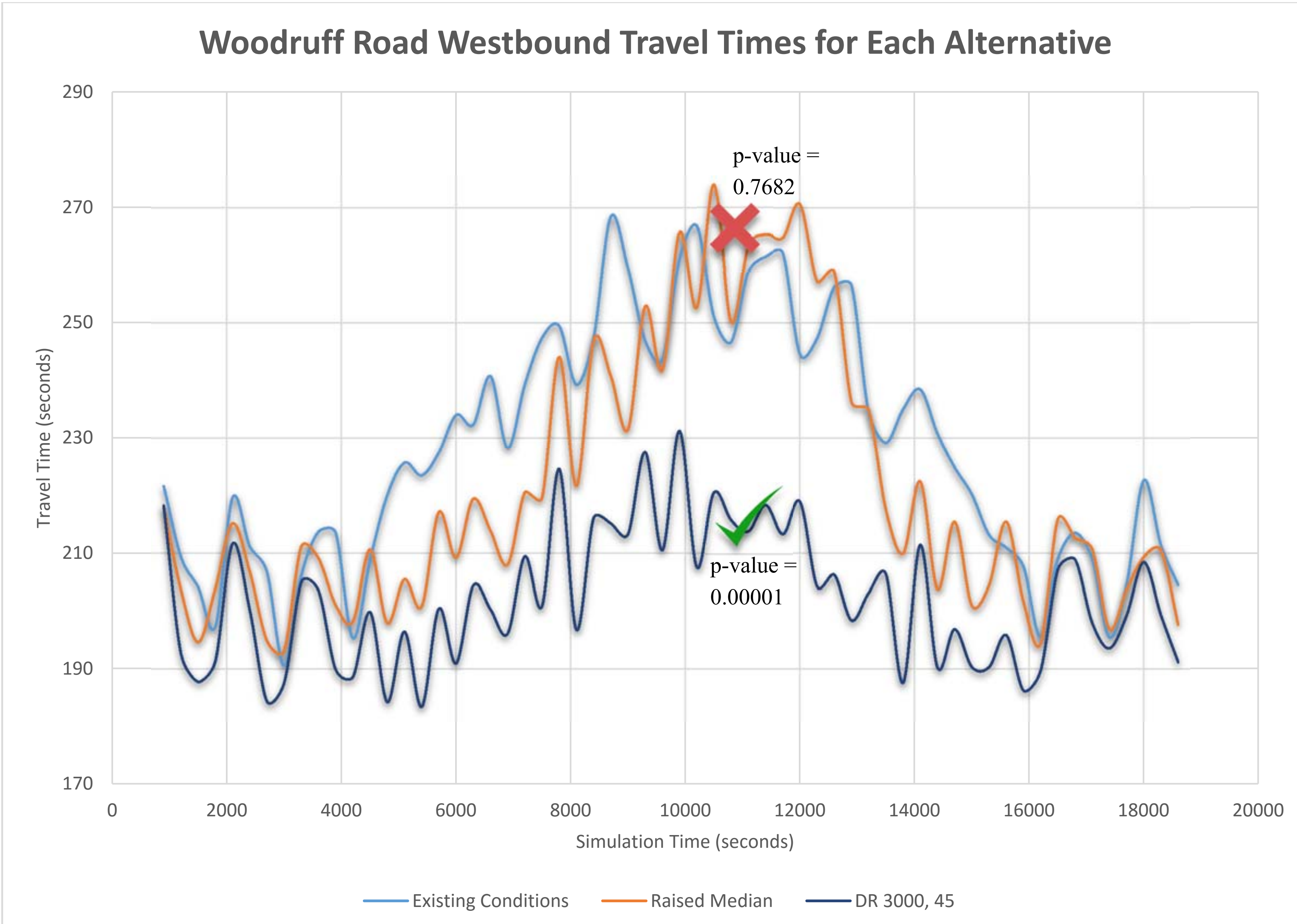


***Figure 26**: Woodruff Road Demand-Responsive Westbound Travel Times*

Figures 25 and 26 above reveal a difference in resulting travel times by direction of travel. In the Eastbound direction, there are stark differences between the Existing Conditions and the Raised Median and Demand-Responsive scenarios. It is the opinion of this researcher that this very noticeable difference may be largely attributable to the signal optimization that was performed and used for both alternative scenarios, simply because the change in travel times is so large – larger than any other change among any other set of scenarios compared, including the peak hour tests conducted for the 'traditional' access management scenarios – which themselves included one Raised Median scenario. Nonetheless, the combination of signal optimization and access control (both permanent and demand-responsive) led to major decreases in Eastbound travel times for the 5-lane Woodruff Road corridor. In the Westbound direction of travel, the Raised Median scenario produced travel time patterns very similar to that of the Existing Conditions. However, the demand-responsive scenario showed a noticeable decrease in travel times during the middle (peak) portion of the simulation.

In order to compare the differences in travel time for each scenario in both directions more holistically, Figure 27 was created by summing the travel times in both East and Westbound directions. Considering both directions of travel, it appears that the demand responsive scenario did make the mainline more efficient, in terms of travel time, over the course of the entire 5-hour run (for both off-peak and peak traffic conditions).

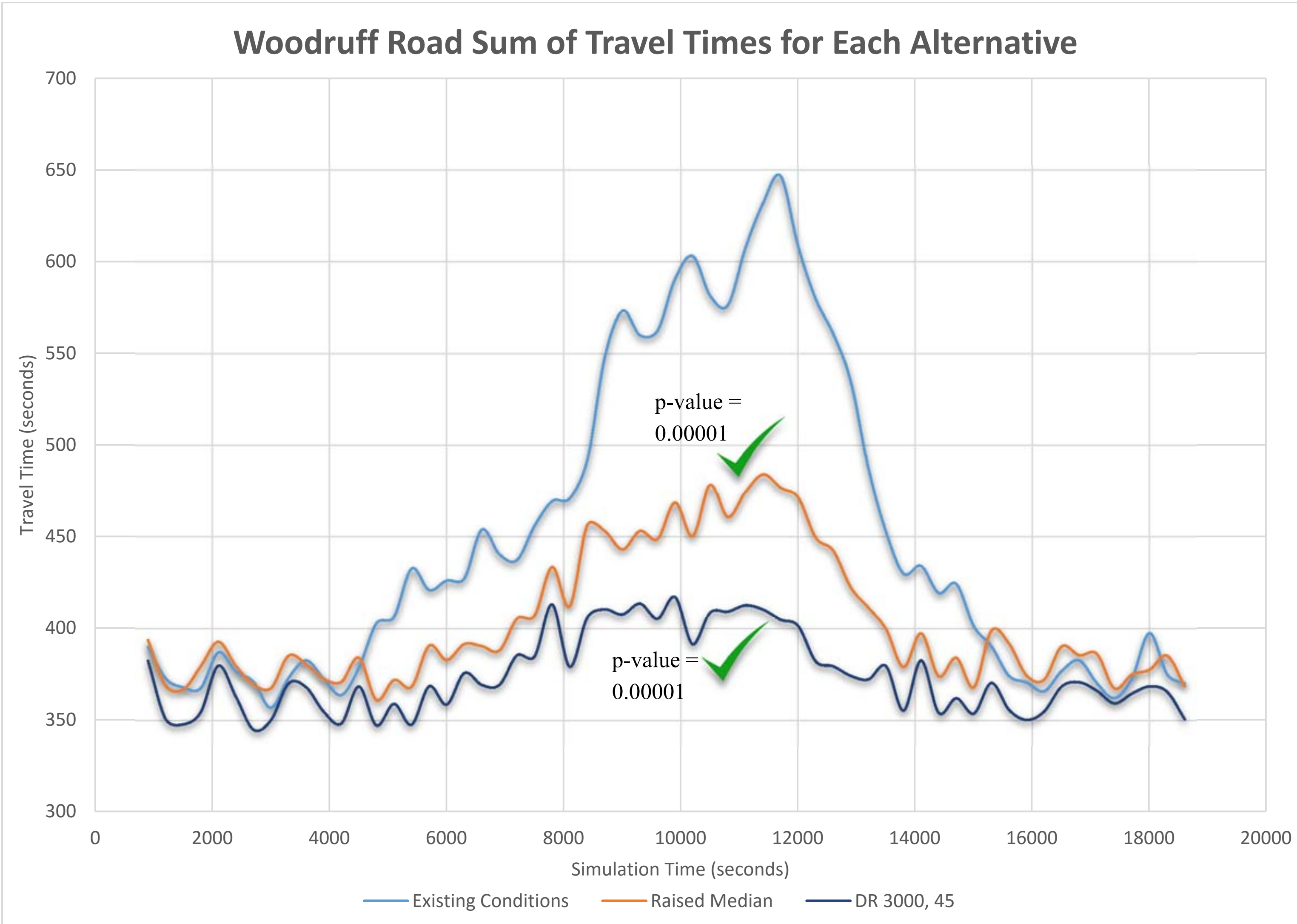


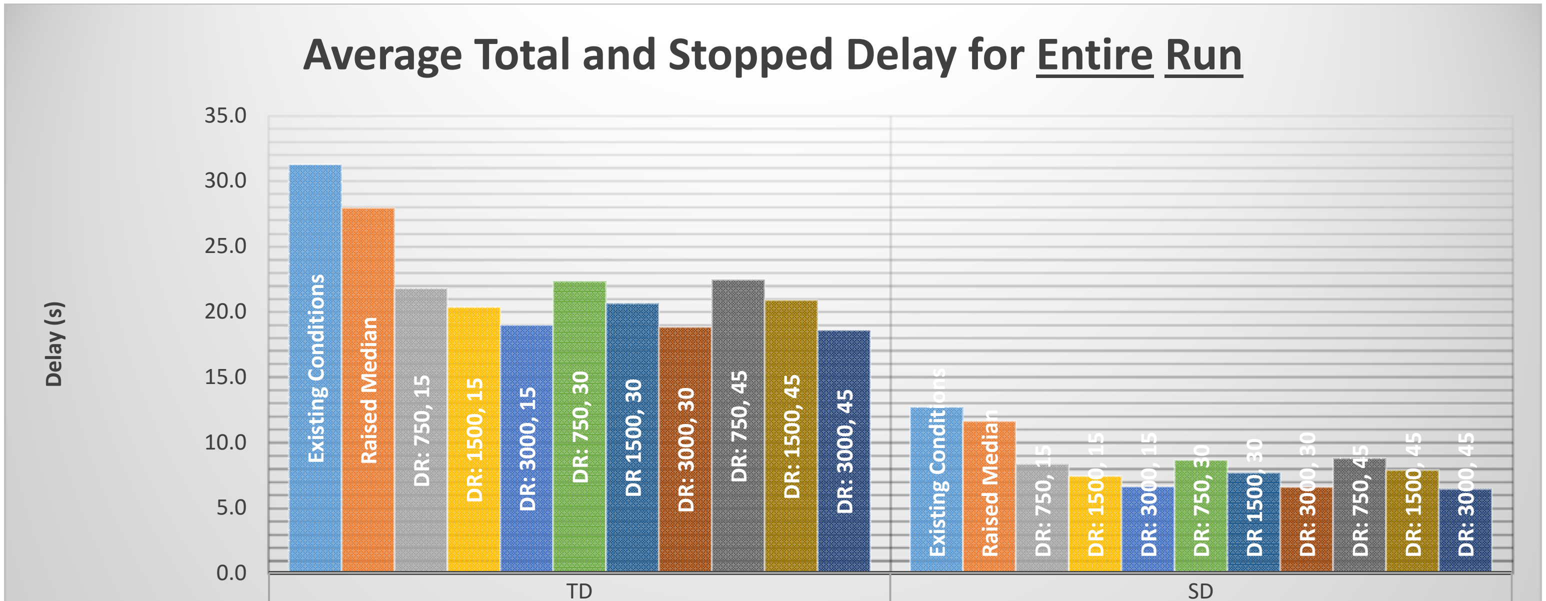
***Figure 27**: Woodruff Road Demand-Responsive Sum of Travel Times*

4.1.2.2. Driveway Traffic Total Delay and Stopped Delay

Figures 28 and 29 on the following pages show the Total Delay and Stopped Delay for the entire 5-hour run and during the peak hour for the 11 scenarios (existing, raised median, and 9 demand-responsive). For both the simulation as a whole, and during the peak hour, the Raised Median and Demand-Responsive Scenarios lowered the total and stopped delay when compared to the existing conditions.

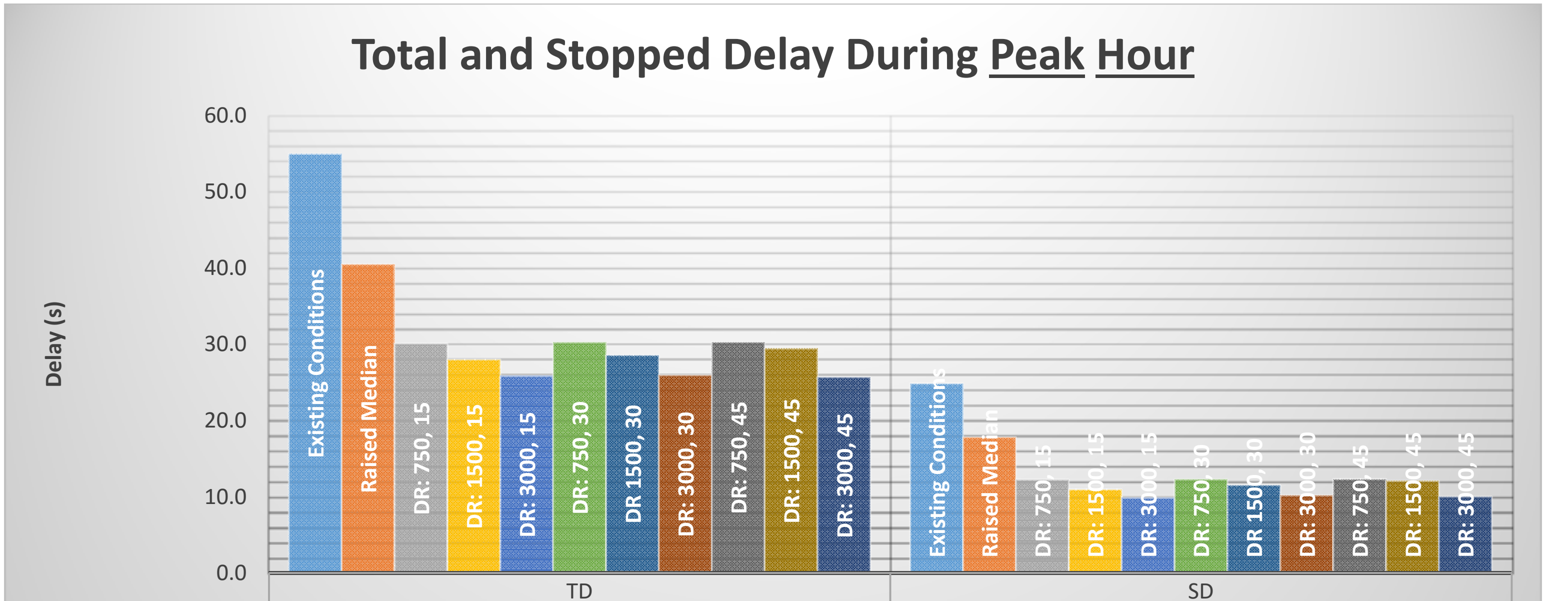
Comparing the 9 different Demand-Responsive scenarios, a similar trend can be observed as that which was seen in the travel time results. For each response time tested, the highest volume threshold produced the lowest stopped and total delay. In other words, changing volume thresholds for the demand-responsive scenarios had a greater effect on the travel times than on the time at which the access control was changed (response time). The 'best' demand-responsive scenario in terms of total and stopped delay was the **DR: 3000, 45** alternative: access changed from fully open to right-in, right-out when the volume reached 3000 vph with the volume recalculated – and control decisions changed – every 45 seconds. Therefore, this demand-responsive scenario will be compared to the existing condition and raised median scenarios.

Figures 29 and 30 which follow Figures 28 and 29 show the total and stopped delay for Woodruff Road as a function of simulation time.



	TD	SD
Existing Conditions	31.3	12.7
Raised Median	27.9	11.6
DR: 750, 15	21.8	8.3
DR: 1500, 15	20.3	7.4
DR: 3000, 15	19.0	6.6
DR: 750, 30	22.3	8.7
DR 1500, 30	20.6	7.7
DR: 3000, 30	18.8	6.6
DR: 750, 45	22.4	8.8
DR: 1500, 45	20.9	7.9
DR: 3000, 45	18.6	6.4

Figure 28: *Woodruff Road Demand-Responsive Total and Stopped Delay for Entire Simulation Run*



	TD	SD
Existing Conditions	55.0	24.9
Raised Median	40.5	17.8
DR: 750, 15	30.1	12.3
DR: 1500, 15	28.0	11.0
DR: 3000, 15	25.8	9.9
DR: 750, 30	30.3	12.3
DR 1500, 30	28.6	11.6
DR: 3000, 30	26.0	10.2
DR: 750, 45	30.3	12.3
DR: 1500, 45	29.5	12.1
DR: 3000, 45	25.7	10.0

Figure 29*: Woodruff Road Demand-Responsive Total and Stopped Delay for Peak Hour*

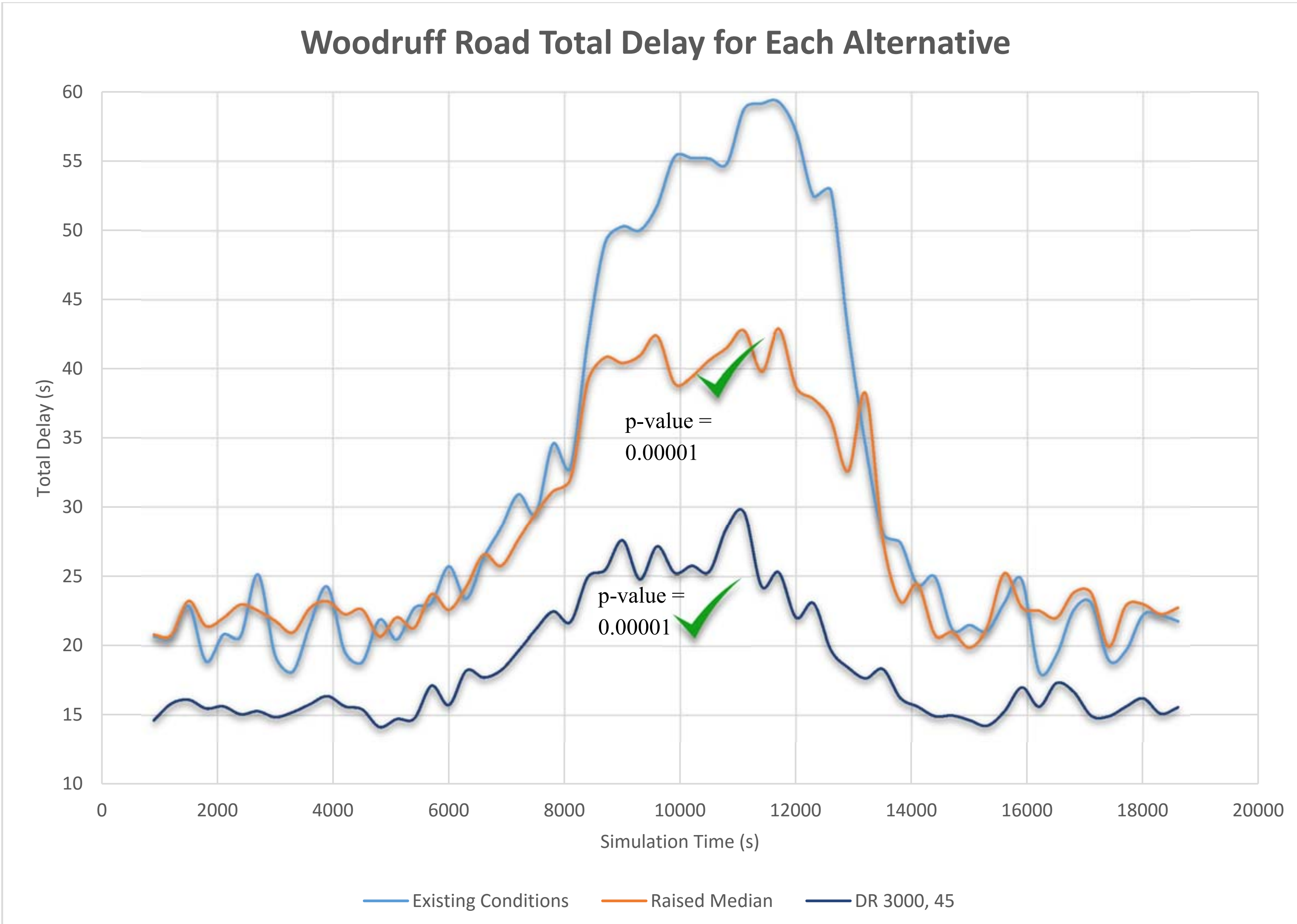


***Figure 30**: Woodruff Road Demand-Responsive Total Delay as a Function of Time*

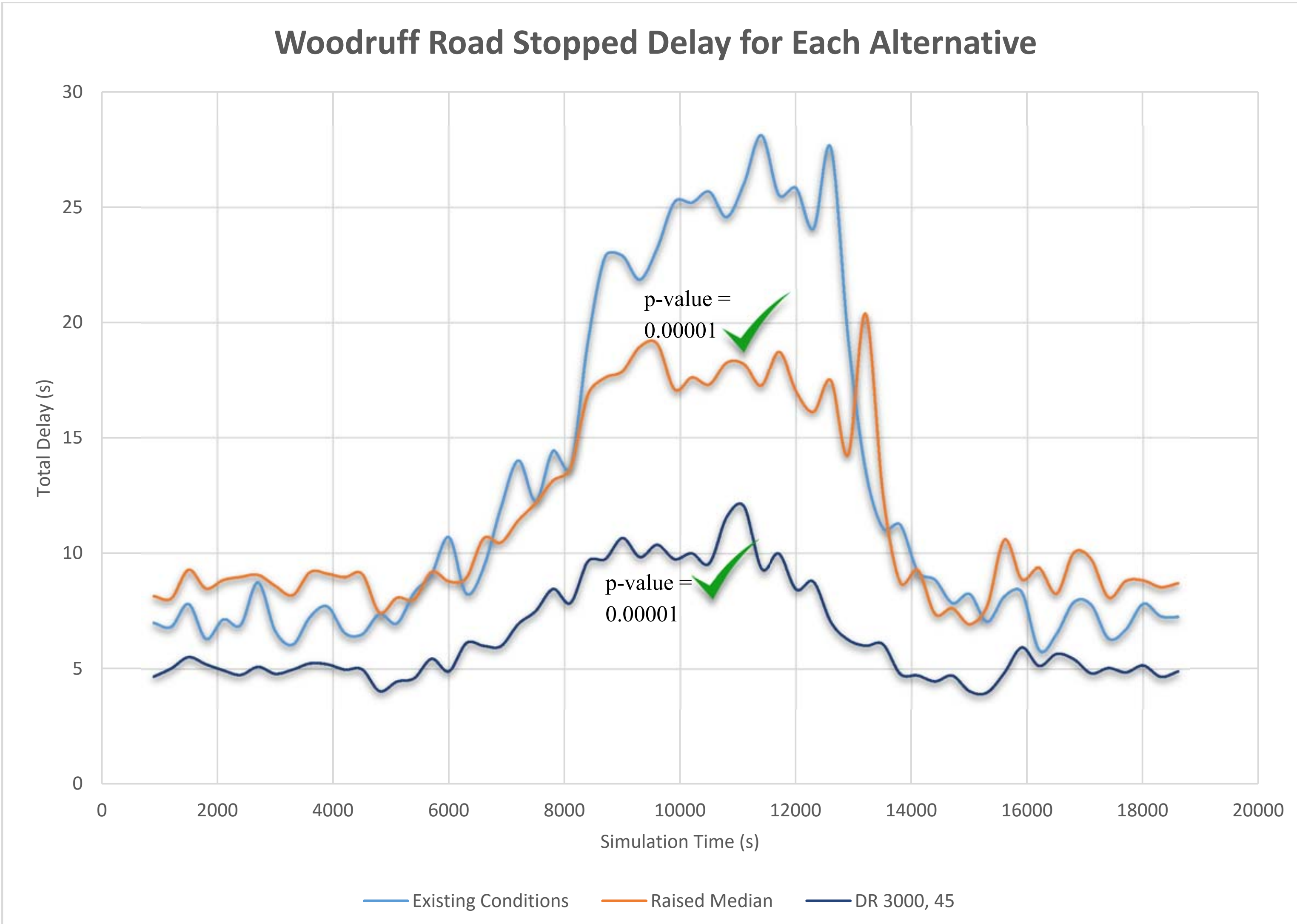


Figure 31*: Woodruff Road Demand-Responsive Stopped Delay as a Function of Time*

Figures 30 and 31 above (showing total and stopped delay as a function of simulation time respectively) show very similar patterns as the simulation proceeds from start to finish. Both figures show that the Raised Median scenario had similar total and stopped delay to the Existing Conditions during off-peak conditions but lower delays during the peak traffic conditions. The Demand-Responsive scenario, on the other hand, showed lower total and stopped delays during the entirety of the run, both in off-peak and peak conditions.

It appears, then, according to the results of the analysis for the Woodruff Road corridor segment, that the implementation of a raised median leads to lower delays per vehicle (for driveway ingress and egress traffic) during peak hour traffic conditions. Additionally, varying access between fully open and right-in/right-out can further reduce delays for driveway traffic in both off-peak and peak conditions.

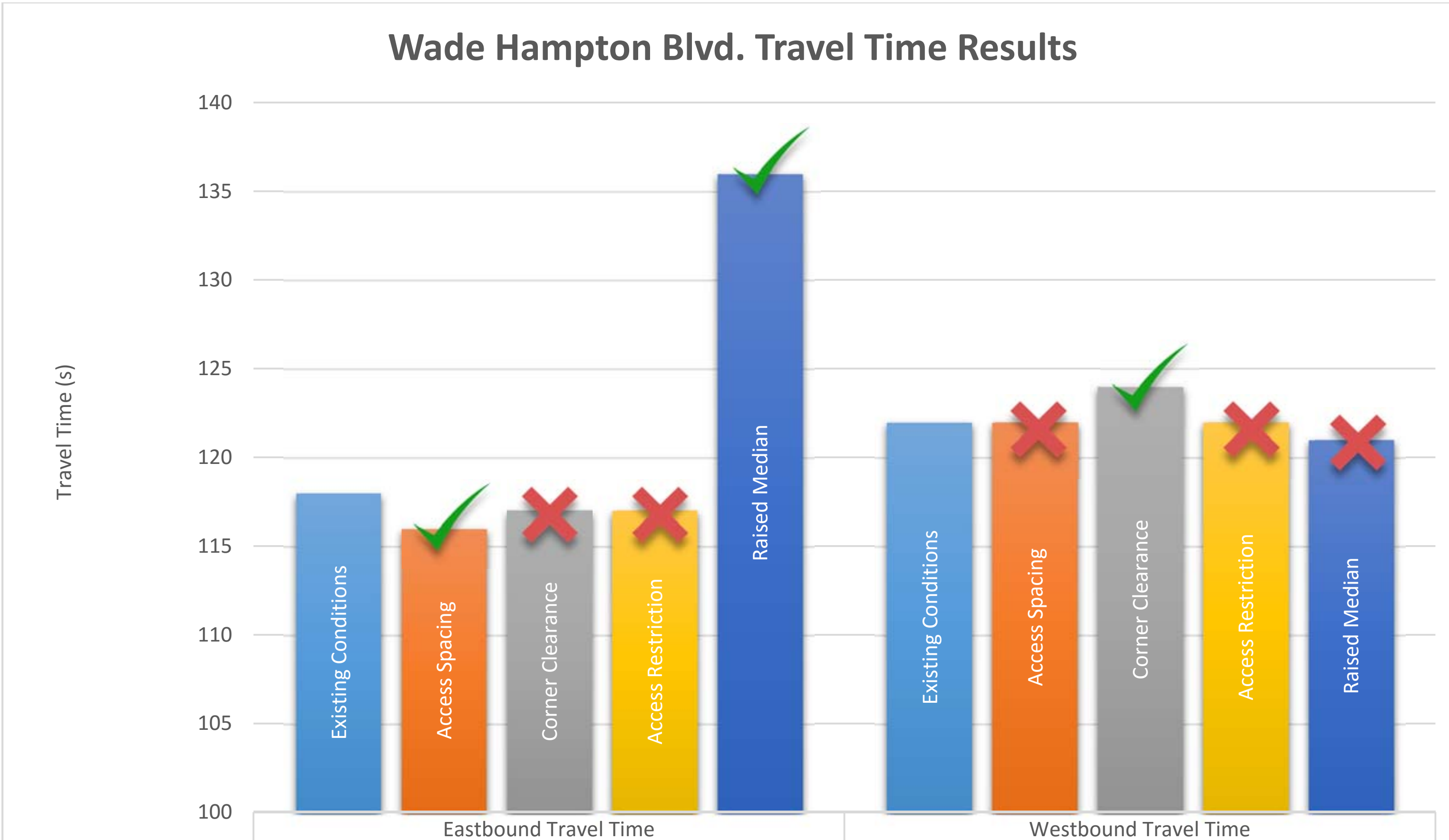
4.2. Wade Hampton Blvd.

4.2.1. Traditional Access Management Scenarios

As with the Woodruff Road Corridor, recall that for each traditional access management scenario, the model was run for one peak-hour time period with peak-hour traffic volumes as collected in the field. Average Eastbound and Westbound travel times, as well as the total delay and stopped delay of egress and ingress traffic for each driveway along the corridor were collected for the entire run. The delay measures of effectiveness were collected as average delay per vehicle for the entire run. The results of the travel time and delay MOE's for Wade Hampton Blvd. are presented below in the following sections.

4.2.1.1. Mainline Corridor Segment Travel Times

Figure 32 on the following page displays the mainline corridor travel time results numerically as well as graphically for each of the 5 (1 base + 4-alternative) scenarios.



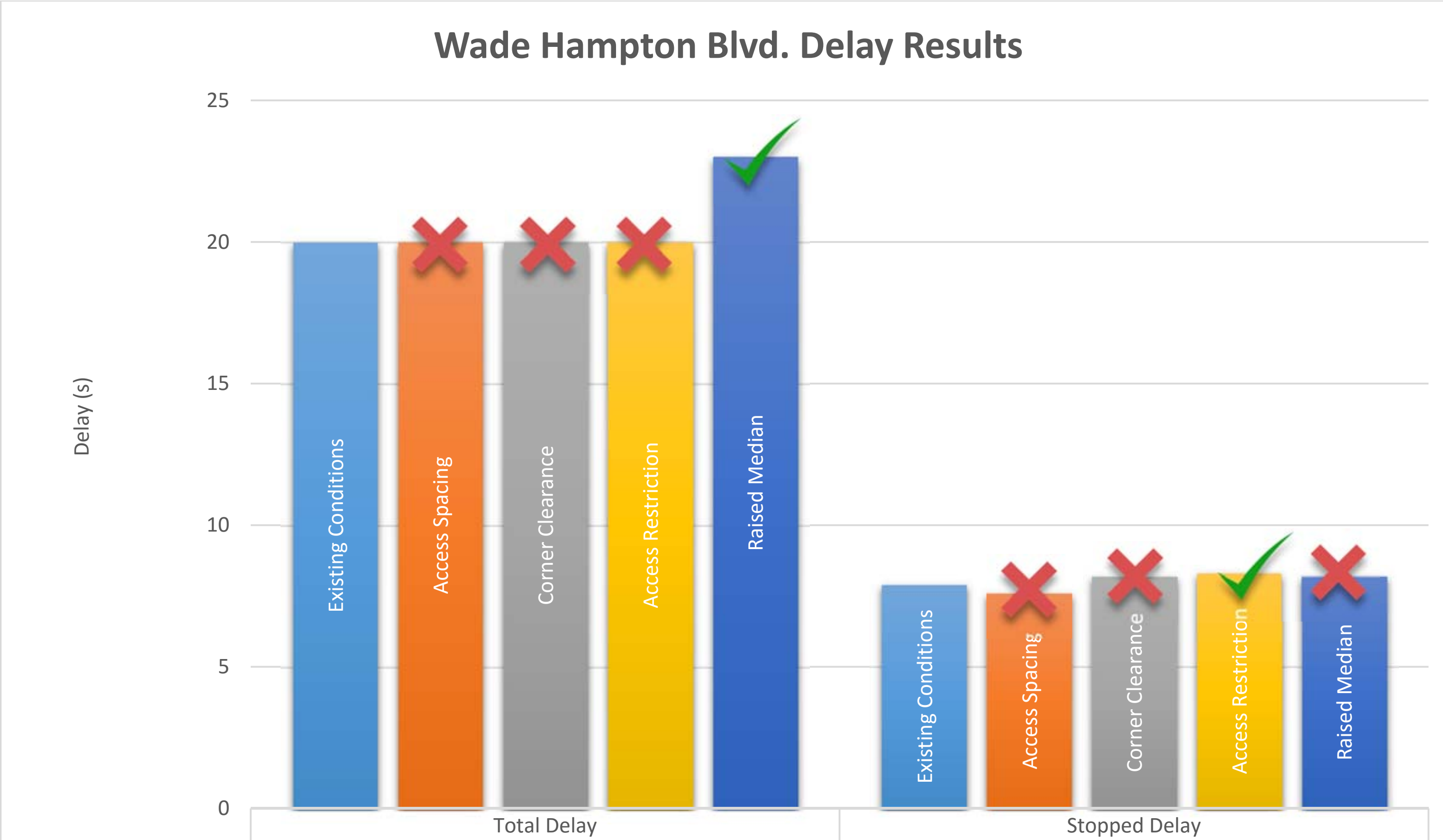
	Eastbound Travel Time		Westbound Travel Time	
Existing Conditions	118	p-value	122	p-value =
Access Spacing	116	0.0068	122	0.9999
Corner Clearance	117	0.1264	124	0.0062
Access Restriction	117	0.1360	122	0.9999
Raised Median	136	0.0001	121	0.1056

***Figure 32**: Wade Hampton Blvd. Traditional Access Management Strategy Travel Time*

Travel time results varied by direction of travel. In the *Eastbound* direction, three of the alternative scenarios (Access Spacing, Corner Clearance, and Access Restriction) performed similarly to the existing conditions. However, the Raised Median scenario increased travel times by roughly 15%. In the *Westbound* direction, there was little, to no change in travel times across the four alternative scenarios.

4.2.1.2. Driveway Traffic Total Delay and Stopped Delay

Figure 33 on the following page displays the total and stopped delay for ingress and egress driveway traffic numerically as well as graphically for each of the 5 (1 base + 4-alternative) scenarios. As with Eastbound travel times, the only scenario that showed a difference in delay was the Raised Median scenario, which increased total delay by roughly 15%. However, there was negligible difference in stopped delay for each of the scenarios tested.



	Total Delay		Stopped Delay	
Existing Conditions	20	p-value	7.9	p-value =
Access Spacing	20	0.9999	7.6	0.0792
Corner Clearance	20	0.9999	8.2	0.1255
Access Restriction	20	0.9999	8.3	0.0450
Raised Median	23	0.0001	8.2	0.0712

***Figure 33**: Wade Hampton Blvd. Traditional Access Management Strategy Driveway Delay*

4.2.1.3. Summary of Objective 1 Results

Table 20 below shows the results of percent changes in each MOE for each strategy, compared to the existing conditions scenario for Wade Hampton Blvd., and Figure 34 on the following page shows these percent changes graphically such that the total change in MOEs can be compared for each strategy. From the values in the table and the graphical representation of the figure, it is readily noticeable that the results are different than those for Woodruff Road, indicating that operational impacts of traditional access management strategies are site-specific. Of the four strategies, implementation of the raised median had the most negative operational impacts. On the other hand, though the results were not overwhelmingly noticeable, the access spacing strategy had the most positive operational impacts.

Table 20: *Percent Changes from Existing Conditions for Each Alternative Strategy*

Wade Hampton Blvd (7-lane)*				
Strategy	Eastbound Travel Time (s)	Westbound Travel Time (s)	Total Delay (s)	Stopped Delay (s)
Access Spacing	-2%	0%	0%	-4%
Corner Clearance	-1%	2%	0%	4%
Access Restriction	-1%	0%	0%	5%
Raised Median	15%	-1%	15%	4%

**Negative % indicates a decrease*

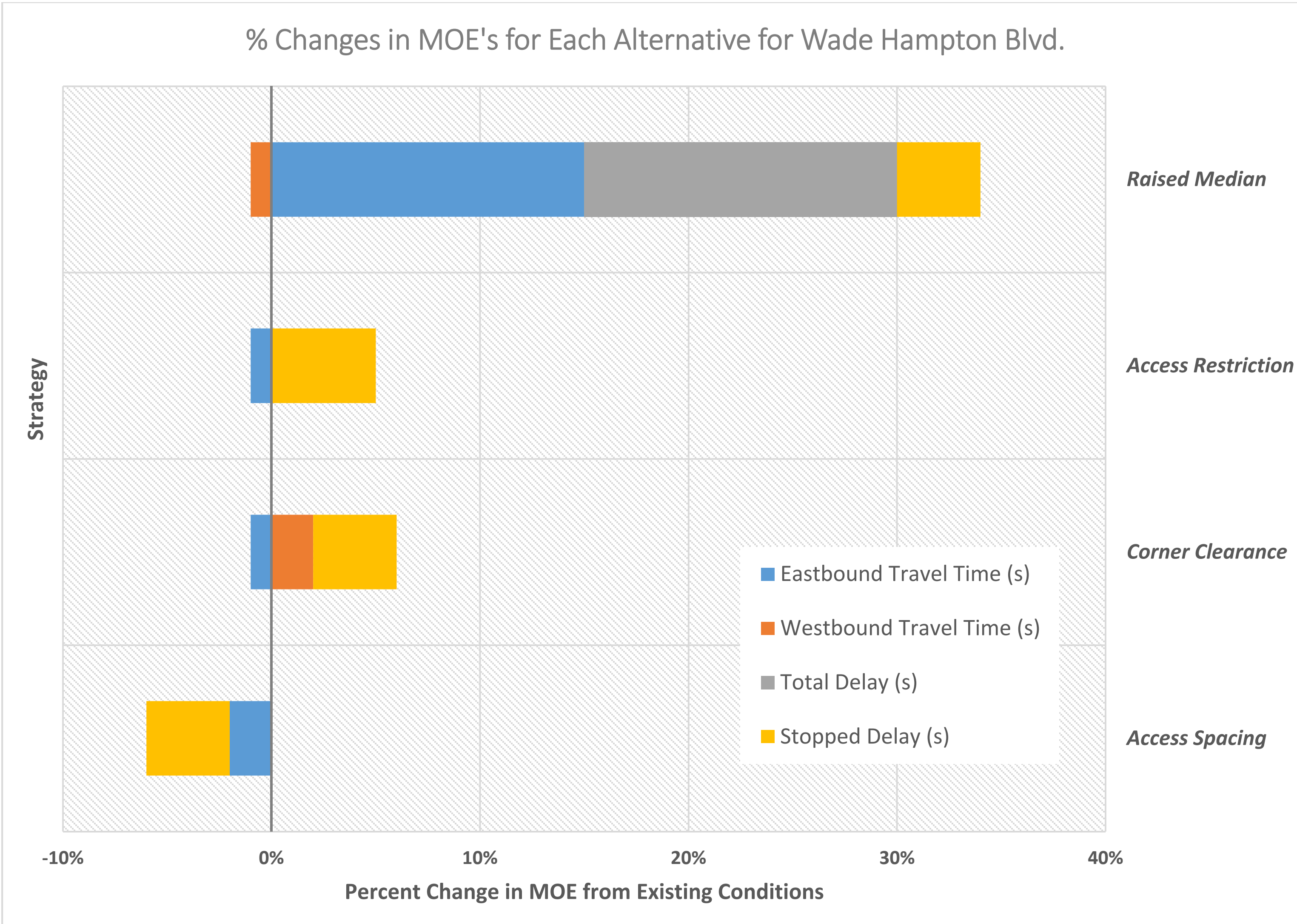


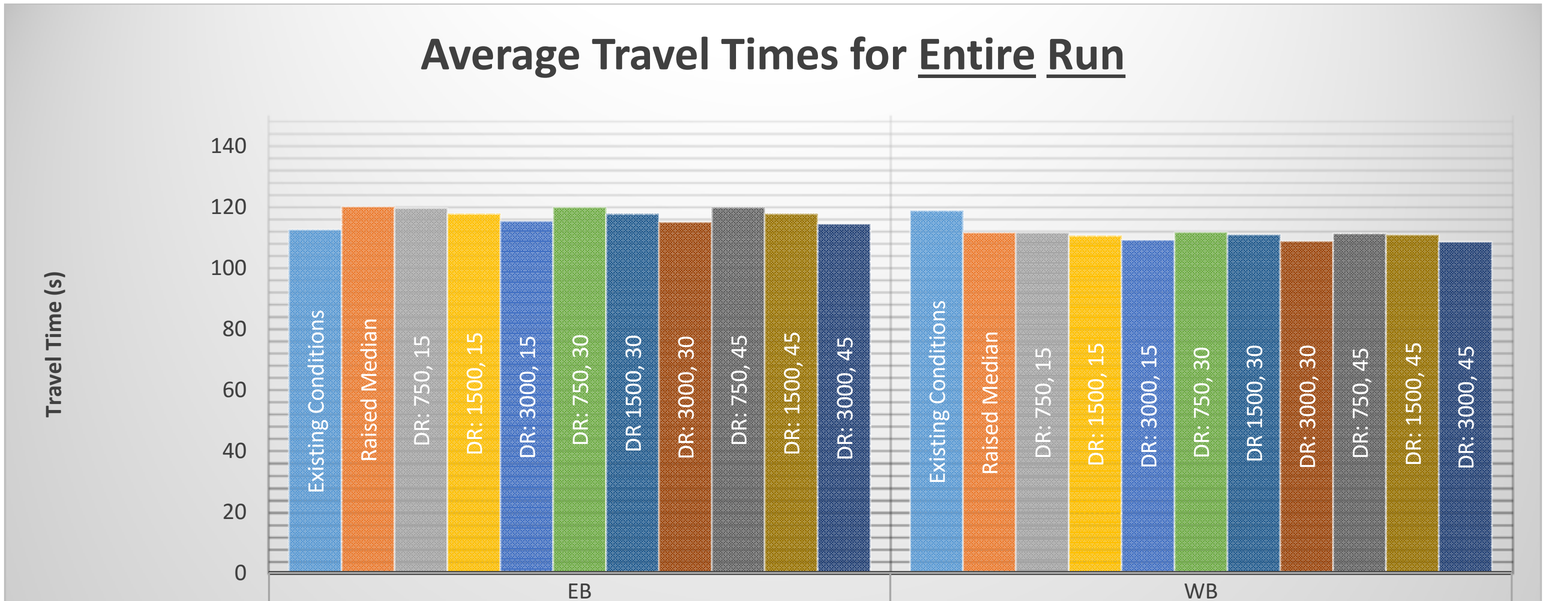
***Figure 34**: Changes in MOE's for Each Alternative Along Wade Hampton Blvd.*

4.2.2. Demand-Responsive Access Management Scenarios

Again, as with the Woodruff Road corridor, for each demand-responsive access management scenario, the model was run a 5-hour time period including one hour of off-peak traffic, followed by one hour of linearly increasing traffic loading, followed by one hour at peak traffic, followed by one hour of linearly decreasing traffic loading, concluding with one hour of off-peak traffic. The same MOE's (Eastbound/Westbound travel times and Total and Stopped Delay) were collected for these scenarios as well. However, since the model was run for different loading conditions, the results are presented differently than for the traditional access management scenarios. For each of the MOEs, the average values for the entire simulation time and the average values for the peak-hour will be presented in tabular form for existing condition scenario, the raised median scenario, and for each of the 9 demand-responsive scenarios. Additionally, in order to display the impact of each scenario over the course of the changing volume loadings, graphical representations of the MOE's over the course of the 5-hour simulation time will be presented comparing the existing condition scenario, the raised median scenario, and the most-favorable of the 9 demand-responsive scenarios.

4.2.2.1. Mainline Corridor Travel Times

Figures 35 and 36 on the following pages show the Eastbound and Westbound travel times for the entire 5-hour run and during the peak hour for the 11 scenarios (existing, raised median, and 9 demand-responsive).



	EB	WB
Existing Conditions	113	119
Raised Median	120	112
DR: 750, 15	120	112
DR: 1500, 15	118	111
DR: 3000, 15	115	109
DR: 750, 30	120	112
DR 1500, 30	118	111
DR: 3000, 30	115	109
DR: 750, 45	120	111
DR: 1500, 45	118	111
DR: 3000, 45	114	109

***Figure 35**: Wade Hampton Blvd. Demand-Responsive Travel Times for Entire 5-hour Run*

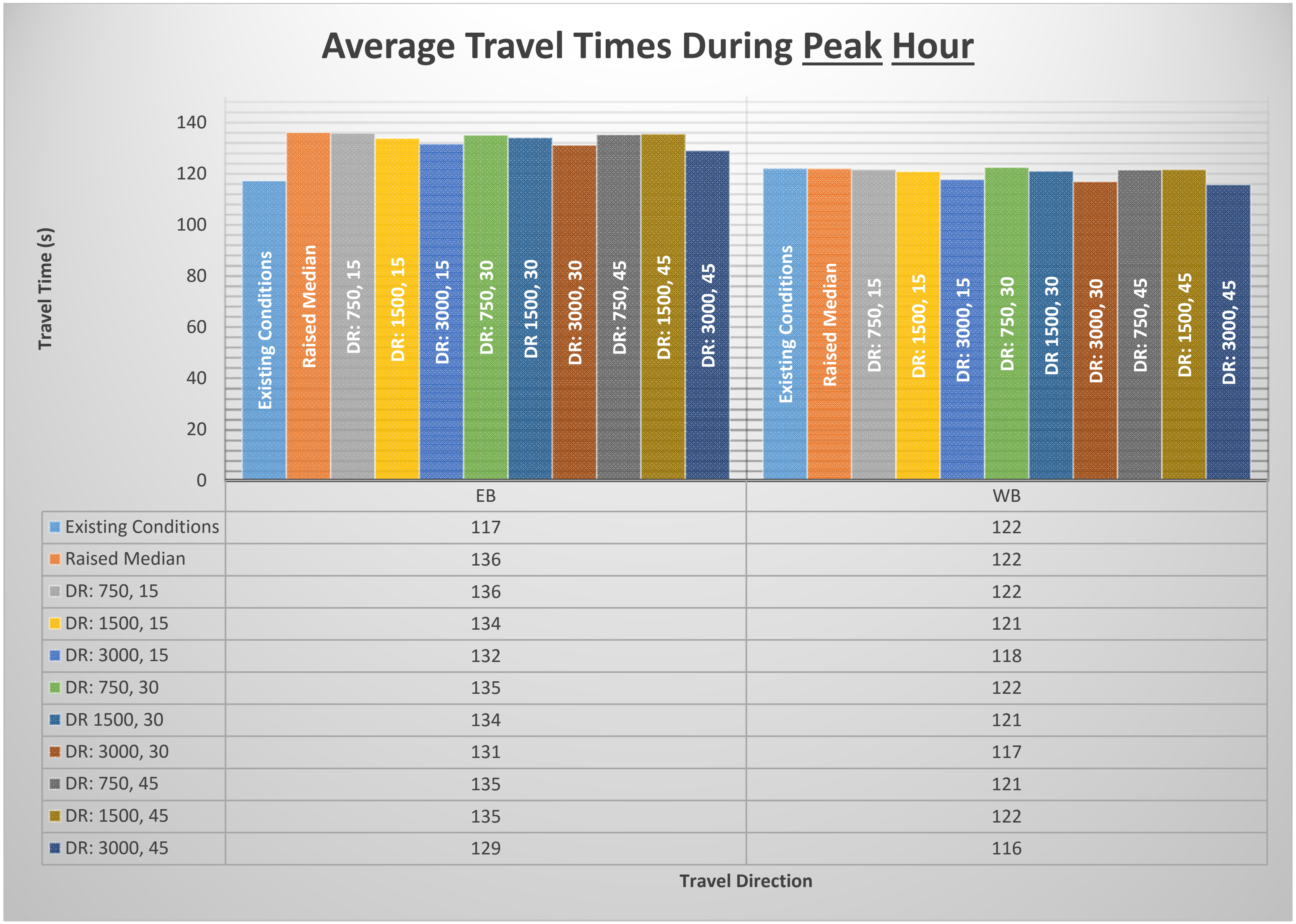


Figure 36*: Wade Hampton Blvd. Demand-Responsive Travel Times for Peak Hour*

Several trends can be seen from the data. First, in the Eastbound direction of travel, for both the average travel times for the entire simulation run and during the peak hour, the alternative scenarios (raised median and demand-responsive) showed increased travel times from the existing conditions. In the Westbound direction of travel, the raised median and demand-responsive alternatives produced comparable or lower travel times, however the difference is very slight.

Similarly to the trends observed on Woodruff Road, another trend can be noted concerning the different demand-responsive scenarios. For each of the response times tested, the highest volume threshold produced the lowest travel times. Additionally, the volume thresholds for each response time were relatively similar. In other words, the changing volume thresholds for the demand-responsive scenarios had a greater effect on the travel times than on the time at which the access control was changed (response time). The 'best' demand-responsive scenario in terms of travel time, in both directions, for both the entire run and during the peak hour was the **DR: 3000, 45** alternative: access changed from fully open to right-in, right-out when the volume reached 3000 vph with the volume recalculated – and control decisions changed – every 45 seconds. Therefore, this demand-responsive scenario will be compared to the existing condition and raised median scenarios. Figures 37 and 38 on the following pages show the Eastbound and Westbound travel times for the entire run as a function of time.

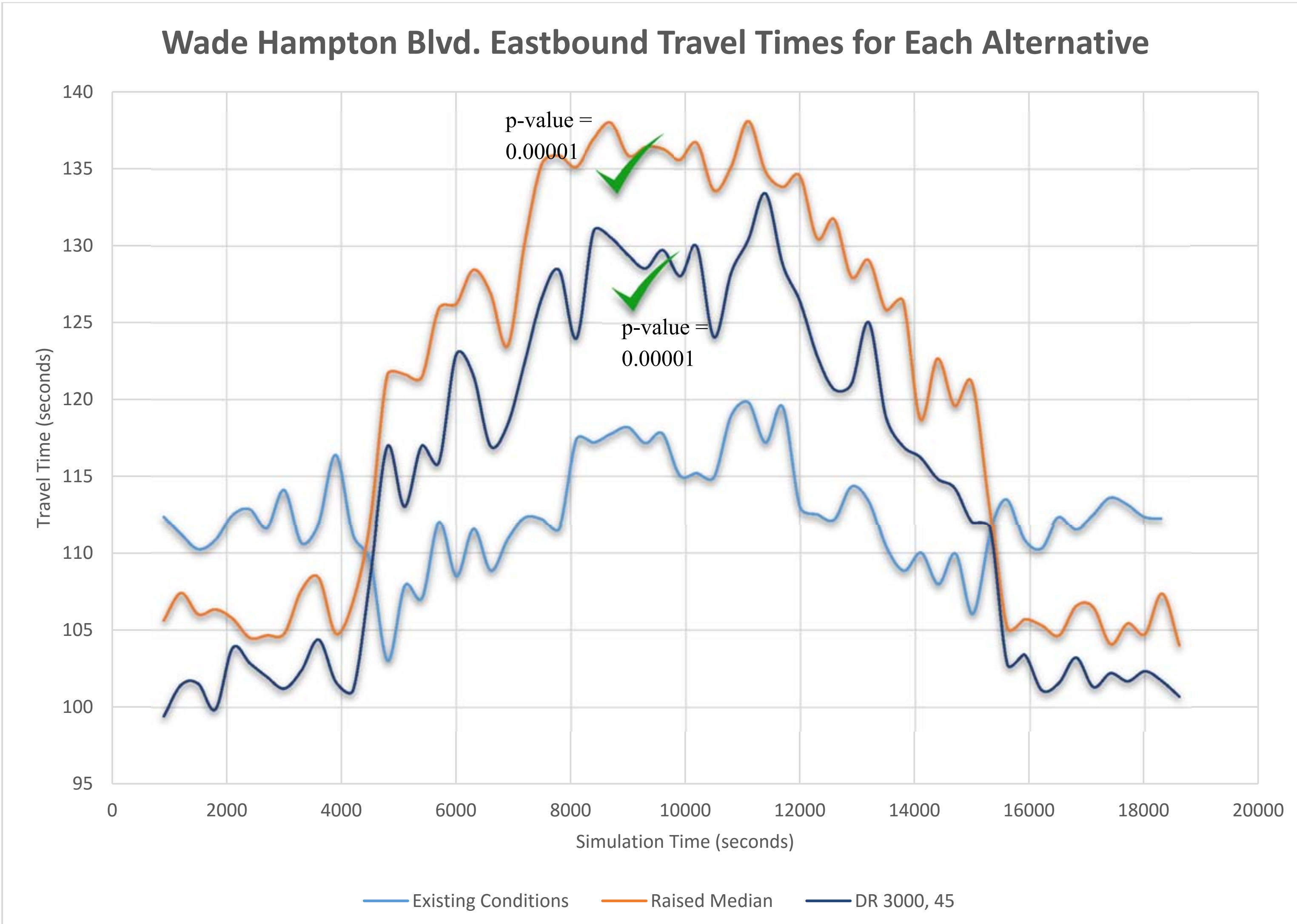


***Figure 37**: Wade Hampton Blvd. Demand-Responsive Eastbound Travel Time as a Function of Simulation Time*

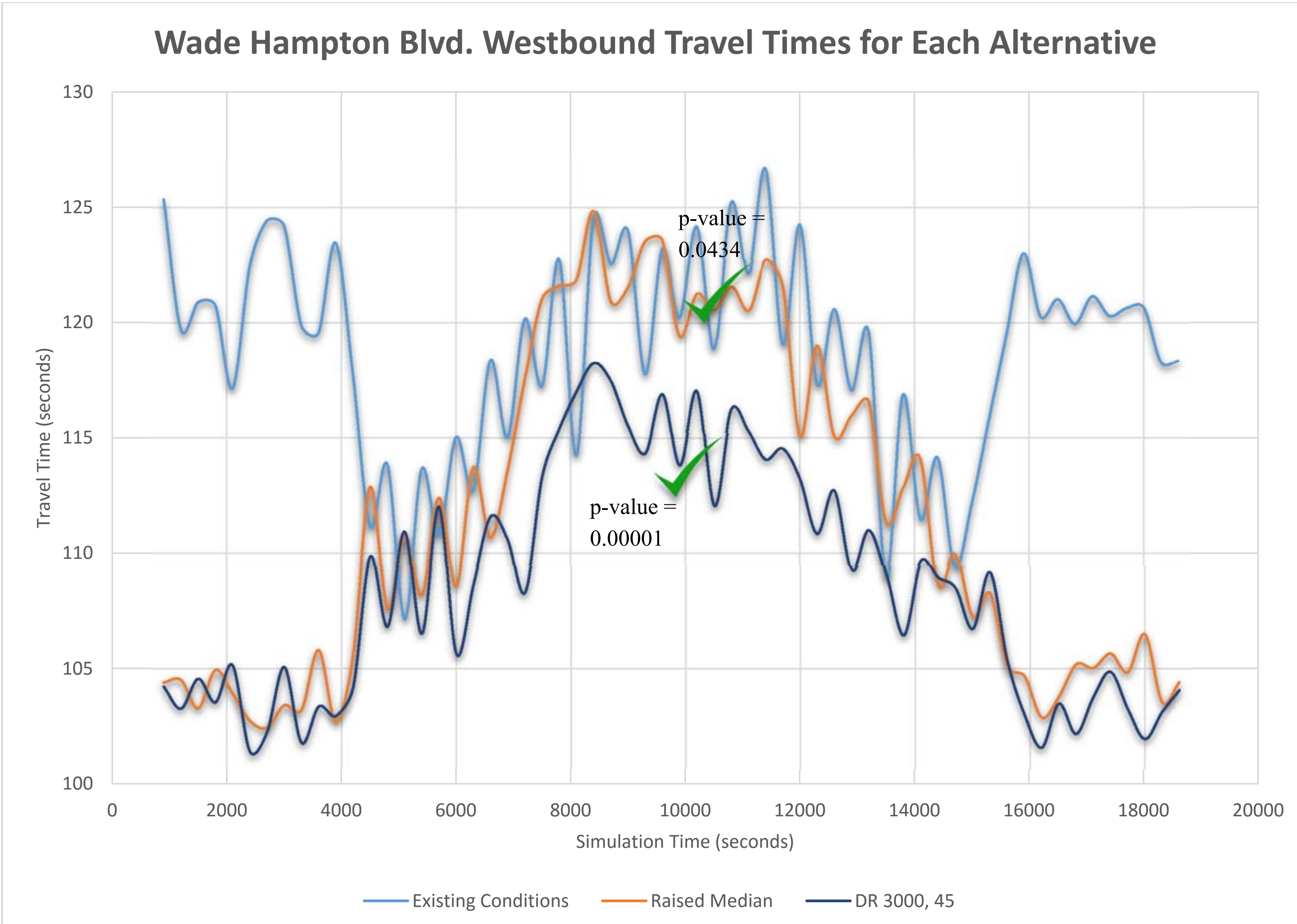


__Figure 38__: Wade Hampton Blvd. Demand-Responsive Eastbound Travel Time as a Function of Simulation Time

Figures 37 and 38 above reveal a difference in resulting travel times by direction of travel. In the Eastbound direction, the Raised Median displayed slightly lower travel times during off-peak traffic conditions, but higher travel times during peak-hour traffic conditions when compared to the existing conditions. The Demand-Responsive Scenario showed even lower travel times during off-peak conditions, and slightly reduced travel times during peak conditions (when compared to the raised median scenario). However, the peak-condition travel times for the Demand-Responsive Scenario were still higher than those of the existing conditions. In the Westbound direction, Existing Condition and Raised Median travel times during peak-traffic conditions were very similar. However, the Raised Median scenario produced lower travel times during the off-peak conditions. The Demand-Responsive scenario, on the other hand, shows lower travel times then both the existing conditions and the raised median scenarios, both in the off-peak conditions, as well as in the peak conditions.

Figure 39 below shows the sum of travel times in both the Eastbound and Westbound directions for a holistic comparison of the different alternatives. From this figure, the Demand-Responsive scenario appears to improve the efficiency of the mainline (in terms of travel times) during both the off-peak and peak conditions, when compared to the Existing Conditions and Raised Median scenarios.

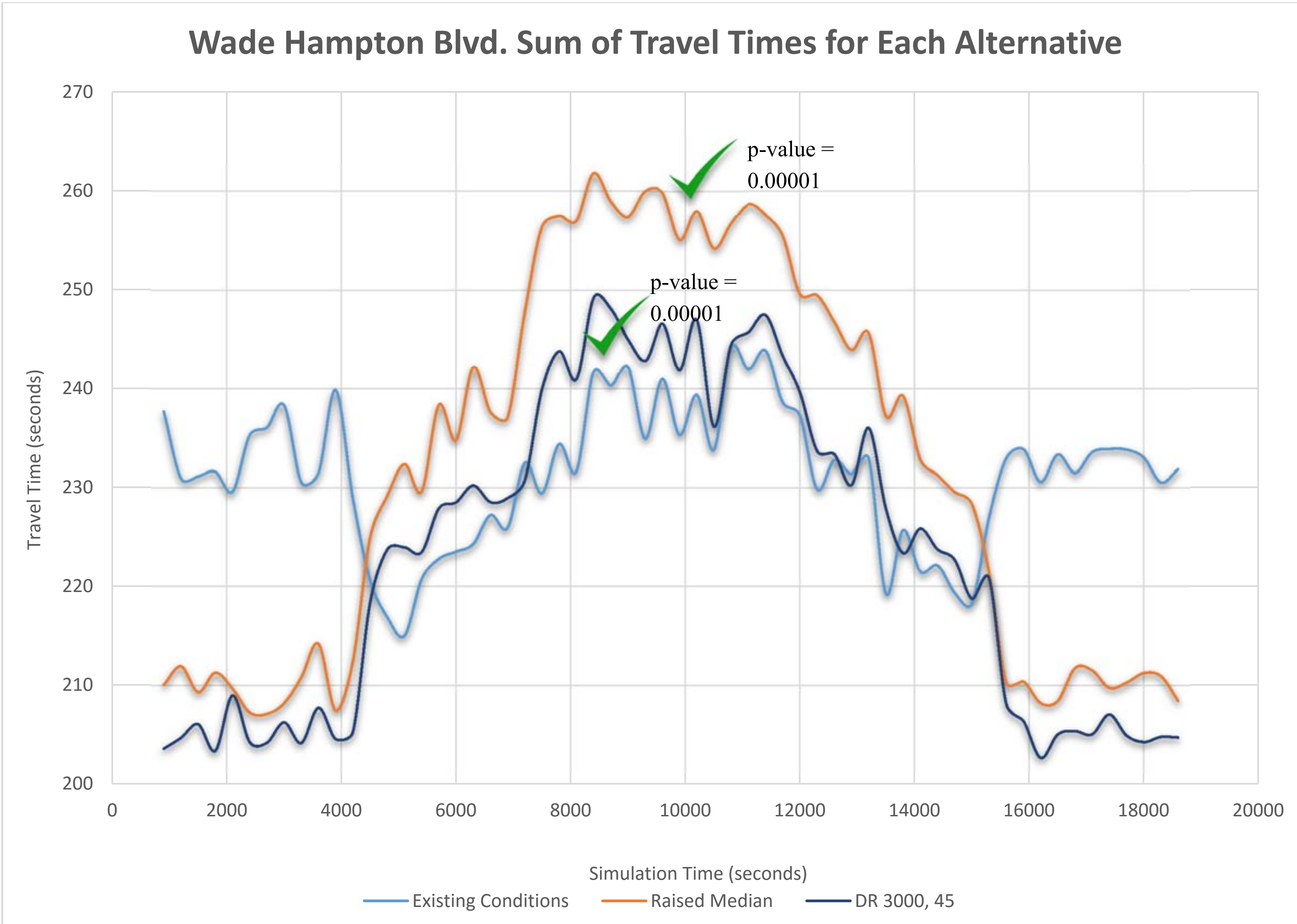
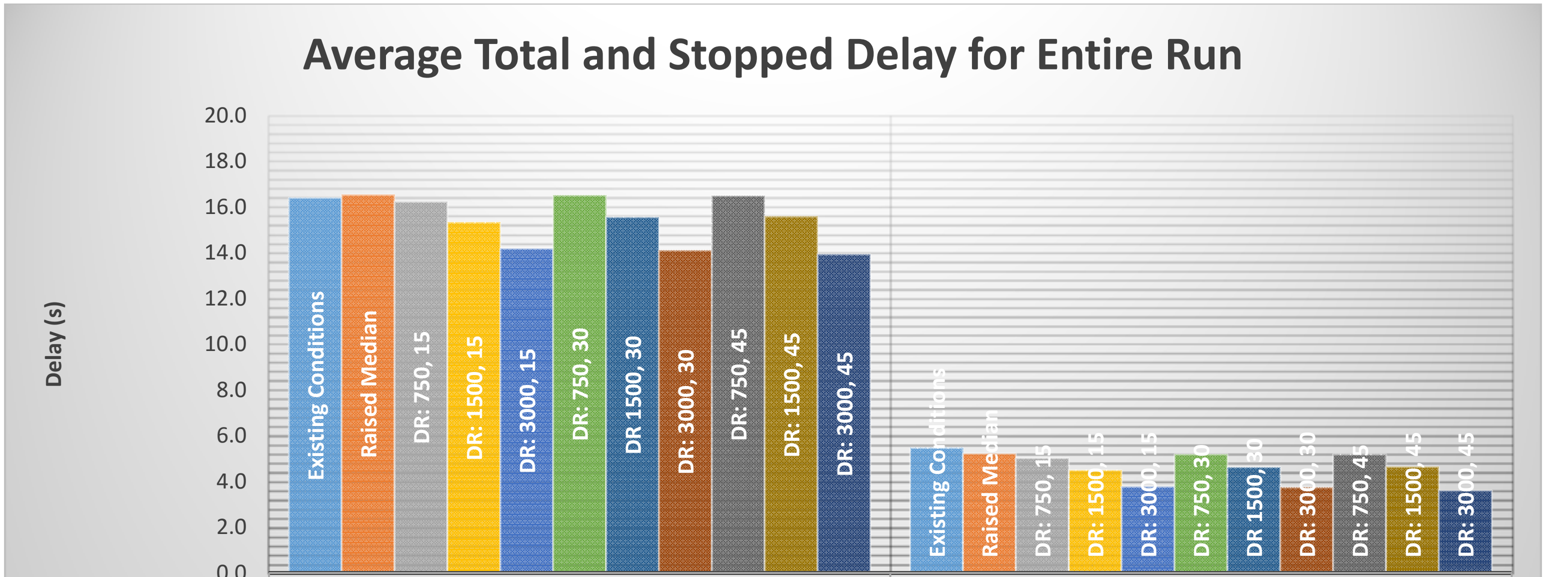


Figure 39*: Wade Hampton Blvd. Demand-Responsive Sum of Travel Time as a Function of Simulation Time*

4.2.2.2. Driveway Traffic Total Delay and Stopped Delay

Figures 40 and 41 on the following pages show the Total Delay and Stopped Delay for the entire 5-hour run and during the peak hour for the 11 scenarios (existing, raised median, and 9 demand-responsive). For the simulation as a whole, the total and stopped delay of the Raised Median scenario was comparable to the existing conditions. During the peak hour, the total delay of the Raised Median scenario was slightly higher than the existing conditions, while the stopped delay was comparable.

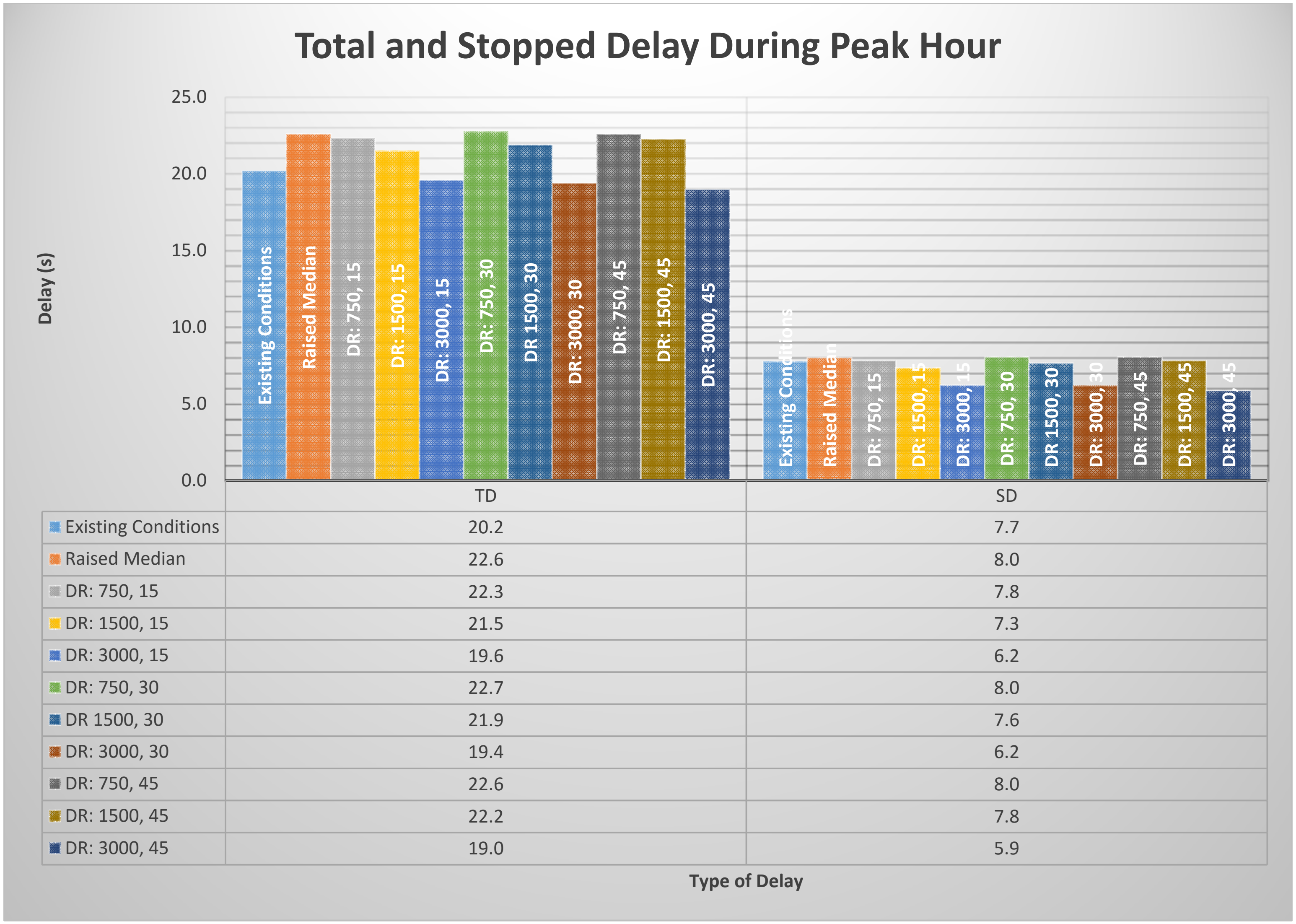
Comparing the 9 different Demand-Responsive scenarios, a similar trend can be observed as that which was seen in the travel time results. For each response time tested, the highest volume threshold produced the lowest stopped and total delay. In other words, changing volume thresholds for the demand-responsive scenarios had a greater effect on the travel times than on the time at which the access control was changed (response time). The 'best' demand-responsive scenario in terms of total and stopped delay was the **DR: 3000, 45** alternative: access changed from fully open to right-in, right-out when the volume reached 3000 vph with the volume recalculated – and control decisions changed – every 45 seconds. It showed lower total and stopped delay for the run as a whole as well as during the peak hour. Therefore, this demand-responsive scenario will be compared to the existing condition and raised median scenarios. Figures 42 and 43 which follow Figures 40 and 41 below show the total and stopped delay for Woodruff Road as a function of simulation time.



	TD	SD
Existing Conditions	16.4	5.5
Raised Median	16.5	5.2
DR: 750, 15	16.2	5.0
DR: 1500, 15	15.3	4.5
DR: 3000, 15	14.2	3.8
DR: 750, 30	16.5	5.2
DR 1500, 30	15.6	4.6
DR: 3000, 30	14.1	3.7
DR: 750, 45	16.5	5.2
DR: 1500, 45	15.6	4.6
DR: 3000, 45	13.9	3.6

Type of Delay

***Figure 40**: Wade Hampton Blvd. Demand-Responsive Total and Stopped Delay for Entire Simulation Run*



	TD	SD
Existing Conditions	20.2	7.7
Raised Median	22.6	8.0
DR: 750, 15	22.3	7.8
DR: 1500, 15	21.5	7.3
DR: 3000, 15	19.6	6.2
DR: 750, 30	22.7	8.0
DR 1500, 30	21.9	7.6
DR: 3000, 30	19.4	6.2
DR: 750, 45	22.6	8.0
DR: 1500, 45	22.2	7.8
DR: 3000, 45	19.0	5.9

Figure 41*: Wade Hampton Blvd. Demand-Responsive Total and Stopped Delay for Peak Hour*

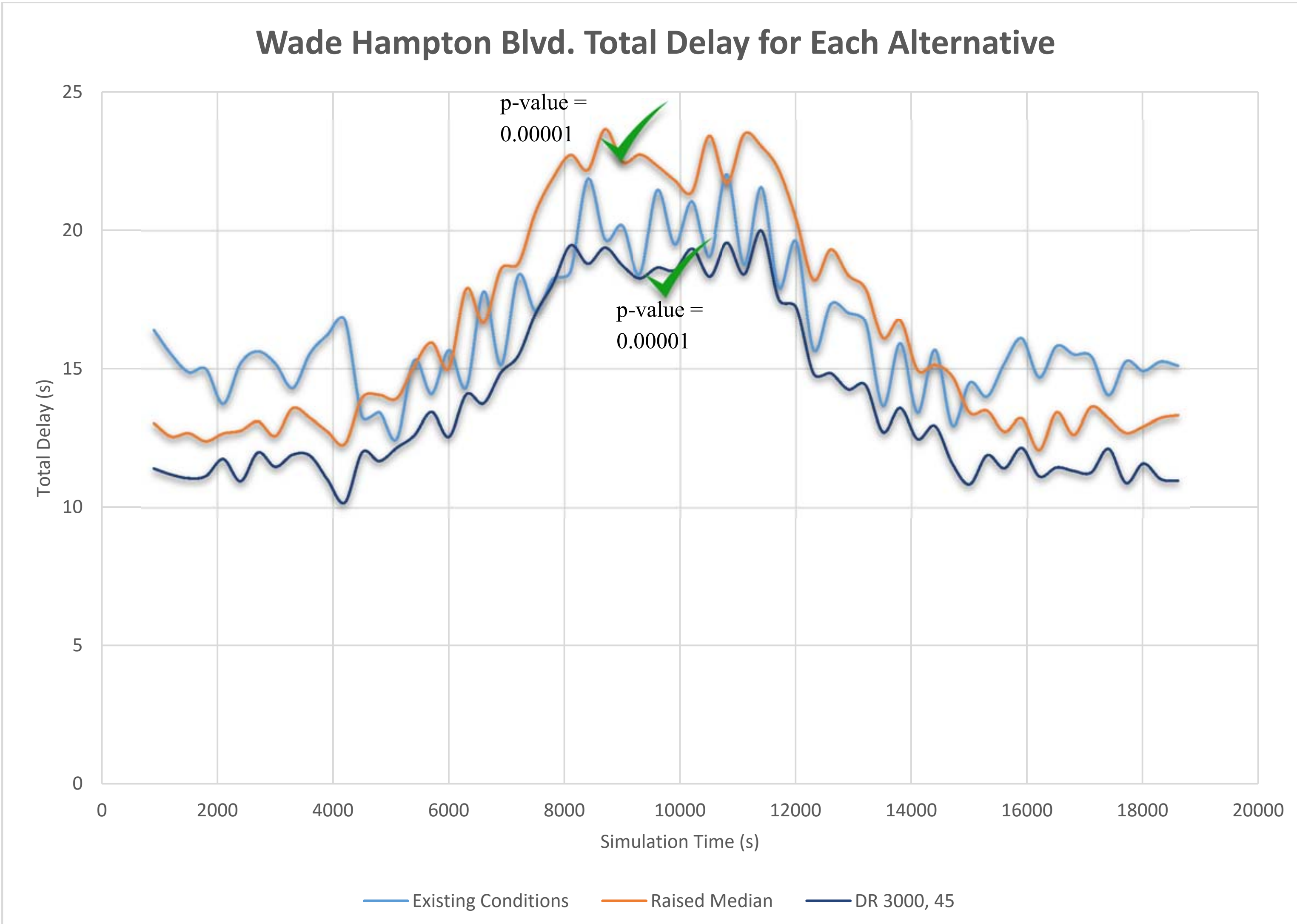


Figure 42: *Wade Hampton Blvd. Demand-Responsive Total Delay as a Function of Time*

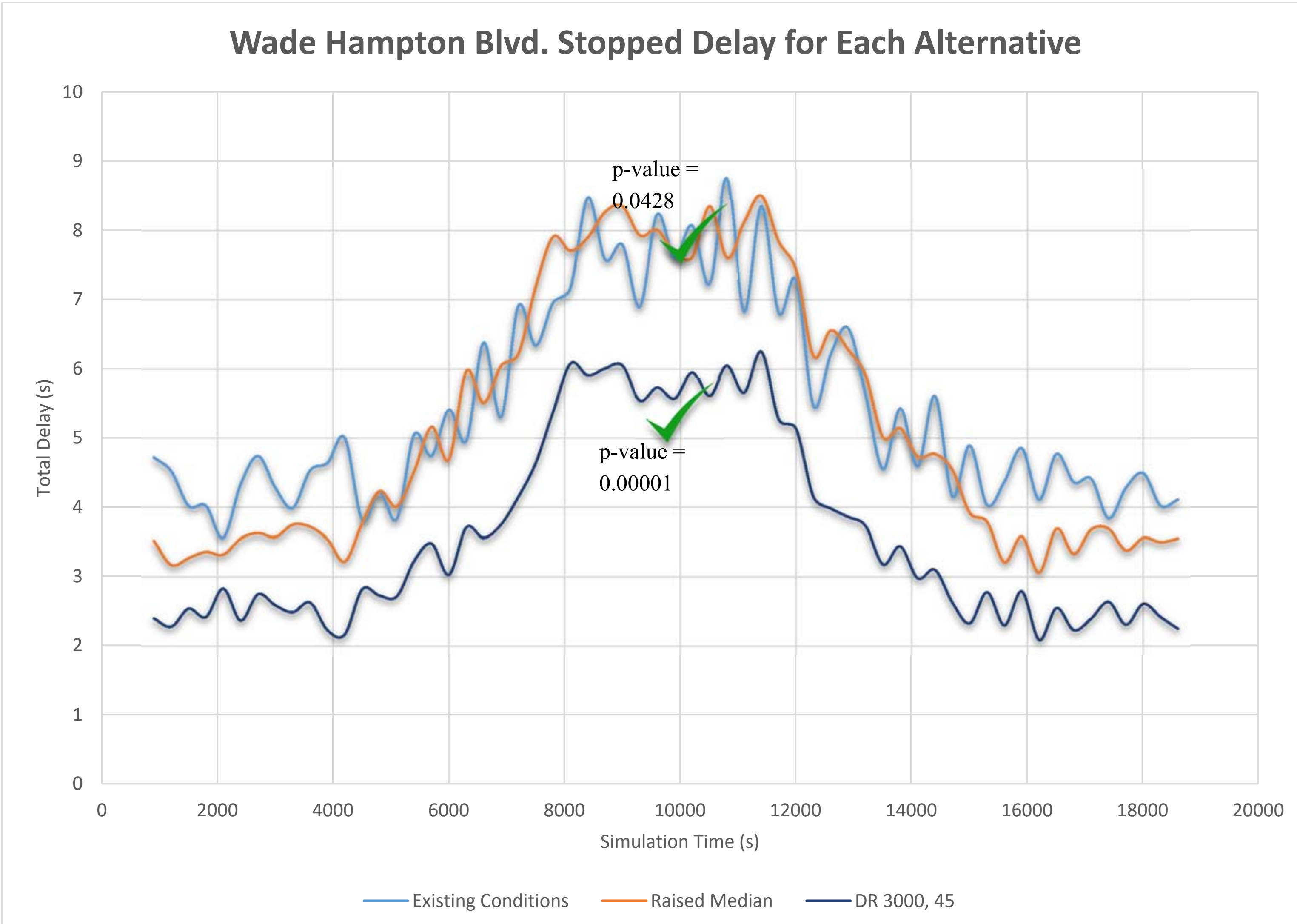


***Figure 43**: Wade Hampton Blvd. Demand-Responsive Stopped Delay as a Function of Time*

Figures 42 and 43 above (showing total and stopped delay as a function of simulation time respectively) show slightly different patterns as the simulation proceeds from start to finish.

Figure 42 shows that the Raised Median total delay was slightly lower during off-peak conditions, and slightly higher during peak conditions, than the existing conditions scenario, while the Demand-Responsive scenario slightly lowered total delay during both the off-peak and peak traffic conditions when compared to both the Raised Median and Existing Conditions scenarios.

Figure 43 shows that the Raised Median stopped delay was slightly lower during off-peak conditions, but comparable to the existing conditions stopped delay during peak conditions. The Demand-Responsive scenario, on the other hand, showed lower stopped delay, when compared to the other two scenarios, for the entirety of the run.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

The two objectives of this research were to (1) quantify and compare the operational impacts of traditional access management strategies (listed in the previous subsection) on arterials and, (2) quantify and compare operational impacts of demand-responsive access control with permanent access control and no access control conditions.

5.1. Objective 1

For the first objective, the four traditional access management strategies studied were *access spacing, corner clearance, access restriction, and implementation of a raised median.* These strategies were implemented on two different corridor segments (5-lane and 7-lane) and analyzed for mainline travel times in both directions of travel. In addition to travel time data, total delay and stopped delay for all ingress and egress driveway traffic along the length of the corridor was also collected in order to provide a more holistic view of the impacts of each. Results of the analysis varied by corridor.

In the Eastbound direction of travel on the 5-lane corridor (Woodruff Road), each of the alternative access management strategies – access spacing, corner clearance, access restriction, and raised median – caused improvements to mainline travel times, decreasing travel times in this direction by 18%, 13%, 9%, and 4% respectively. In the Westbound direction of travel on the 5-lane corridor (Woodruff Road), the access spacing, and corner clearance strategies did not change the mainline travel time, while the access restriction strategy increased travel times by 1% and the raised median strategy decreased travel times by 3%. For this same corridor, the access spacing strategy decreased total and stopped delay by 12%, while the corner clearance strategy decreased total delay and stopped delay by 8% and 12% respectively. The access restriction strategy increased total and stopped delay by 2% and 4% respectively, while the raised median strategy increased total delay by 4% and decreased stopped delay by 8%.

In the Eastbound direction of travel on the 7-lane corridor (Wade Hampton Blvd.), the access spacing, corner clearance, and access restriction strategies decreased travel times by 2%, 1%, and 1% respectively – negligible changes. However, the raised median strategy increased travel times by 15%. In the Westbound direction, there were similarly negligible changes for each of the alternative strategies. The strategy that caused the most noteworthy changes to delay was the raised median strategy which increased total delay by 15% and stopped delay by 4%. The access spacing, corner clearance, and access restriction strategies did not change the total delay, however did cause 4% decrease, 4% increase, and 5% increase in stopped delay respectively.

From these results, the following conclusions, and recommendations, seem appropriate. First, the operational impacts of each strategy are very site specific. For the Woodruff Road corridor, each strategy, for the most part, improved the operational performance of the corridor, whereas for Wade Hampton Blvd., the impacts were less noticeably positive, and in fact tended more towards increases in travel times and delay. With that being said, it did appear that, among the traditional access management strategies tested, the 'access spacing' strategy performed positively on both corridors. ***Therefore, the access spacing strategy, which consolidates driveways such that they achieve the SCDOT***

ARMS Manual spacing requirements, is the most recommended for implementation according to the findings of this research.

5.1.1. Recommendation for Further Research

Another observation from this research is that signal timing has a significant impact on travel times and delays. This perhaps goes without saying, but in observing the simulation run for the raised median scenario for Wade Hampton Blvd. (the scenario which saw the most dramatic increases in MOE's), it appeared that the increase was due primarily to signal timing changes needed to accommodate the additional U-turning traffic, which gave less green time to the through movements than in the existing signal timing plans, thereby increasing travel times. *Accordingly, a recommendation for further research from Objective 1 of this study is to explore further the signal optimization for different access management strategies used.* There may be other signal timing plans that would improve travel times and delay for alternatives which create significant numbers of U-turning/left-turning movements.

5.2. Objective 2

For the second objective, three alternatives were tested and compared: (a) Existing Conditions, (b) Permanent Access Control (simulating a raised median) with U-turns handled at signalized intersections, and (c) Demand-Responsive Access Control according to prevailing volumes on the mainline.

As in Objective 1, the results differed according to the corridor. On Woodruff Road, implementing a raised median (scenario 'b'), greatly decreased travel times during the peak hour but not during off-peak conditions, whereas on Wade Hampton Blvd., implementing a raised median increased travel times during the peak hour but decreased them during off-peak conditions. Similarly, the raised median greatly decreased total and stopped delay on Woodruff Road during peak traffic conditions while not changing them during off-peak conditions, while conversely, for Wade Hampton Blvd., the raised median did not change total or stopped delay for the entire run-time (5-hours). The results from the raised median scenario served as a comparison scenario (in addition, of course to the existing conditions) for the primary alternative in question for this objective: *demand-responsive access control.*

The first step in comparing demand-responsive access control to the other two scenarios (existing conditions and raised median), was to determine the demand-responsive parameters which produced the 'best' results, in terms of travel times and delay. Three different volume thresholds (750, 1500, and 3000 vph) were tested for three different response times (15, 30, and 45 seconds) for a total of nine (9) different demand-responsive scenarios. The volume threshold represented the volume at which the access was changed to only right-in/right-out. For response times in which the volumes were less than these thresholds, the road operated with fully-open access (with lefts being handled via the TWLTL). Comparing the 9 different demand-responsive scenarios revealed that the 3000 vph, 45 second scenario produced the lowest travel times and delays for both corridors.

Comparing this demand-responsive scenario, then, to the other two scenarios on both corridors allows for the following conclusions. Despite the difference in how the raised median performed on each corridor, the demand-responsive strategy lowered travel times and delays. In other words, on Woodruff Road, the demand-responsive strategy produced even lower travel times and delays than the raised median – which had already greatly reduced these MOEs compared to the existing conditions. And on Wade Hampton Blvd, the demand responsive scenario lowered travel times such that the lower travel times during the off-peak conditions produced by the raised median were still experienced, and the higher travel times of the raised median scenario were mitigated to the point that the travel times were very similar to that of the existing conditions.

Therefore, it is the conclusion of this research that alternating access between fully-open to right-in/right-out based on prevailing traffic conditions, has the potential to get the most out of a corridor, by producing lower travel times and delays during both peak and off-peak traffic conditions.

5.2.1. Recommendation for Further Research

As with Objective 1, it was observed that signal timing had a noticeable impact on the MOE's analyzed. Signals were optimized for the raised median scenario and these signal timings used for the demand-responsive scenarios. However, the timings did not change as access control changed. Therefore, a recommendation for further research would be to explore dynamic signal timing along with dynamic access control –

alternating signal timing with access control to further maximize travel times and delays.

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