

ABSTRACT

GONTARUK, FEDERICO JORGE. Operational Impacts of Double Crossover Diamond Interchanges. (Under the direction of Dr. Joseph E. Hummer.)

Operational and safety issues at many freeway-to-surface street interchanges have increased in recent years, as they are unable to handle larger traffic volumes. Limitations in right-of-way availability, high land prices, and expensive structures have pushed engineers to find innovative solutions to solve some of these issues. Examples of these innovative designs include the roundabout interchange, the single point urban interchange, and the double crossover diamond (DCD) interchange (also known as the diverging diamond interchange), among others. This thesis provides results from the first year of the major study commissioned by the FHWA to evaluate the first few DCD installations in the US. This thesis includes findings on operational performance at DCD interchanges built in recent years at I-44 and MO-13 in Springfield, MO; US-60 and National Avenue in Springfield, MO; I-270 and Dorsett Road in Maryland Heights, MO; and US-129 and Bessemer Street in Alcoa, TN. The thesis also includes an analysis of queue interaction and movement capacities at DCD interchanges. This thesis showed that DCDs are especially beneficial to left turn movements, and that progression could be also achieved for arterial through movements. The thesis also provides a model to calculate capacity at DCD interchanges based on previous research on queue interaction at closely spaced intersections. Finally, the thesis describes the program of future work planned to fully investigate the operational and safety impacts of DCDs.

Operational Impacts of Double Crossover Diamond Interchanges

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1.0 INTRODUCTION

1.1 Problem Definition

Many freeway-to-surface street (i.e., service) interchanges in the United States have reached their design life and experience significant operational and safety problems. Especially serious are interchanges in urban, suburban, and fringe areas where the traffic volumes have grown up through the years to exceed the capacity of the interchange. In such cases, queues may build up on the surface street blocking other intersections, or queues may build up on the off-ramps to block freeway lanes. Safety problems arise due to spillback or due to drivers experiencing undue delay and feeling pressure to act, making poor decisions in gap acceptance or lane changing.

The traditional simple diamond interchange exhibits many of these problems in areas where traffic demands have outgrown service life predictions. It relies on vehicle storage between the ramp terminals, and is thus susceptible to spillback, and it is difficult to establish progression for surface street traffic through both signals. Conventional solutions to congested simple diamond interchanges, including bridge widening, ramp widening, shortening the distance between ramp terminals (creating a “tight diamond”), and signal system installation, can bring some relief. But benefits are often temporary as the traffic volumes continue to grow. Changing the nature of interchanges by adding one to four loop ramps (creating a partial or full cloverleaf interchange) often requires large amounts of expensive right-of-way (ROW), and may create unsafe and inefficient weaving areas. Agencies have also constructed flyover structures to carry left turns or through traffic on a third level over the existing interchange. This is of course an extremely expensive and disruptive option, and often still leaves an operational and safety problem on the surface street level.

Unconventional interchange design solutions provide a menu of other options that agencies can explore to overcome the pitfalls associated with conventional solutions for congested or unsafe diamond interchanges. Unconventional designs typically involve

rerouting one or more movements—often left turns—to reduce the number of conflict points remaining in the middle of the interchange. This allows a reduction in signal phases, less lost time, fewer opportunities for crashes, and a host of other potential operational, safety, and environmental benefits. The most prominent unconventional service interchange design with no loops and no (substantive) weaving at the moment is the double crossover diamond (DCD) interchange, also known as the diverging diamond interchange (Figure 1).

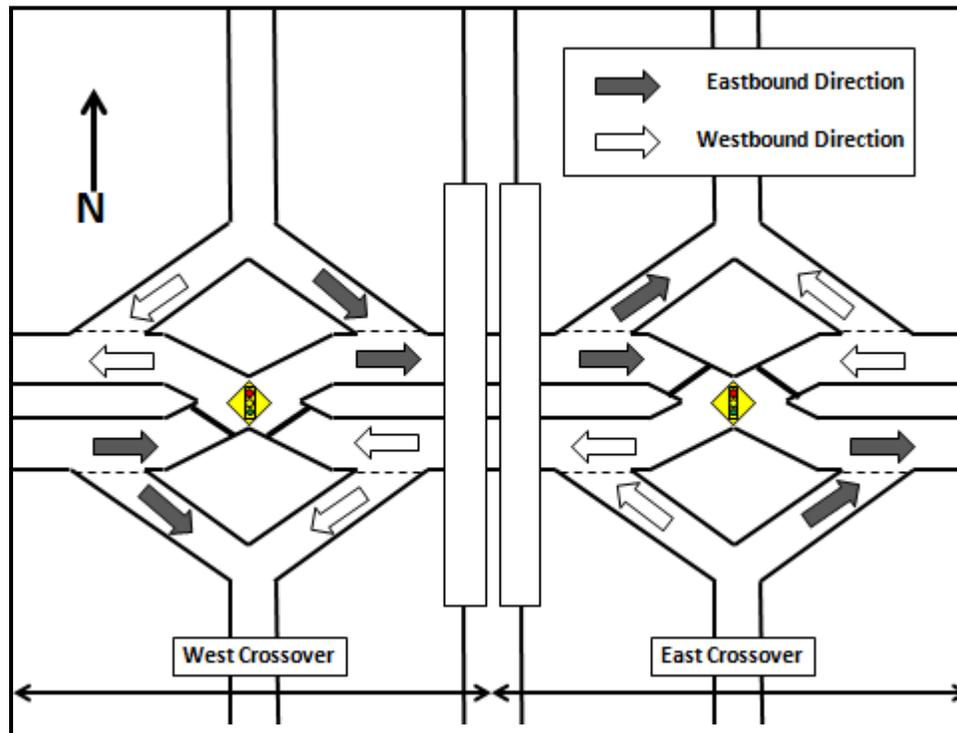


Figure 1: Schematic Illustration of DCD Interchange (East-West Arterial)

1.2 Background

The general layout of a DCD interchange is similar to other diamond-type designs; however, unique to DCD are two signalized crossovers located at each end of the interchange where surface street traffic crosses to the left side of the roadway. This feature allow for direct left turns into the freeway on-ramp and from the freeway off-

ramp to the surface street. The off-ramp left turn and the on-ramp right turn movements could operate either as signalized or unsignalized movements.

Consider the DCD schematic illustration in Figure 1. Vehicles approaching the interchange from the east and continuing through the interchange in the westbound direction, will switch to the left side of the road at the East Crossover. Later, they will switch back to the right side of the road at the West Crossover. If a vehicle going in the westbound direction needs to turn to the freeway on the southbound direction, it will make a free flow left turn movement just before the West Crossover. A vehicle exiting the freeway in the northbound direction, and making a left turn movement at the DCD interchange, will merge directly with the westbound through movement at the East Crossover. This merge left turn movement could be signalized or unsignalized. Right turn movements on-ramp or off-ramp operate just as in a conventional diamond interchange.

Double crossover diamond interchanges were introduced in France in the 1970s. Currently, there are three locations where this type of interchange has been constructed in France: Versailles, Le Perreux-sur-Marne, and Seclin. In the United States, I-44 and MO Route 13 in Springfield, Missouri was the first location where a DCD interchange was constructed and opened in June 2009. Other DCD interchanges have opened in American Fork, Utah; Alcoa, Tennessee; Springfield, Missouri; Maryland Heights, Missouri; and Lexington, Kentucky. Other locations where DCD interchanges are under construction or in late design stages include Rochester, New York; Kansas City, Missouri; and American Fork, Utah (1). Refer to Figure 2 for an illustration of the I-44 and Route 13 DCD interchange in Springfield, Missouri.



Figure 2: I-44 and Route 13 DCD interchange in Springfield, Missouri.

1.3 Research Objectives

This thesis provides results from the first year of the major study commissioned by the Federal Highway Administration (FHWA) to evaluate the first few DCD installations in the US. Based on the project objectives outlined by FHWA, the purpose of this research is to:

1. Evaluate the operational impacts of converting an existing diamond interchange into a DCD through *before/transition* and *after* study, and
2. Investigate how queue interaction affect capacity at DCD interchanges and develop a model to accurately calculate capacity, and optimize offsets and signal timings.

Operational impacts of DCD interchanges will be analyzed through travel time, queue length, and other data collected at the sites under study. Comparison of travel time data at the DCD interchange (after scenario) with travel time data along the same road before construction (before scenario) or within the first two weeks after opening (transition

scenario), will assist determining if the DCD interchange has any positive impact in travel time and progression for drivers. Queue length comparison will help determining if capacity at the interchange improved after construction of the DCD.

1.4 Scope

The scope of the research included in this report will be limited to four recently constructed DCD interchanges in the United States. The interchanges are:

1. US Route 129 and Bessemer Street, Alcoa, TN;
2. I-44 and MO-13, Springfield, MO;
3. US Route 60 and National Avenue, Springfield, MO; and
4. I-270 and Dorsett Road, Maryland Heights, MO.

Site 1 will be analyzed for *transition* and *after* conditions; while sites 2, 3 and 4 will be analyzed for *after* conditions only.

The three analysis periods are defined as follows:

- *Before*: One week of data collection prior to beginning of construction at the DCD site with a focus on operational performance measures.
- *Transition*: One week of data collection performed within two weeks of opening at the DCD site under the new traffic configuration.
- *After*: One week of data collection performed between three and six months of opening at the DCD site under the new traffic configuration.

1.5 Organization of Thesis

This thesis is organized into six chapters. This first chapter described the problem, background information on DCD interchanges, research objectives, and scope of the project. The second chapter presents a literature review of relevant studies on DCD interchanges. Chapter three describes the methodology of the operational study, which

includes the site selection procedure, data collection and data analysis methods. Chapter four contains the results of the operational analyses. Chapter five presents a paper investigating queue interaction at DCD intersections. Finally, chapter six contains the conclusions of the research project, including recommendations and future research needs.

2.0 LITERATURE REVIEW

2.1 Double Crossover Diamond Interchanges

While the DCD Interchange is a recent unconventional design alternative in the United States, the first of this type of interchange was built in France in the 1970s. Not much is known about the conditions that led to the creation of the DCD in France; however, the first to propose this type of design in the United States was Chlewicki in a paper submitted to the 2nd Urban Street Symposium in Anaheim, California in July 2003 (2). In that paper, Chlewicki discussed two types of intersection/interchange design: synchronized split-phasing intersection and the DCD (introduced as the “diverging diamond interchange” in the paper). With regards to the DCD, Chlewicki compared a conventional diamond interchange located at I-695 (Baltimore Beltway) and MD 140 (Reisterstown Road) in Baltimore County, Maryland, with a DCD design at the same location. The report utilized Synchro 5 to compare the phasing and geometric strategies and SimTraffic 5 to compare operational performance. The main findings were that the total delay for the conventional diamond was about three times as great as the DCD, the stop delay was over four times worse for the conventional diamond, and that total stops were approximately half for the DCD when compared to the conventional diamond interchange.

Bared, Edara, and Jagannathan conducted a more extensive VISSIM analysis of DCD interchanges under two scenarios: a four-lane and a six-lane DCD interchange design. Each of the scenarios was further analyzed under high, medium, and low volumes conditions (3). The results of the operational performance of DCD interchanges were compared to a conventional diamond interchange. The measure of effectiveness (MOE) included average delay time per vehicle, average stop time per vehicle, average number of stops per vehicle, average queue length, and maximum queue length. Figure 3, with a summary of the delay results for the 4-lane surface street designs, shows that the DCD was far better at the higher levels of demand than the simple diamond. An evaluation of

pedestrian delay showed that the DCD processed them fairly well. In summary, the major findings of the Bared, Edara, and Jagannathan study showed that DCD demonstrated better performance and offered lower delays, fewer stops, lower stop times, and shorter queue lengths, especially for higher volumes. Also, the service volume of left-turn movements at DCD interchanges was twice that of the corresponding left-turn service volumes of the conventional diamond.

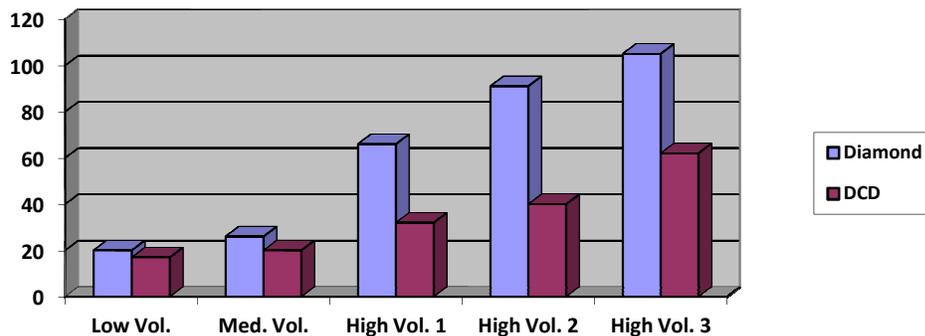


Figure 3: Delay (sec/veh) results for 4-lane surface street (3)

Speth performed a comparison of DCD interchanges to a conventional diamond, and a single point urban interchange (SPUI) by utilizing Synchro 7 for signal timing optimization and SimTraffic and VISSIM 4.2 for development of operational performance outputs (4). The MOEs analyzed included average vehicle delay, queue lengths, number of vehicles served, and average number of stops. As in the Bared et al study, the DCD performed better than the conventional diamond and SPUI under all volume scenarios, but especially under higher volumes.

In 2007, FHWA presented the results of an evaluation of driver behavior on a DCD, utilizing an advanced driving simulator (5). The study was conducted utilizing a very complete set of signs, a DCD with a reduced set of signs, and a comparable simple diamond. The results showed that the DCD with full signing or the DCD with reduced signing had fewer driver errors, fewer red light violations, and lower mean speeds than

the diamond. The researchers concluded that any concern about wrong way movements at a DCD is “not warranted” and that the DCD will deliver safety benefits over a simple diamond where it is adopted.

FHWA also prepared a report in 2010, titled “Alternative Intersections/Interchanges: Informational Report (AIIR)”, where an extensive chapter on DCD is presented (6). The chapter discusses important aspects of the design, including signal phasing, signing and marking, lighting, pedestrian and bicyclist accommodation, construction sequencing, and access management at nearby properties. The AIIR chapter concludes with a statement that the DCD would be most applicable at junctions with:

- Heavy volumes of left turns onto freeway ramps;
- Moderate and unbalanced through volumes on bridge approaches on the arterial road;
- Moderate to very heavy off-ramp left turn volumes; and
- Limited bridge deck width availability.

The Missouri Department of Transportation (MoDOT), as a pioneer in the design and construction of DCD interchanges in the United States, prepared a report in 2010 describing the experience of the agency with DCD interchanges (7). In the report, MoDOT presented guidelines for the design, construction, and operation of DCD interchanges. Some considerations included:

- Design Speed: Passenger vehicles should be able to proceed through the DCD at 20-30 mph; MoDOT’s past and current designs are allowing speeds of about 25 mph. Also, WB-67s, MoDOT’s designated design vehicle, should be able to proceed through a DDI at 20 mph and make all turning movements to and from ramps at 15 mph;
- Horizontal Crossover Geometrics: Missouri has used crossing angles ranging from 40-50° in its designs. Tangent length should be 15-20 feet along the inner

edge of pavement before the crossover, and 10-15 feet after the crossover. Curve radii used in MoDOT DCD designs range generally from 150-300 feet;

- Lane Width: MoDOT recommends a lane width of 15 feet along the through lanes of the DCD interchange. Standard lane width may be achievable when the horizontal geometrics allow it;
- Traffic Separation between Crossovers: Since driving in the left side of the road is counterintuitive for drivers, MoDOT has provided physical separation between opposing directions of traffic at DCD interchanges for channelization and safety reasons;
- Signals: Nearside signals should be considered. Off-ramp right and left turn signalization may be required for safety reasons;
- Signing: Guide signing should be carefully designed due to the absence of clear guidelines. Missouri used a mirrored KEEP RIGHT regulatory sign to provide an additional reminder to KEEP LEFT;
- Pedestrians: Signs could help pedestrians to look to the correct side for oncoming traffic. In Missouri, pedestrians facilities have been designed along the median of the DCD interchange or on the outside of the through lanes;
- Glare Screens: If glare screens are used, it is important not to block the sight distance of vehicles traveling in opposing directions on the cross route;
- Signal Operations: When signalizing the off-ramp lefts, the distance between the crossover intersection and the “distance to clear” for the off-ramp left turn can be significant, which increases the amount of yellow and all-red intervals. Since signal phases are reduced, one controller can be used to accommodate both signalized intersections.

Xu et al presented a paper at the 2011 Transportation Research Board (TRB) Annual Meeting describing a new method to calculate control delay at DCD interchanges (8). The report utilized a previous model for determining control delay at conventional diamond interchanges, and added a function to calculate delay of internal movements at a

DCD. The performance of the new model was examined in traffic simulation and compared with the simulated control delay results.

In February 2011, MoDOT and HDR Engineering, Inc. presented the results of a performance evaluation for the first DCD constructed in the United States, located at I-44 and MO-13, in Springfield, MO (9). This evaluation analyzed traffic operations, safety and public perceptions between the previous conventional diamond design and the DCD interchange. Traffic operation aspects were analyzed utilizing VISSIM, and compared “pre-construction” to “post construction” conditions. Some of the MOEs under analysis were average delay time per vehicle, total delay time, total stops, and travel times. Major findings of the MoDOT/HDR report included:

- Traffic Operations:
 - Decrease in traffic delay and traffic queuing for the left turn movements within the DCD;
 - Slightly increase in travel time for through movements along the DCD;
 - Increase in traffic volumes will be handled more efficiently by the DCD when compared to a conventional interchange design;
 - Over-sized loads up to 18 foot wide and 200 foot long have successful moved through the DCD;
 - Better overall traffic flow performance of the DCD compared to a conventional diamond interchange.
- Safety Aspects:
 - Total crashes were down by 46% in the first year of operation;
 - Introduction of free-flow or yield control left turn movements at the DCD reduced left turn right angle crashes by 72% and eliminated left turn type collisions;
 - Slightly reduction in rear-end crashes, possibly due to how left turns are handled at a DCD interchange;

- No definite crash pattern was noticed in the review that could lead to conclude that within a DCD a certain type of crash increased.
- Public Perception Survey:
 - Approximately 80% of public expressed that traffic flow had improved and traffic delay had decreased at DCD interchange;
 - A very large percentage (87%) stated that a crash was more likely to occur at a conventional interchange when compared to a DCD interchange;
 - Approximately 80% of the public expressed approval on how pedestrian, bicyclists and large trucks are handled in the DCD when compared to a conventional diamond design;
 - A very high percentage (91%) understood how to navigate through a DCD interchange, and expressed good understanding on signing, pavement markings, current design of islands, and signals.

As a final note, Time Magazine introduced the concept of the DCD interchange to the general public in their February 7, 2011 issue (10). In this article, the author highlights the safety aspects of the DCD interchange, as well as providing an illustration describing the basic traffic operations of the DCD.

2.2 Queue Interaction at Closely Spaced Intersections

Chapter five of this thesis presents a paper investigating queue interaction at DCD intersections. Previous work on queue interaction and coordination includes research by Rouphail and Akçelik (11), Prosser and Dunne (12), Chaudhary et al (13) and Sun et al (14).

Rouphail and Akçelik (11) introduced in 1992 a model for analyzing queue interaction at paired intersections. This model utilizes an iterative method to calculate a reduced saturation flow rate at the upstream intersection, when downstream queues prevent discharge of the upstream intersection at its full saturation flow rate. The iterative process is applied until an equilibrium state is achieved between the length of the downstream

queue and the discharge characteristics of the upstream platoon. This model was limited to one upstream and one downstream traffic stream. Further, the effective green period of the upstream signal is not changed.

Prosser and Dunne (12) continued research on queue interaction and presented a graphical model to calculate movement capacities by reducing the effective green at the upstream approach, when a downstream queue interferes with the upstream traffic discharge. This model allows for multiple upstream traffic streams, and it is the most closely model applicable to the DCD interchange case. This paper will utilize the Prosser and Dunne model as the basis for the DCD interchange queue interaction model.

Chaudhary et al (13) developed a procedure for coordinating diamond interchanges with adjacent traffic signals, and applied the methodology to two sites in Texas. The report recommends using the procedure for coordinating adjacent signals when a four-phase cycle is selected. The primary objective of the methodology was to provide progression to the arterial through traffic at the interchange, while the secondary objective was to provide progression to the adjacent street traffic turning traffic that passes through the interchange.

Sun et al (14) presented a method of addressing stochastic variation at closely spaced signalized intersections to provide secondary coordination to minor movements with significant traffic volumes. Using a secondary signal controller that overrides the phase termination decisions of the conventional controller, the traffic from the upstream intersection can arrive and join the queue at the downstream left-turn lane, reducing systemwide average delay and number of stops per vehicle under a wide range of traffic volumes by nearly 20% under heavier demand conditions.

3.0 OPERATIONAL METHODOLOGY

This chapter describes the methodology used for the operational analysis conducted for this thesis. The operational analysis involved comparison of *transition* conditions to *after* conditions for the Alcoa DCD site and evaluations of *after* conditions for the MO-13 Springfield DCD site, the National Avenue Springfield DCD site, and the Maryland Heights DCD site. Also, an calibration and validation of *after* conditions using the Synchro/SimTraffic simulation model for the MO-13 Springfield DCD site will be presented. This chapter describes the process behind the observation periods, site selection, data collection, and data analysis.

3.1 Analysis Periods

The three analysis periods are defined as follows:

- *Before*: One week of data collection prior to beginning of construction at the DCD site with a focus on operational performance measures. Since all of the DCD interchanges under study had been opened to traffic or under construction at the beginning of this project, before data collection was not performed for this study.
- *Transition*: One week of data collection performed within two weeks of opening at the DCD site under the new traffic configuration. Data items included in this report are peak-hour queue lengths, erratic maneuvers, field-observed conflicts, signal compliance, 15-minute traffic volumes, saturation flow rate, and origin-destination travel times.
- *After*: One week of data collection performed between three and six months after opening at the DCD site under the new traffic configuration. Data items included in this report are peak-hour queue lengths, erratic maneuvers, field-observed conflicts, signal compliance, 15-minute traffic volumes, saturation flow rate, and origin-destination travel times.

3.2 Site Selection

At the time of the beginning of this research project, only three DCD interchanges were open in the United States. Those interchanges included: I-44 and MO-13, Springfield, MO (opened in June 2009), US Route 60 and National Avenue, Springfield, MO (opened in July 2010), and I-15 and Pioneer Crossing, American Fork, UT (opened in August 2010). I-270 and Dorsett Road, Maryland Heights, MO (opened in October 2010) and US Route 129 and Bessemer Street, Alcoa, TN (opened in January 2011) opened soon after the project started in September 2010.

The scope of work described in the FHWA request for proposal (RFP) specifically asked for analysis of four interchanges in the first year report, and two additional interchanges in the final report. The DCD interchanges to be included in the first year report were I-44 and MO-13, Springfield, MO, US Route 60 and National Avenue, Springfield, MO, I-15 and Pioneer Crossing, American Fork, UT, and I-270 and Dorsett Road, Maryland Heights, MO. After consultations with the FHWA, analysis of the interchange at I-15 and Pioneer Crossing, American Fork, UT was replaced by the recently opened in US Route 129 and Bessemer Street, Alcoa, TN.

This thesis will focus on the four interchanges to be studied in the FHWA research project. The two remaining interchanges will be analyzed in a concurrent research project being conducted at the Institute for Transportation Research and Education at North Carolina State University.

In summary, this report will analyze the following DCD interchanges:

1. US Route 129 and Bessemer Street, Alcoa, TN;
2. I-44 and MO-13, Springfield, MO;
3. US Route 60 and National Avenue, Springfield, MO;
4. I-270 and Dorsett Road, Maryland Heights, MO.

3.2.1 US Route 129 and Bessemer Street, Alcoa, TN DCD Interchange

The Alcoa, TN site (US Route 129 and Bessemer Street) was selected to be the first site for this project. This site was to open shortly after the beginning of the research project, providing a location that could not only have *after* data, but also *transition* data, collected in January 2011 for the *transition* period and in June 2011 for the *after* period, thereby serving as a good first site. This site is unique in that it is an atypical, skewed, outspread configuration, as well as being an underpass, which will provide for some variation between the various DCD locations. Refer to Figure 4 for an illustration of the current DCD interchange at Alcoa, TN.



Figure 4: US Route 129 and Bessemer Street, Alcoa, TN (Source: Google Maps)

3.2.2 I-44 and MO-13, Springfield, MO DCD Interchange

The second DCD to be studied is the I-44 and MO-13 interchange in Springfield, MO. Data collection for this site was in March 2011 and the site will not provide *before* or *transition* data as it had already been open for over a year at the time of the data collection effort. There had been ample adjustment time for the local traffic at the time of the data collection. The MO-13 DCD is a “traditional” DCD in that it is an overpass using the existing bridge, while also being very compact. Refer to Figure 5 for an illustration of the current DCD interchange at MO-13 in Springfield, MO.



Figure 5: I-44 and MO-13, Springfield, MO (Source: Google Maps)

3.2.3 US Route 60 and National Avenue - Springfield, MO

Data collection for the National Avenue site in Springfield, MO began March 2011 and, similar to the nearby MO-13 DCD site, *before* and *transition* data could not be collected as it was open prior to the beginning of the project. The National Avenue site is the second overpass to be analyzed and also used the existing bridge. Refer to Figure 6 for an illustration of the current DCD interchange at National Avenue in Springfield, MO.



Figure 6: US-60 and National Avenue - Springfield, MO (Source: Google Maps)

3.2.4 I-270 and Dorsett Road - Maryland Heights, MO

Data collection at this site took place in August 2011. This site was an underpass and, as was the case with the two Missouri sites, *before* and *transition* data was not collected. This site is more traditional than the Alcoa site since it is much more compact. This variation in the two underpasses is likely important for analysis purposes. Refer to Figure 7 for an illustration of the current DCD interchange at Dorsett Road in Maryland Heights, MO.



Figure 7: I-270 and Dorsett Road - Maryland Heights, MO (Source: Google Maps)

3.3 Data Collection

Field data collected for this thesis include 15-minute traffic volumes, saturation flow rates, peak period queues, erratic maneuvers, field-observed conflicts, and origin-destination travel times. Fifteen minute traffic volumes and saturation flow rates were

utilized as Synchro/SimTraffic inputs for simulation of *after* conditions for the MO-13 DCD site.

Peak period queues, origin-destination travel times, erratic maneuvers, and field-observed conflicts were collected to perform the *transition* comparison to *after* conditions at the Alcoa, TN DCD; and to evaluate operational performance of *after* conditions at the MO-13 DCD, National Avenue DCD, and Dorsett Road DCD.

3.3.1 Fifteen Minute Traffic Volumes

Traffic volumes at 15-minute intervals were collected from video recordings of the DCD interchanges under study. The video recordings were collected at the time of each of the site visits, during the AM peak period (7:00 am to 9:00 am), and during the PM peak period (4:30 pm to 6:30 pm). I installed two cameras facing each other at each of the DCD crossovers to capture all six movements at a DCD interchange ramps and crossovers. These movements include the through movements at each direction in the crossover, the off-ramp right and left turns, and the on-ramp right and left turns, as shown in Figure 1 for each side of the DCD. The videos were later processed in the office and the peak period traffic volumes were obtained by direct observations of the videos using a time-stamp macro in Microsoft Excel 2007.

3.3.2 Saturation Flow Rates

Saturation flow rates were obtained by direct observation in the field utilizing the methodology described in the ITE Manual of Transportation Engineering Studies (15). This methodology requires the use of a stopwatch to record the elapsed interval in seconds since the rear axle of the fourth vehicle discharging from a queue crosses the stop bar of an approach; until the rear axle of the seventh, eighth, ninth, or tenth vehicle (whichever vehicle was the latest in the queue) crosses the same stop bar. The time was written down in a tally sheet and the value of saturation flow rate in vehicles per hour was later calculated using the methodology from the ITE Manual of Transportation

Engineering Studies. Refer to Table 1 for illustration of saturation flow rates collected in the field for all four DCD sites.

Table 1: Saturation Flow Rates

DCD Site	Scenario	Crossover	Direction	Lane	Sat. Flow (vph)	Number of Cycles Observed	Number of Vehicles Observed
Alcoa	After	East	WB	Both	1773	29	121
			EB	Both	1718	15	57
		West	EB	Both	1758	23	115
			WB	Both	1713	4	12
MO-13	After	South	NB	Left	1662	38	228
				Right	1668	35	194
		North	SB	Left	1400	20	103
				Right	1575	22	103
National Avenue	After	South	NB	Left	1813	36	208
				Center	1833	19	83
				Right	1874	5	26
		North	SB	Left	1817	25	140
				Center	1762	22	111
				Right	1625	8	35
Dorsett Road	After	East	WB	Both	1851	31	169
			EB	Both	1676	21	85
		West	EB	Both	1823	46	203
			WB	Both	1833	28	116

3.3.3 Peak Period Queues

I measured the maximum back of the queue length on a per-lane and per-cycle basis, at each of the arterial through movements. These measurements give a sense of the congestion levels at the DCD, have implications for spillback potential to merge and diverge points, and are validation items for traffic simulation analysis. Peak period queues were recorded utilizing tally sheets during the same intervals as the 15 minute traffic volumes; the AM peak period (7:00 am to 9:00 am), and the PM peak period (4:30 pm to 6:30 pm). The value of maximum back of the queue was converted to feet by

multiplying the number of vehicles at the back of the queue by the vehicle spacing at rest (L_j), which accounts for percent of heavy traffic and is calculated as follows:

$$L_j = \left(1 - \frac{\%_{hv}}{100}\right) L_c + \frac{\%_{hv}}{100} L_{hv} ; \text{ where,}$$

L_c : Car spacing at rest (ft)

L_{hv} : Heavy vehicle spacing at rest (ft)

$\%_{hv}$: Percentage of heavy traffic

3.3.4 *Origin-Destination Travel Times*

Origin-destination travel times were collected using in-vehicle GPS units for the through movement on the minor roadway, as well as all left turn movements through the intersection. These movements were considered as the most important for analysis of the operational performance of a DCD. No through travel times were collected for the freeway, since those movements were free-flowing and not directly affected by the DCD. GPS data was downloaded for all movements and processed into “routes” using the TravTime 2.0 software package.

Travel times were collected always during the AM peak period, and the PM peak period, and at some DCD sites on the AM off-peak, the noon peak, and the PM off-peak. However, only travel times collected during the AM peak period and the PM peak period were utilized in this study because they are the most relevant to planning and design decisions. Travel times were collected directly from the GPS unit and software; other MOEs are derived from the travel times such as average number of stops, and average control delay. In TravTime, a vehicle is considered to come to a stop when its speed is less than 5 mph. Control delay in TravTime is defined as the time from when the vehicle is below the stopped speed within the delay influence distance (500 ft) of a checkpoint to when the vehicle achieves freeflow speed after passing the checkpoint.

3.3.5 *Erratic Maneuvers and Field-Observed Conflicts*

I conceptually divided the DCD interchange into eight potential conflict zones within one side of the DCD interchange (the other side is equivalent) as shown in Figure 8. Of these conflict zones, areas 1 through 4 were presumed to be the most critical in evaluating the safety performance of the interchange and they were therefore the focus of the field conflict study. Areas 1 and 2 correspond to the two approaches to the DCD crossover point, which were monitored for various conflict types including rear-end conflicts, wrong-way maneuvers, illegal u-turns, red-light-running, and lane-changing conflicts. Area 3 corresponds to the left turn maneuver from the freeway. Similarly, area 4 corresponds to right turn maneuvers from the freeway. It was hypothesized that diverge conflicts in areas 5 through 8 do not require as detailed of a study, since these conflicts are largely consistent with what would be expected at conventional diamond interchanges.

The conflict data were gathered using manual tally sheets by members of the research team and binned in 15-minute intervals. Team members were trained to identify and differentiate between conflicts and erratic maneuvers; however, the conflict study requires some degree of engineering judgment by the observer, and therefore some caution should be taken when interpreting conflict data results.

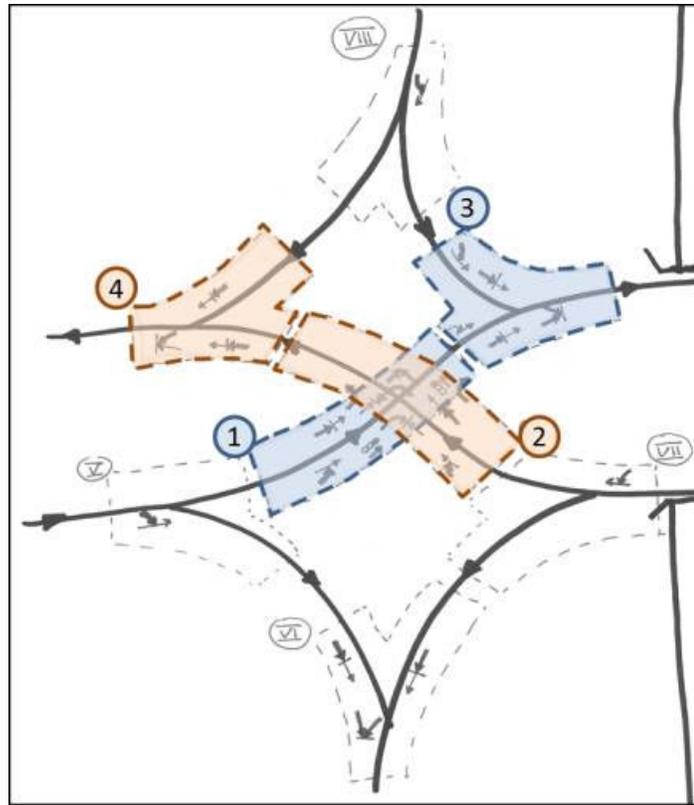


Figure 8: DCD Conflict Locations

3.4 Data Analysis

The US Route 129 and Bessemer Street DCD site in Alcoa, TN was the only site where a direct comparison of field data was possible. For this site only, a *transition to after* data analysis was performed. The *transition to after* data analysis included origin-destination travel times for all through and left turn movements, average stops per vehicle, average delay per vehicle, level of service, average peak period queues, and a comparison of conflicts and erratic maneuvers.

For the remaining sites at I-44 and MO-13 in Springfield, MO, US Route 60 and National Avenue in Springfield, MO, and I-270 and Dorsett Road in Maryland Heights, MO, an evaluation of *after* traffic conditions was performed. The MOEs for the *after* analysis were origin-destination travel times for all through and left turn movements, average

stops per vehicle, average delay per vehicle, level of service, average peak period queues, and a comparison of conflicts and erratic maneuvers.

Also, a calibration and validation of *after* conditions using the Synchro/SimTraffic simulation model for the MO-13 Springfield DCD site was performed. Synchro/SimTraffic 7.0 traffic analysis software package was chosen to simulate *after* conditions. Synchro was used to input the lane geometry and to optimize signal timings. To obtain a more realistic traffic simulation model, the Synchro/SimTraffic model was calibrated to the *after* data collected in the field. Since the 15-minute traffic volumes were only collected at the crossover intersections of the DCDs under study, traffic volumes at adjacent intersections were imported from previous studies whenever possible. The turning movement counts for adjacent intersections at I-44 and MO-13 in Springfield, MO, were obtained from the MoDOT/HDR report from February 2011 (1). After the Synchro model was built, the associated SimTraffic simulation software was used to calculate the travel time, average delay per vehicle, and level of service.

4.0 OPERATIONAL RESULTS

This section presents the operational results of the *transition* to *after* comparison of the Alcoa, TN DCD site, as well as the *after* evaluation of the MO-13 DCD site in Springfield, MO, the National Avenue DCD site in Springfield, MO and the Dorsett Road DCD site in Maryland Heights, MO. Also, a SimTraffic model of the MO-13 DCD site, calibrated with field data, is presented to evaluate how simulation can replicate field conditions.

To perform the origin-destination travel time analysis, the arterial at each DCD site was divided into four segments for the through movements, and into three segments for the left turn movements. From the raw GPS data collected in the field, the TravTime 2.0 software was utilized to obtain origin-destination travel time, average control delay and average stops per segment. Highway Capacity Manual 2010, Chapter 22 *Interchange Ramp Terminals* (16) was used to obtain level of service (LOS) for both the through and left turn movements. Note that while the LOS is given for the total route, the calculation only took into account the average delay at each of the two crossovers, not the entire route, as described in the Highway Capacity Manual 2010 methodology. For each site, the travel time runs collected in the field were processed into AM peak period travel times (6:30 am to 9:30 am), and PM peak period travel times (3:30 pm to 6:30 pm), to maximize the total number of runs available for analysis.

The sample size for the origin-destination travel time runs for all sites was constrained by the available number of drivers and number of days when field data was collected. In all sites, except Alcoa, TN, two drivers were assigned opposite routes and travel times were collected for the entire duration of the field data collection effort. Field data collection occurred from Monday afternoon to Wednesday morning at each site. Therefore, two AM peak periods and two PM peak periods were available for data collection. In the Alcoa, TN site, only one driver was available; however, since traffic volumes were low when compared to other sites, a similar number of runs were collected.

The peak period queue analysis was achieved by comparing the average queue per lane in terms of vehicles at each approach, both in the *transition* to *after* and in the *before* to *after* analysis. The peak period queues were expressed as mean, minimum and maximum queues. Values for standard deviation and number of observations were also included. For each site, the peak period queues collected in the field were divided into AM peak period queues (7:00 am to 9:00 am), and PM peak period queues (4:30 pm to 6:30 pm). These two periods were the only time when peak hour queues were collected.

4.1 US Route 129 and Bessemer Street DCD, Alcoa TN Analysis

As previously stated, the US Route 129 and Bessemer Street DCD interchange in Alcoa, TN, was the only site where *transition* data was available. Therefore, this is the only DCD site to be analyzed from a *transition* to *after* perspective. Field data will be analyzed for both scenarios and compared to each other. Also, this will be the only site where erratic maneuvers and field observed conflicts will be compared for each scenario. Refer to Appendix A for the field data collected for Alcoa DCD.

4.1.1 Analysis of Average Travel Time, Average Control Delay and Average Number of Stops for Alcoa, TN DCD

Figures 9 and 10 illustrate the AM and PM peak hour traffic volume counts for the *transition* and *after* scenarios, respectively. Analysis of the traffic counts indicates that the annual growth rate was approximately 10% for the AM peak hour and the PM peak hour combined. While the annual growth rate seems excessive and this was likely due to the small sample size (one day of data for both the *transition* and *after* scenarios), it was expected that traffic will increase during the *after* period, since data collection occurred during the summer months; while *transition* data collection occurred during winter.

Also, no change in lane geometry and signal timing was observed from the *transition* to *after* period. Weather during the data collection effort was fair and traffic volumes were not affected by weather conditions.

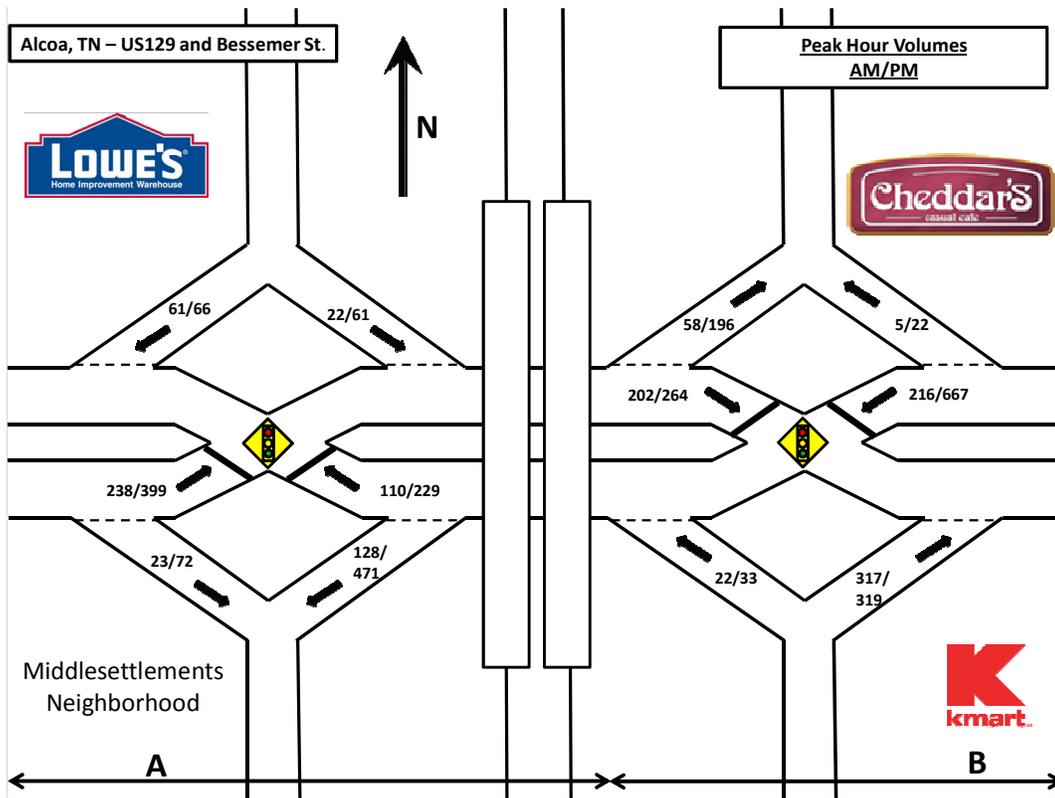


Figure 9: Alcoa Transition Traffic Volumes

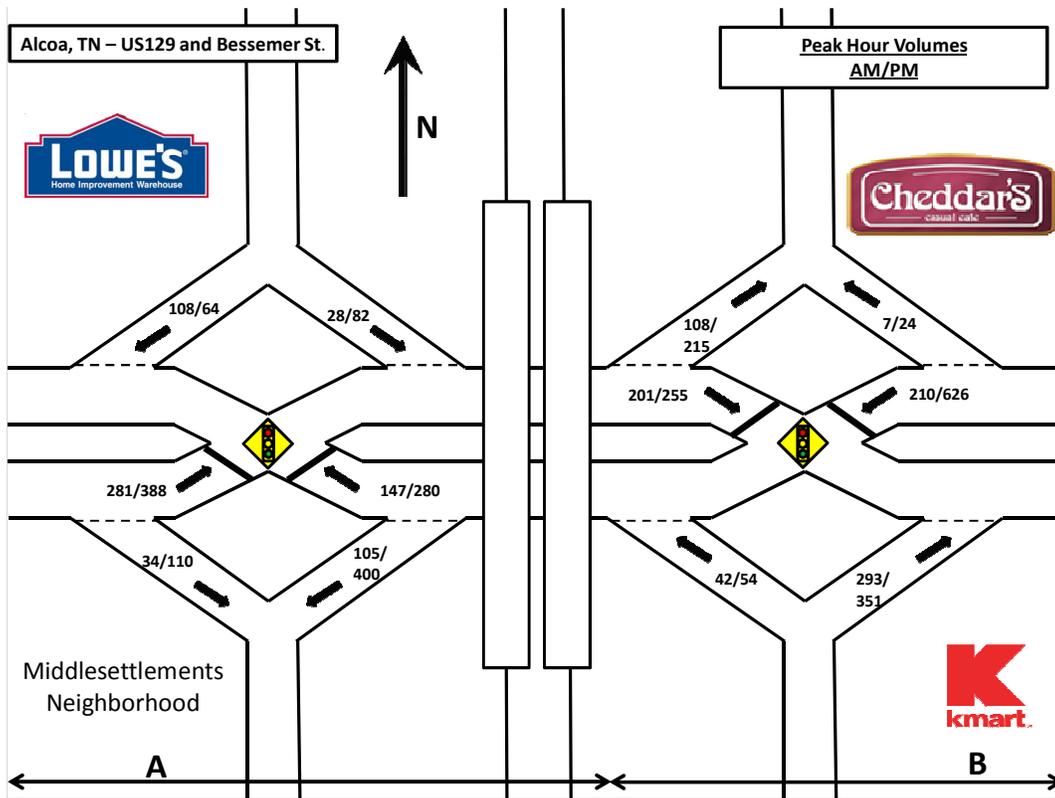


Figure 10: Alcoa After Traffic Volumes

Table 2 and 3 summarize the origin-destination travel times (in minutes), average control delay (in seconds), level of service (LOS), and the average number of stops for the Alcoa, TN DCD site, for the AM peak period and for the PM peak period respectively. A negative value at the right hand side of the tables indicates that the *after* period had a lower MOE than the *transition* period. Further, each of the six routes is subdivided in four segments for the arterial through movements (east-to-west, west-to-east), and in three segments for all left turn movements at the DCD (east-to-south, south-to-west, west-to-north, north-to-east).

During the AM peak period (6:30 am to 9:30 am), the after scenario showed a decrease in average travel time and average number of stops for all segments, except for the south-to-west and the north-to-east routes. The south-to-west average travel time increased by approximately 8%; however, the average control delay for the same segment decreased by 30%. On the contrary, while the east-to-south route showed a decrease in average travel time and average number of stops, the average control delay increased by 11%. The north-to-east route showed an increase in all three MOEs. The north-east route in the AM peak period was the only one that had a statistically significant change, at the 95 percent level, in travel time between the transition and after periods according to t-tests as shown in Table 2. A closer look at the north-to-east route shows that the east crossover to Calderwood Street segment experienced the largest increase in each of the MOEs under study.

During the PM peak period (3:30 pm to 6:30 pm), the after scenario showed a decrease in average travel time and average number of stops for all routes. The average control delay also decreased at all routes, except for the south-to-west and west-to-north routes. A closer look at the west-to-north route, shows that the Wooddale Street to the west crossover segment experienced the largest increase in average control delay.

Analysis of the level of service (LOS) indicates that all segments had a LOS D or better during both peak hours. The arterial through routes experimented LOS C and D, while the left turn routes had LOS C or better during both peak hours under both scenarios.

Table 2: Alcoa DCD Transition to After Analysis AM Peak Period

AM Peak	Transition						After						Avg. Travel Time		Avg. Control Delay		Avg. # Stops		t-test			
	Avg. Travel Time (min)	St. Dev. Travel Time	Avg. Control Delay (s)	Avg. # of Stops	LOS	# Runs	Avg. Travel Time (min)	St. Dev. Travel Time	Avg. Control Delay (s)	Avg. # of Stops	LOS	# Runs	Diff.	% Diff.	Diff.	% Diff.	Diff.	% Diff.	t-dist	d.f.	p-value	
East - West																						
Route Start-Calderwood St. (0.24 mi)	0.98	0.41	48	0.67		6	0.99	0.45	55.2	0.64		14	0.01	1.02%	7.2	15.00%	-0.03	-4.48%				
Calderwood St.-East Crossover (0.31 mi)	0.7	0.17	15.6	0.5		6	0.7	0.17	12	0.43		14	0	0.00%	-3.6	-23.08%	-0.07	-14.00%				
East Crossover-West Crossover (0.13 mi)	0.57	0.27	34.8	0.5		6	0.52	0.29	31.8	0.43		14	-0.05	-8.77%	-3	-8.62%	-0.07	-14.00%				
West Crossover-Wooddale St. (0.17 mi)	0.6	0.22	34.8	0.33		6	0.46	0.16	16.2	0.29		14	-0.14	-23.33%	-18.6	-53.45%	-0.04	-12.12%				
Total East - West (0.85 mi)	2.85	0.48	133.2	2	C	6	2.67	0.59	115.2	1.79	C	14	-0.18	-6.32%	-18	-13.51%	-0.21	-10.50%	0.716	11.687	0.489	
West - East																						
Route Start-Wooddale St. (0.25 mi)	0.55	0.19	16.2	0.33		6	0.37	0.03	0	0		17	-0.18	-32.73%	-16.2	-100.00%	-0.33	-100.00%				
Wooddale St.-West Crossover (0.17 mi)	0.66	0.22	28.8	0.5		6	0.7	0.25	31.8	0.59		17	0.04	6.06%	3	10.42%	0.09	18.00%				
West Crossover-East Crossover (0.13 mi)	0.6	0.28	35.4	0.5		6	0.5	0.24	28.8	0.47		17	-0.1	-16.67%	-6.6	-18.64%	-0.03	-6.00%				
East Crossover-Calderwood St. (0.31 mi)	1.09	0.31	33.6	0.83		6	1.08	0.33	29.4	1		17	-0.01	-0.92%	-4.2	-12.50%	0.17	20.48%				
Total West - East (0.86 mi)	2.9	0.29	114	2.16	D	6	2.65	0.39	90	2.06	D	17	-0.25	-8.62%	-24	-21.05%	-0.1	-4.63%	1.650	11.905	0.127	
East - South																						
Route Start-Calderwood St. (0.24 mi)	0.9	0.43	45	0.67		3	0.66	0.35	51.6	0.33		6	-0.24	-26.67%	6.6	14.67%	-0.34	-50.75%				
Calderwood St.-East Crossover (0.31 mi)	0.8	0.13	16.2	0.67		3	0.94	0.33	16.2	1		6	0.14	17.50%	0	0.00%	0.33	49.25%				
East Crossover-West Crossover (0.1 mi)	0.24	0.02	0	0		3	0.23	0.02	0	0		6	-0.01	-4.17%	0	0.00%	0	0.00%				
Total East - South (0.65 mi)	1.94	0.54	61.2	1.34	B	3	1.83	0.34	67.8	1.33	B	6	-0.11	-5.67%	6.6	10.78%	-0.01	-0.75%	0.322	2.827	0.778	
South - West																						
Route Start-East Crossover (0.1 mi)	0.41	0.19	15.6	0.8		5	0.35	0.14	17.4	0.5		6	-0.06	-14.63%	1.8	11.54%	-0.3	-37.50%				
East Crossover-West Crossover (0.11 mi)	0.38	0.11	12	0.6		5	0.52	0.27	11.4	0.67		6	0.14	36.84%	-0.6	-5.00%	0.07	11.67%				
West Crossover-Wooddale St. (0.17 mi)	0.47	0.08	33	0		5	0.49	0.13	13.2	0.5		6	0.02	4.26%	-19.8	-60.00%	0.5	100.00%				
Total South - West (0.38 mi)	1.26	0.53	60.6	1.4	B	5	1.36	0.2	42	1.67	B	6	0.1	7.94%	-18.6	-30.69%	0.27	19.29%	0.399	4.950	0.710	
West - North																						
Route Start-Wooddale St. (0.25 mi)	0.65	0.22	22.8	0.5		4	0.37	0.01	3	0		5	-0.28	-43.08%	-19.8	-86.84%	-0.5	-100.00%				
Wooddale St.-West Crossover (0.17 mi)	0.61	0.11	13.2	0.75		4	0.7	0.21	22.8	0.6		5	0.09	14.75%	9.6	72.73%	-0.15	-20.00%				
West Crossover-East Crossover (0.11 mi)	0.28	0.01	0	0		4	0.32	0.03	0	0		5	0.04	14.29%	0	0.00%	0	0.00%				
Total West - North (0.53 mi)	1.54	1.66	36	1.25	A	4	1.39	0.23	25.8	0.6	B	5	-0.15	-9.74%	-10.2	-28.33%	-0.65	-52.00%	0.179	3.092	0.869	
North - East																						
Route Start-West Crossover (0.1 mi)	0.27	0.01	8.4	0		3	0.31	0.07	10.2	0.4		5	0.04	14.81%	1.8	21.43%	0.4	100.00%				
West Crossover-East Crossover (0.09 mi)	0.52	0.05	18.6	1		3	0.53	0.22	26.4	0.8		5	0.01	1.92%	7.8	41.94%	-0.2	-20.00%				
East Crossover-Calderwood St. (0.31 mi)	0.6	0.04	7.8	0		3	1.11	0.32	34.8	0.6		5	0.51	85.00%	27	346.15%	0.6	100.00%				
Total North - East (0.5 mi)	1.39	0.06	34.8	1	B	3	1.95	0.24	71.4	1.8	C	5	0.56	40.29%	36.6	105.17%	0.8	80.00%	4.965	4.773	0.008	

Table 3: Alcoa DCD Transition to After Analysis PM Peak Period

PM Peak	Transition						After						Avg. Travel Time		Avg. Control Delay		Avg. # Stops		t-test			
	Avg. Travel Time (min)	St. Dev. Travel Time	Avg. Control Delay (s)	Avg. # of Stops	LOS	# Runs	Avg. Travel Time (min)	St. Dev. Travel Time	Avg. Control Delay (s)	Avg. # of Stops	LOS	# Runs	Diff.	% Diff.	Diff.	% Diff.	Diff.	% Diff.	t-dist	d.f.	p-value	
East - West																						
Route Start-Calderwood St. (0.24 mi)	1.28	0.30	50.6	1		6	0.89	0.57	46.2	0.5		6	-0.39	-30.47%	-4.4	-8.70%	-0.5	-50.00%				
Calderwood St.-East Crossover (0.31 mi)	0.76	0.29	14.7	0.50		6	1.17	0.56	24.6	1.17		6	0.407	53.28%	9.9	67.35%	0.67	134.00%				
East Crossover-West Crossover (0.13 mi)	0.70	0.30	31.8	0.67		6	0.56	0.32	28.2	0.5		6	-0.14	-20.38%	-3.6	-11.32%	-0.17	-25.00%				
West Crossover-Wooddale St. (0.17 mi)	0.80	0.28	34.32	0.83		6	0.77	0.71	19.8	0.5		6	-0.03	-3.35%	-14.5	-42.31%	-0.33	-40.00%				
Total East - West (0.85 mi)	3.54	1.04	131.42	3	C	6	3.39	1.4	118.8	2.67	C	6	-0.15	-4.33%	-12.6	-9.60%	-0.33	-11.00%	0.215	9.230	0.834	
West - East																						
Route Start-Wooddale St. (0.25 mi)	0.69	0.31	24	0.62		8	0.43	0.11	10.2	0.29		7	-0.26	-37.68%	-13.8	-57.50%	-0.33	-53.23%				
Wooddale St.-West Crossover (0.17 mi)	0.78	0.19	25.8	0.75		8	0.61	0.19	20.4	0.43		7	-0.17	-21.79%	-5.4	-20.93%	-0.32	-42.67%				
West Crossover-East Crossover (0.13 mi)	0.68	0.29	36.6	0.62		8	0.51	0.25	28.2	0.43		7	-0.17	-25.00%	-8.4	-22.95%	-0.19	-30.65%				
East Crossover-Calderwood St. (0.31 mi)	1.07	0.46	39.6	0.75		8	1.26	0.36	39	1		7	0.19	17.76%	-0.6	-1.52%	0.25	33.33%				
Total West - East (0.86 mi)	3.22	0.63	126	2.74	D	8	2.81	0.34	97.8	2.15	C	7	-0.41	-12.73%	-28.2	-22.38%	-0.59	-21.53%	1.594	11.012	0.139	
East - South																						
Route Start-Calderwood St. (0.24 mi)	1.4	0.08	58.8	1		3	1.21	0.34	50.4	1		4	-0.19	-13.57%	-8.4	-14.29%	0	0.00%				
Calderwood St.-East Crossover (0.31 mi)	0.99	0.15	26.4	1		3	0.97	0.28	25.8	0.75		4	-0.02	-2.02%	-0.6	-2.27%	-0.25	-25.00%				
East Crossover-West Crossover (0.1 mi)	0.27	0.05	0	0		3	0.22	0	0	0		4	-0.05	-18.52%	0	0.00%	0	0.00%				
Total East - South (0.65 mi)	2.66	0.16	85.2	2	B	3	2.4	0.39	76.2	1.75	B	4	-0.26	-9.77%	-9	-10.56%	-0.25	-12.50%	1.205	4.182	0.295	
South - West																						
Route Start-East Crossover (0.1 mi)	0.67	0.24	30.6	0.75		4	0.53	0.33	22.2	0.67		3	-0.14	-20.90%	-8.4	-27.45%	-0.08	-10.67%				
East Crossover-West Crossover (0.11 mi)	0.42	0.1	13.2	0.75		4	0.51	0.05	16.8	1		3	0.09	21.43%	3.6	27.27%	0.25	33.33%				
West Crossover-Wooddale St. (0.17 mi)	0.6	0.18	25.2	0.25		4	0.42	0.03	37.2	0		3	-0.18	-30.00%	12	47.62%	-0.25	-100.00%				
Total South - West (0.38 mi)	1.69	0.45	69	1.75	C	4	1.46	0.27	76.2	1.67	C	3	-0.23	-13.61%	7.2	10.43%	-0.08	-4.57%	0.840	4.883	0.448	
West - North																						
Route Start-Wooddale St. (0.25 mi)	0.96	0.64	15	1.25		4	0.52	0.2	14.4	0.25		4	-0.44	-45.83%	-0.6	-4.00%	-1	-80.00%				
Wooddale St.-West Crossover (0.17 mi)	0.55	0.11	12.6	0		4	0.69	0.37	36	0.5		4	0.14	25.45%	23.4	185.71%	0.5	100.00%				
West Crossover-East Crossover (0.11 mi)	0.3	0.08	0	0		4	0.27	0.05	4.8	0		4	-0.03	-10.00%	4.8	100.00%	0	0.00%				
Total West - North (0.53 mi)	1.81	0.79	27.6	1.25	A	4	1.48	0.61	55.2	0.75	C	4	-0.33	-18.23%	27.6	100.00%	-0.5	-40.00%	0.661	5.639	0.538	
North - East																						
Route Start-West Crossover (0.1 mi)	0.3	0.04	10.8	0.67		3	0.32	0.07	10.8	0.5		4	0.02	6.67%	0	0.00%	-0.17	-25.37%				
West Crossover-East Crossover (0.09 mi)	0.44	0.14	16.8	1		3	0.42	0.28	20.4	0.5		4	-0.02	-4.55%	3.6	21.43%	-0.5	-50.00%				
East Crossover-Calderwood St. (0.31 mi)	1.25	0.25	40.8	1		3	0.88	0.45	36.6	0.5		4	-0.37	-29.60%	-4.2	-10.29%	-0.5	-50.00%				
Total North - East (0.5 mi)	1.99	0.17	68.4	2.67	B	3	1.62	0.59	67.8	1.5	C	4	-0.37	-18.59%	-0.6	-0.88%	-1.17	-43.82%	1.190	3.634	0.320	

While the Alcoa DCD analysis was focused on the *transition* to *after* scenarios, another way to illustrate the operational impacts of the DCD is to show an illustration of how vehicles progress through the interchange. Let's consider the case of one of the arterial through routes, the east-to-west route, and one left turn route, the west-to-north route. Figures 11 and 12 show the time-space diagram and average number of stops, respectively, for the east-to-west route during *after* conditions during the PM peak period. Figures 13 and 14 show the time-space diagram and average number of stops, respectively, for the west-to-north route during *after* conditions during the PM peak period.

In the case of the east-to-west route, the time-space diagram shows that in five of six runs, the vehicle was stopped at the east crossover, and later progressed through the west crossover in three of the five runs. This can also be seen in Figure 12, which shows that the east crossover had a larger number of stops than the previous intersection, but the number of stops decreased in the two downstream intersections. In the case of the west-to-north route, the time-space diagram shows that while it is likely that a vehicle will stop at the west crossover, it will later progress without stopping at the left-turn on-ramp before the east crossover. This is expected, since one of the features of a DCD interchange is the free flow left turn on-ramp. This can also be seen in Figure 14, where the left turn on-ramp has no stops.

Appendix B shows time-space diagrams and average number of stops figures for all routes, during both peak periods, for *transition* and *after* scenarios.

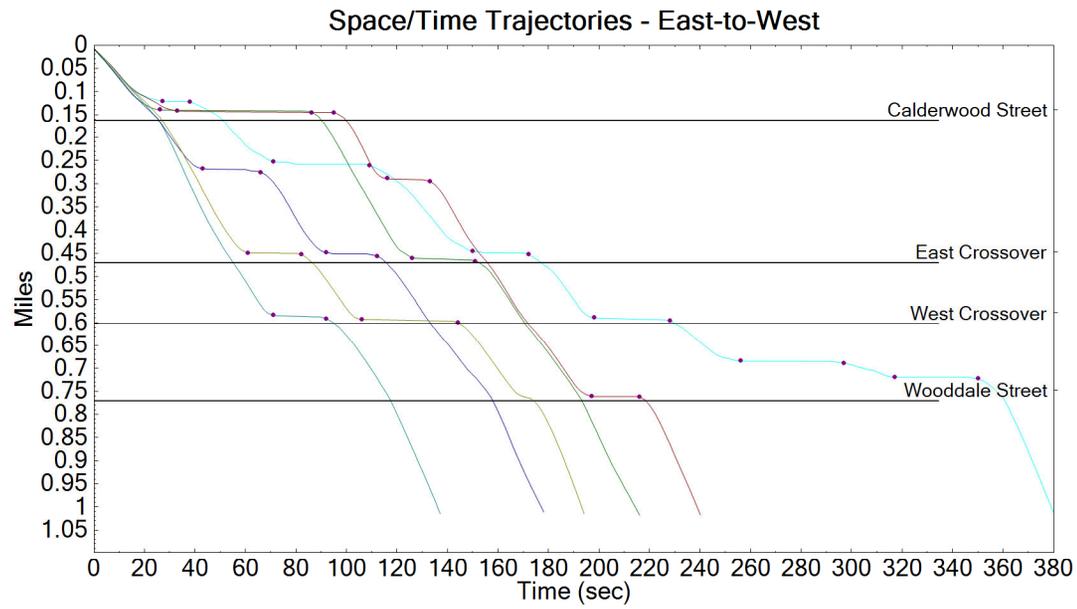


Figure 11: Time-Space Diagram for East-to-West Route – *After* Conditions – PM Peak Period

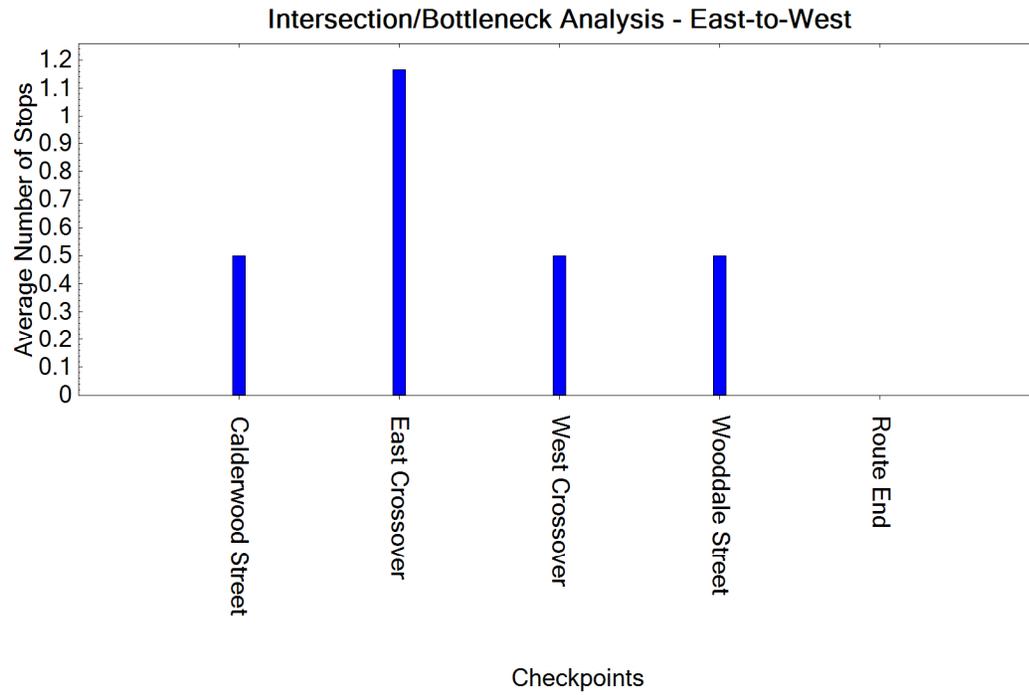


Figure 12: Average Number of Stops East-to-West Route – *After* Conditions – PM Peak Period

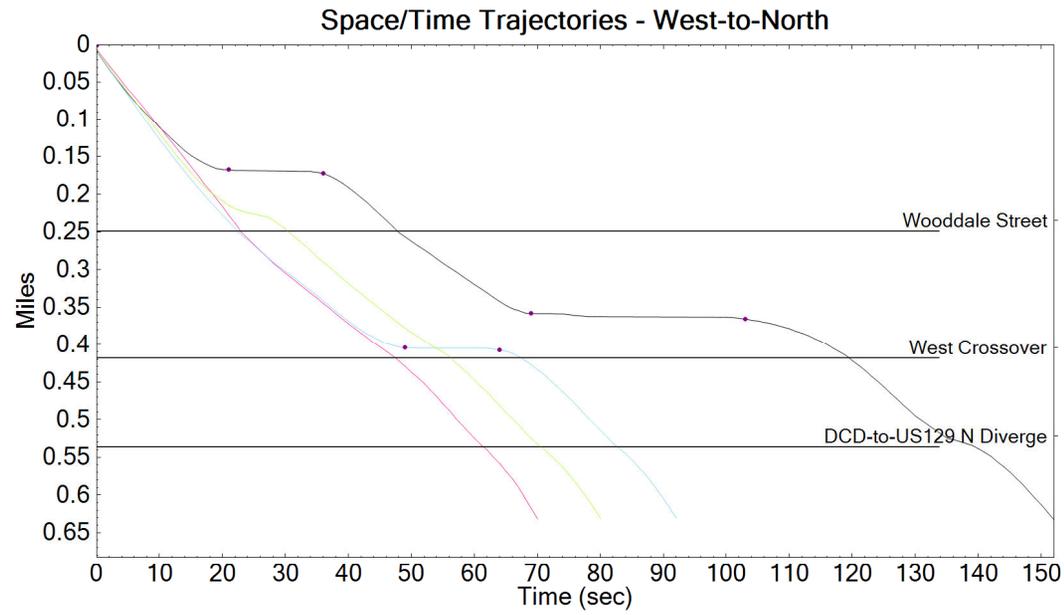


Figure 13: Time-Space Diagram for West-to-North Route – *After* Conditions – PM Peak Period

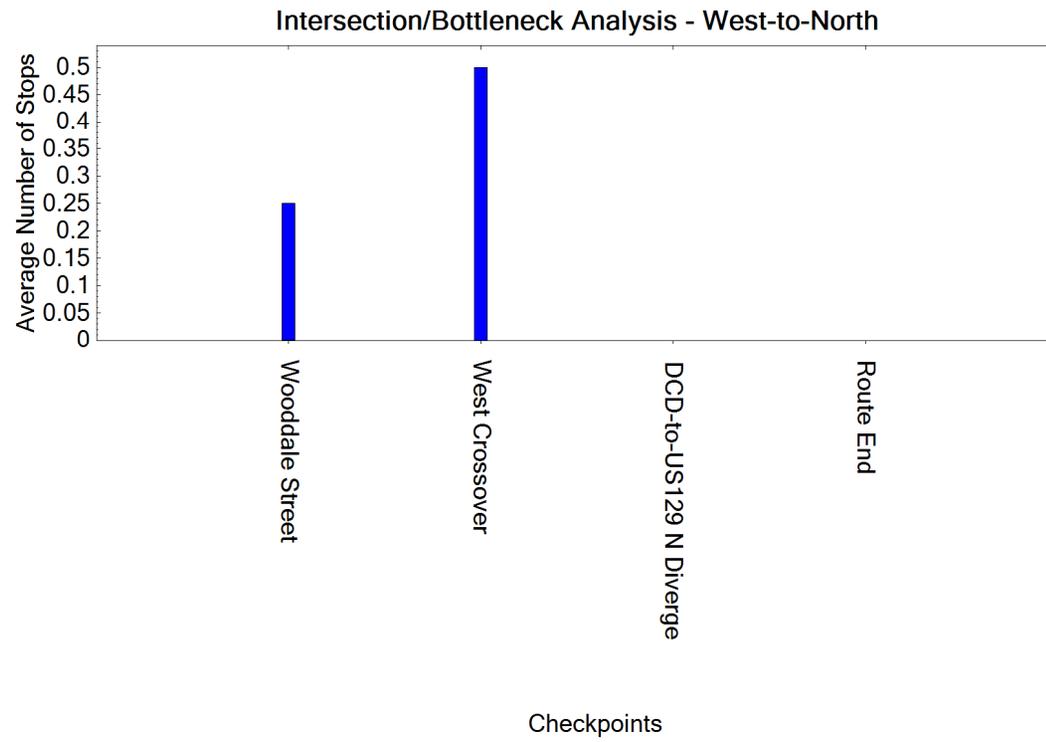


Figure 14: Average Number of Stops West-to-North Route – *After* Conditions – PM Peak Period

4.1.2 Analysis of Peak Period Queues for Alcoa, TN DCD

Table 4 and Table 5 provide a summary of the AM and PM peak period queues for the west crossover and east crossover, respectively, under *transition* and *after* conditions. The west crossover data in Table 4 show that there was an increase in average queue size at each of the approaches during the AM peak period. A t-test showed that the difference in westbound mean queue length between the *transition* and *after* periods was significant at the 95 percent level. However, the queues were quite small in almost all cases, with the highest mean being only 2.3 vehicles, so the practical effect of any difference is slight.

In the case of the east crossover, average queues decreased from *transition* to *after* period in all cases. It is important to note that in the east crossover, the westbound direction had two lanes, and their average, minimum, 95th percentile, and maximum queue values from both lanes were added together. Nonetheless, the significant difference of an average of almost three vehicles less queued in the after period as compared to the *transition* period westbound in the PM peak period may be noticeable to motorists.

Table 4: Alcoa DCD West Crossover Peak Period Queues

West Crossover	Transition		After		Diff.	% Diff.	Diff.	% Diff.	t-test EB Queues			t-test WB Queues			
	EB Center Lane Queues (veh)	WB Center Lane Queues (veh)	EB Center Lane Queues (veh)	WB Center Lane Queues (veh)					EB Center Lane Queues	WB Center Lane Queues	t-dist	d.f.	p-value	t-dist	d.f.
AM Peak Period															
Mean	1.9	0.7	2.3	1	0.4	21.05%	0.3	42.86%	1.823	257.854	0.070	2.141	194.173	0.033	
Min	0	0	0	0	0	0.00%	0	0.00%							
95th Per.	7	4	6	4	-1	-14.29%	0	0.00%							
Max	10	5	11	5	1	10.00%	0	0.00%							
Std.Dev.	1.94	0.97	1.8	1.3	-0.14	-7.22%	0.33	34.02%							
n	186	186	116	116	N/A	N/A	N/A	N/A							
PM Peak Period															
Mean	3.6	2.2	3.3	2.2	-0.3	-8.33%	0	0.00%	0.788	159.366	0.432	0.000	174.044	1.000	
Min	0	0	0	0	0	0.00%	0	0.00%							
95th Per.	8	6	10	6	2	25.00%	0	0.00%							
Max	15	12	11	7	-4	-26.67%	-5	-41.67%							
Std.Dev.	2.87	1.96	2.9	1.8	0.03	1.05%	-0.16	-8.16%							
n	184	184	84	84	N/A	N/A	N/A	N/A							

Table 5: Alcoa DCD East Crossover Peak Period Queues

East Crossover	Transition		After		Diff.	% Diff.	Diff.	% Diff.	t-test EB Queues			t-test WB Queues		
	EB Center Lane Queues (veh)	WB All Lanes Queues (veh)	EB Center Lane Queues (veh)	WB All Lanes Queues (veh)	EB Center Lane Queues	WB All Lanes Queues (veh)	t-dist	d.f.	p-value	t-dist	d.f.	p-value		
AM Peak Period	1.4	2.5	1.3	2.1	-0.1	-7.14%	-0.4	-16.00%	0.491	208.911	0.624	1.075	196.402	0.284
Mean	0	0	0	0	0	0.00%	0	0.00%						
Min	5	7	4	7	-1	-20.00%	0	0.00%						
95th Per.	7	16	9	11	2	28.57%	-5	-31.25%						
Max	1.46	2.96	1.5	2.4	0.04	2.74%	-0.56	-18.92%						
Std.Dev.	103	103	108	108	N/A	N/A	N/A	N/A						
n	EB Center Lane Queues (veh)	WB All Lanes Queues (veh)	EB Center Lane Queues (veh)	WB All Lanes Queues (veh)	EB Center Lane Queues	WB All Lanes Queues (veh)	t-dist	d.f.	p-value	t-dist	d.f.	p-value		
PM Peak Period	2.6	9.1	2.3	6.3	-0.3	-11.54%	-2.8	-30.77%	1.127	183.212	0.261	3.746	153.315	0.000
Mean	0	0	0	0	0	0.00%	0	0.00%						
Min	6	20	7	14	1	16.67%	-6	-30.00%						
95th Per.	9	26	8	19	-1	-11.11%	-7	-26.92%						
Max	1.82	5.67	1.9	4.5	0.08	4.40%	-1.17	-20.63%						
Std.Dev.	84	84	115	115	N/A	N/A	N/A	N/A						
n														

4.1.3 Alcoa, TN DCD Conflict and Erratic Maneuver Analysis

This section presents the preliminary results of the conflict and erratic maneuver analysis of the Alcoa DCD interchange. A complete safety analysis of the Alcoa DCD and all other sites will be available in the final report. Since the Alcoa DCD site was the only site in this report that a *transition* and *after* conflict and erratic maneuver completed, it is included here. The results of the *transition* and *after* conflict and erratic maneuver data collected in the field are illustrated in Table 6.

Analysis of the conflict and erratic maneuver data in Table 6 indicates that the total number of conflicts and erratic maneuvers, as well as the corresponding rates per hour decreased significantly from the *transition* to *after* conditions. Almost all types of conflicts and erratic maneuvers also decreased except for the lane change conflict which only increased by one at the west crossover. Also, the reduction in conflicts was likely not related to traffic volumes, since traffic volumes actually increased from *transition* to *after* conditions, as shown in Figures 9 and 10.

During the time of the *transition* site visit, a large number of U-turns at the west crossover were observed. This was likely caused by access management modifications to the residential neighborhood located to the southwest of the interchange, where access to Bessemer Street was restricted as part of the DCD interchange construction project. No left turns out of the residential neighborhood were allowed, only right in and right out maneuvers. Also, the merge point at the west crossover from the southbound off-ramp right turn and the westbound through movement caused several conflicts during the *transition* study period. Sight distance issues were observed at the southbound off-ramp left turn, which created merge conflicts with the eastbound through movement at the west crossover.

By the time of the *after* site visit, the U-Turn erratic maneuvers and merge conflicts had decreased considerably. While the access management conditions that created U-Turns at the west crossover remained, it was observed that, overall, drivers navigated through

the DCD interchange much more safely, and some of the conflicts that could be attributed to unfamiliarity of the DCD interchange (random stops, signal compliance) decreased significantly.

Table 6: Conflict and Erratic Maneuver Summary for Alcoa DCD

Summary of Data	Transition		After		East Crossover		West Crossover	
	East	West	East	West	Diff.	% Diff	Diff.	% Diff
	Crossover	Crossover	Crossover	Crossover				
Number of Events Total	167	360	29	39	-138	-82.63%	-321	-89.17%
Total Time (hours)	9	14.25	8	8	N/A	N/A	N/A	N/A
Rate Events per hour	18.6	25.3	3.6	4.9	-14.9	-80.46%	-20.4	-80.70%
Conflicts vs. Erratic Maneuvers								
Number of Conflicts	59	94	2	6	-57	-96.61%	-88	-93.62%
Number of Erratic Maneuvers	108	266	27	33	-81	-75.00%	-233	-87.59%
Rate Conflicts/Hour	6.6	6.6	0.3	0.8	-6.3	-96.19%	-5.8	-88.63%
Rate EMs/Hour	12.0	18.7	3.4	4.1	-8.6	-71.88%	-14.5	-77.90%
Event Details								
U-Turn	17	95	9	12	-8	-47.06%	-83	-87.37%
Rear-End Conflicts	24	21	0	1	-24	-100.00%	-20	-95.24%
Signal Compliance	45	56	15	9	-30	-66.67%	-47	-83.93%
Stop	47	25	2	8	-45	-95.74%	-17	-68.00%
Merge	15	155	2	4	-13	-86.67%	-151	-97.42%
Lane Change	18	3	1	4	-17	-94.44%	1	33.33%
Backing Up	1	0	0	0	-1	-100.00%	0	0.00%
Left Turn Head on	0	1	0	1	0	0.00%	0	0.00%
Broke Down	0	1	0	0	0	0.00%	-1	-100.00%
Illegal Left	0	2	0	0	0	0.00%	-2	-100.00%
Wrong Way	0	1	0	0	0	0.00%	-1	-100.00%

4.1.4 Summary of the Alcoa, TN DCD Analysis

In summary, the *transition* to *after* analysis of average travel time, average control delay and average number of stops for Alcoa, TN DCD, showed that in most of the routes and peak periods under study, traffic operation performance improved after six months of the opening of the DCD interchange. Further, level of service indicated that for both scenarios, the interchange operated at LOS D or LOS C for the arterial through movements, and at LOS C or better for the left turn movements. The peak period queues also showed an improvement, although less significant than the other MOEs. In terms of

conflicts and erratic maneuvers, there was a significant decrease in total conflicts per hour and erratic maneuvers between *transition* and *after* conditions.

4.2 I-44 and MO-13 DCD Analysis

Data collection at the I-44 and MO-13 DCD in Springfield, MO, only occurred for *after* conditions. Therefore, the contents of this section present an evaluation of the origin-destination travel times, average control delay, average number of stops, level of service, peak period queues, and conflicts and erratic maneuvers for the MO-13 DCD interchange for *after* conditions only. Refer to Appendix C for the field data collected for MO-13 DCD.

Calculation of origin-destination travel times, average control delay and average number of stops was obtained from TravTime software. Level of service was calculated from the methodology presented in Chapter 22 of the Highway Capacity Manual 2010 (HCM 2010). Note that while the LOS is given for the total route, the calculation only took into account the average delay at each of the two crossovers, not the entire route, as described in the HCM 2010 methodology. Peak period queues and conflict data was obtained by direct observations in the field. Caution should be exercised with the analysis of the left turns during the AM and PM peak periods due to small sample sizes.

4.2.1 Analysis of Average Travel Time, Average Control Delay, Average Number of Stops, and Level of Service for MO-13 DCD

Figure 15 illustrates the AM and PM peak hour traffic volume counts for the *after* scenario at MO-13. Table 7 summarizes the origin-destination travel times (in minutes), average control delay (in seconds), the average number of stops, and level of service for the MO-13 DCD site, for the AM peak period and for the PM peak period. MOEs for six routes are presented, south-to-north, north-to-south, south-to-west, west-to-north, north-to-east, and east-to-south.

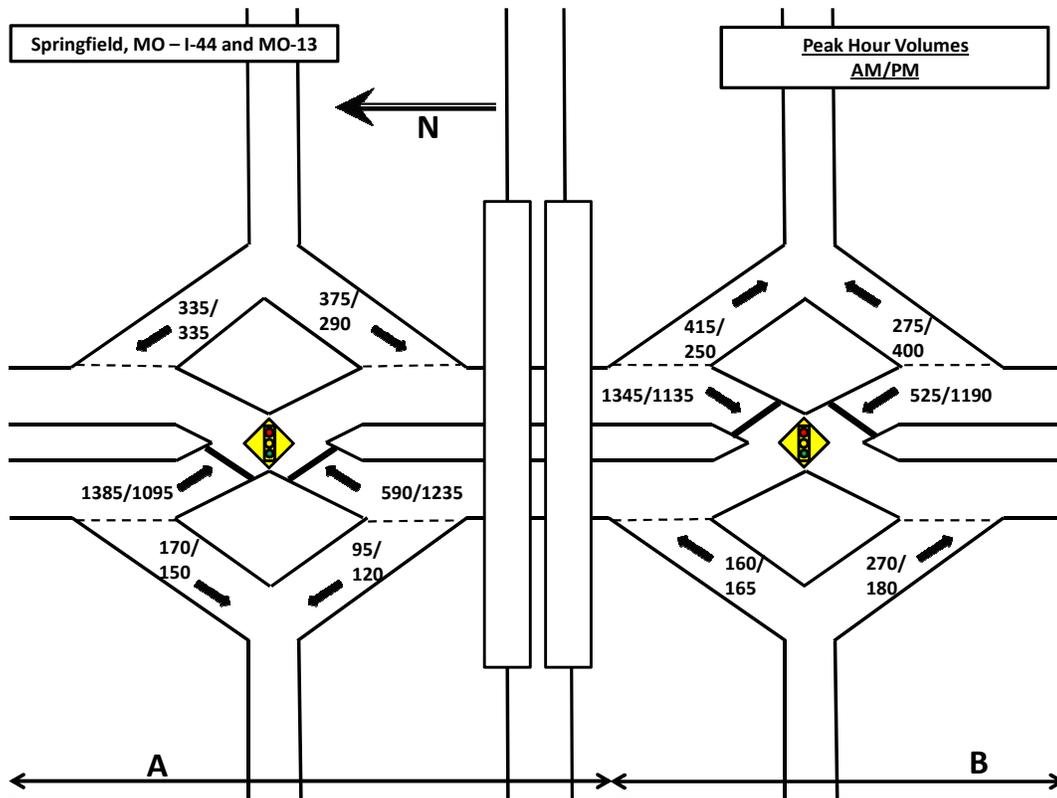


Figure 15: MO-13 After Traffic Volumes

Table 7: MO-13 DCD AM and PM Peak Period *After* Analysis

Route	After AM Peak						After PM Peak					
	Avg. Travel Time (min)	St. Dev. Travel Time	Avg. Control Delay (s)	Avg. # of Stops	LOS	# Runs	Avg. Travel Time (min)	St. Dev. Travel Time	Avg. Control Delay (s)	Avg. # of Stops	LOS	# Runs
South - North												
Route Start-Evergreen St. (0.1 mi)	0.21	0.13	16.2	0.1		10	0.19	0.09	9.6	0.15		13
Evergreen St.-South Crossover (0.17 mi)	1.36	0.6	58.8	1.3		10	1.12	0.31	52.8	0.92		13
South Crossover-North Crossover (0.1 mi)	0.25	0.02	0	0		10	0.29	0.03	7.8	0		13
North Crossover-Norton Rd. (0.13 mi)	0.6	0.57	32.4	0.5		10	0.47	0.23	18.6	0.62		13
Total South - North (0.5 mi)	2.42	0.63	107.4	1.9	D	10	2.07	0.46	88.8	1.69	D	13
North - South												
Route Start-Norton Rd. (0.19 mi)	0.45	0.27	21.6	0.33		9	0.83	0.32	41.4	0.77		13
Norton Rd. - North Crossover (0.13 mi)	0.41	0.35	24.6	0.11		9	0.69	0.42	31.8	0.77		13
North Crossover-South Crossover (0.1 mi)	0.38	0.28	30.6	0.22		9	0.28	0.03	0	0		13
South Crossover-Evergreen St. (0.17 mi)	0.64	0.26	26.4	0.56		9	0.42	0.07	5.4	0		13
Total North - South (0.59 mi)	1.88	0.43	103.2	1.22	D	9	2.22	0.32	78.6	1.54	C	13
South - West												
Route Start-Evergreen St. (0.1 mi)	0.18	N/A	0.6	0		1	0.3	0.2	3	0.33		3
Evergreen St.-South Crossover (0.17 mi)	1.14	N/A	0	1		1	1.44	0.39	0	1.33		3
South Crossover-North Crossover (0.09 mi)	0.44	N/A	0	1		1	0.27	0.03	0	0		3
Total South - West (0.34 mi)	1.76	N/A	0.6	2	A	1	2.01	0.45	3	1.66	A	6
West - North												
Route Start-South Crossover (0.15 mi)	0.25	N/A	0	0		1	0.25	0.03	6.6	0		6
South Crossover-North Crossover (0.08 mi)	0.12	N/A	0	0		1	0.41	0.4	0	0.67		6
North Crossover-Norton Rd. (0.13 mi)	0.38	N/A	0	1		1	1.35	0.71	0	1.17		6
Total West - North (0.36 mi)	0.75	N/A	0	1	A	1	2.01	0.59	6.6	1.84	A	6
North - East												
Route Start-Norton Rd. (0.19 mi)	0.26	0.12	67.2	0.5		2	1.15	0.12	57.6	1		5
Norton Rd.-North Crossover (0.13 mi)	0.69	0.68	0	0.5		2	0.59	0.11	0	0.8		5
North Crossover-South Crossover (0.08 mi)	0.19	0.01	0	0		2	0.29	0.04	0	0		5
Total North - East (0.4 mi)	1.14	0.77	67.2	1	A	2	2.03	0.19	57.6	1.8	A	5
East - South												
Route Start-North Crossover (0.13 mi)	0.47	N/A	22.2	1		1	0.28	0.09	0	0.25		4
North Crossover-South Crossover (0.08 mi)	0.14	N/A	0	0		1	0.61	0.42	0	1		4
South Crossover-Evergreen St. (0.17 mi)	0.95	N/A	0	1		1	0.48	0.09	0	0.25		4
Total East - South (0.38 mi)	1.56	N/A	22.2	2	B	1	1.37	0.32	0	1.5	A	4

Analysis of the AM peak period indicates that, as expected, the north-to-south route has a shorter travel time, average control delay, average number of stops, and level of service when compared to the south-to-north route. This was expected, since during the AM peak period, trips would be attracted to the business areas to the south, and priority should be given to the southbound direction. This, however, impacts the east-to-south left turn movements that merge onto the heavier traffic volumes in the southbound direction, which experience higher travel times, control delay, number of stops, and level of service than the opposing west-to-north movement.

Analysis of the PM peak period indicates that, as opposed to the AM peak period, the south-to-north route has a shorter travel time, when compared to the north-to-south route. Again, this was expected, since during the PM peak period, priority should be given to the northbound direction. Average control delay and number stops are slightly higher for the northbound direction. The west-to-north left turn movements that merge onto the heavier traffic volumes in the northbound direction experience higher travel times, control delays, levels of service, and numbers of stops than the opposing east-to-south movement.

Analysis of the level of service indicates that the arterial through movements operate at LOS D for the south-to-north route during both peak periods, and for the north-to-south route during the AM peak period. The north-to-south route operates at LOS C during the PM peak hour. All left turn movements operate at LOS A during both peak periods, except for the east-to-south route during the AM peak period, which operates at LOS B. Caution should be exercised when interpreting these results, due to the small sample size of the left turn movements. Also, the methodology described in Chapter 22 of the Highway Capacity Manual 2010 only includes average control delay at the two intersections of a diamond interchange; therefore, the level of service for each route includes the delay at the crossovers only.

Figures 16 and 17 show the time-space diagram and average number of stops, respectively, for the south-to-north route during the PM peak period. In the case of the south-to-north route, the time-space diagram in Figure 16 shows that the majority of vehicles stop at the south crossover, but all of them later progress through the north crossover. This can also be seen in Figure 17, which shows that no vehicles stop at the north crossover after clearing the south crossover. This could be interpreted as a measure of good progression at this particular DCD interchange.

Figures 18 and 19 show the time-space diagram and average number of stops, respectively, for the west-to-north route during the PM peak period. In the case of the west-to-north route, the time-space diagram in Figure 18 shows that no vehicle will stop at the south crossover (as expected), but it will later stop at the north crossover in almost all cases. This can also be seen in Figure 19, where the left turn off-ramp has no stops.

Appendix D shows time-space diagrams and average number of stops figures for all routes, during both peak periods, for *after* scenario.

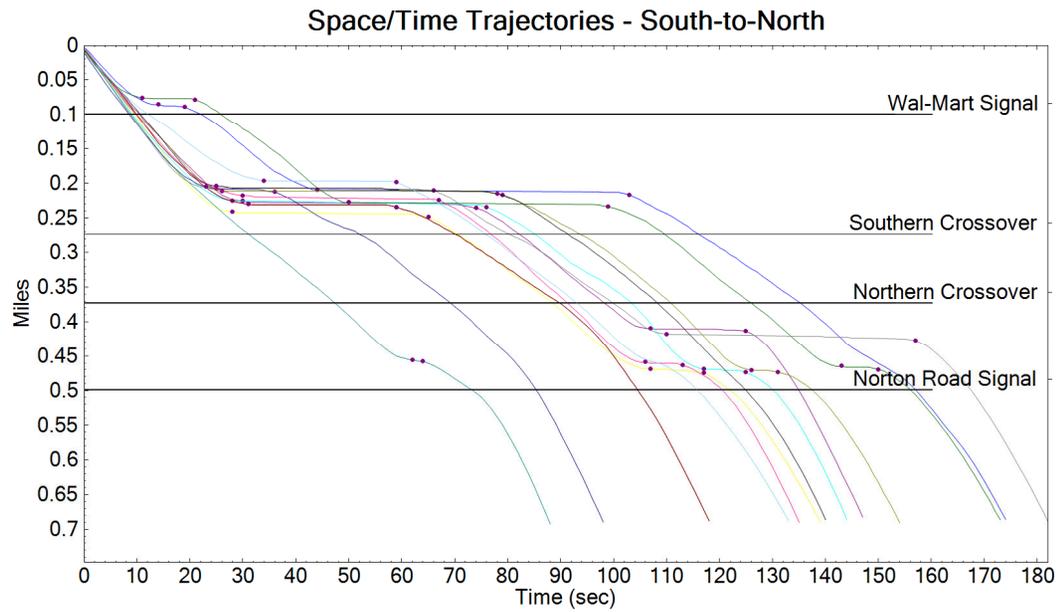


Figure 16: Time-Space Diagram for South-to-North Route – *After* Conditions – PM Peak Period

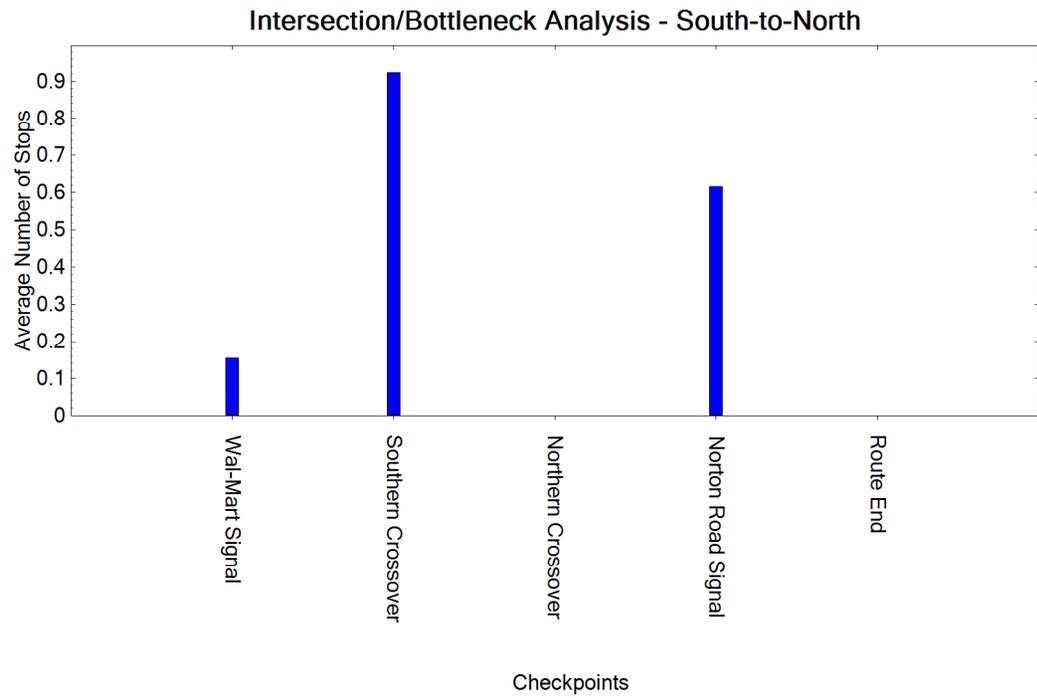


Figure 17: Average Number of Stops South-to-North Route – *After* Conditions – PM Peak Period

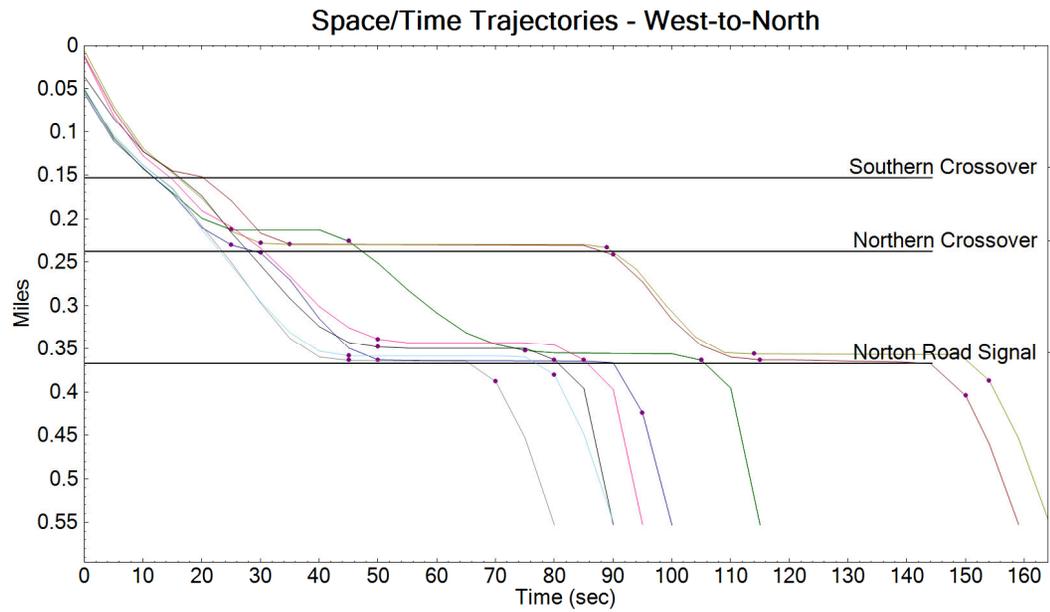


Figure 18: Time-Space Diagram for West-to-North Route – *After* Conditions – PM Peak Period

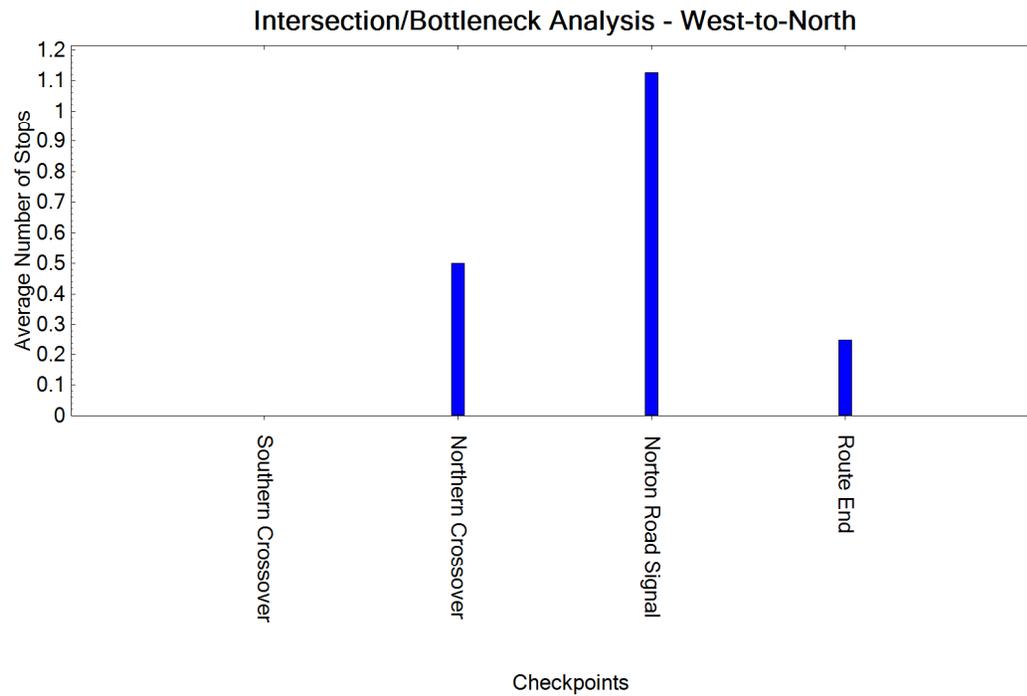


Figure 19: Average Number of Stops West-to-North Route – *After* Conditions – PM Peak Period

4.2.2 Analysis of Peak Period Queues for the MO-13 DCD

Refer to Table 8 and Table 9 for a summary of the AM and PM peak period queues for the south crossover and north crossover, respectively. The number in parenthesis at the Spillback columns refers to the distance to the closest upstream signalized intersection.

At the south crossover, northbound queues did not experience any spillback to the nearest signalized upstream intersection, located approximately 750 feet, during the AM and PM peak periods. Also, queues show higher lane utilization for the leftmost lane, likely due to the location of the northbound left turn on-ramp downstream of this intersection. Southbound queues did not experience any spillback to the nearest upstream signalized intersection; however, queues in the leftmost lane did block the southbound left turn on-ramp located approximately 75 feet upstream of the stop bar. Also, queues show higher lane utilization for the rightmost lane in the southbound direction.

At the north crossover, northbound queues did not experience any spillback to the nearest signalized upstream intersection, located approximately 480 feet; however, queues in the leftmost lane did block the northbound left turn off-ramp approximately 75 feet upstream of the stop bar. Also, queues show higher lane utilization in the rightmost lane. Southbound queues only experienced spillback into the upstream intersection during the AM peak period; however, this was only the case for the maximum queue observed and even the 95th percentile queue did not block the upstream intersection. There was higher lane utilization in the leftmost lane in the southbound direction, likely due to the location of the southbound left turn on-ramp downstream of this crossover.

Table 8: MO-13 DCD South Crossover Peak Period Queues

South Crossover	After							
AM Peak Period	NB Through Left Lane Queue (ft)	Spillback? (750 ft)	NB Through Right Lane Queue (ft)	Spillback? (750 ft)	SB Through Left Lane Queue (ft)	Spillback? (450 ft)	SB Through Right Lane Queue (ft)	Spillback? (450 ft)
Mean	119.4	No	71	No	44.2	No	99	No
Min	21.2	No	21	No	0	No	0	No
Max	254	No	191	No	127	No	212	No
Std. Dev.	53		40		33		48	
n	60		60		60		60	
PM Peak Period	NB Through Left Lane Queue (ft)	Spillback? (750 ft)	NB Through Right Lane Queue (ft)	Spillback? (750 ft)	SB Through Left Lane Queue (ft)	Spillback? (450 ft)	SB Through Right Lane Queue (ft)	Spillback? (450 ft)
Mean	328.2	No	230	No	53	No	117	No
Min	106	No	85	No	0	No	21	No
Max	509	No	424	No	254	No	360	No
Std. Dev.	92		86		46		64	
n	50		50		50		50	

Table 9: MO-13 DCD North Crossover Peak Period Queues

North Crossover	After							
AM Peak Period	NB Through Left Lane Queue (ft)	Spillback? (480 ft)	NB Through Right Lane Queue (ft)	Spillback? (480 ft)	SB Through Left Lane Queue (ft)	Spillback? (600 ft)	SB Through Right Lane Queue (ft)	Spillback? (600 ft)
Mean	34.9	No	52.1	No	158.1	No	124.6	No
Min	0	No	0	No	21	No	21	No
Max	106	No	233	No	678	Yes	276	No
Std. Dev.	24		45		141		60	
n	48		48		48		48	
PM Peak Period	NB Through Left Lane Queue (ft)	Spillback? (480 ft)	NB Through Right Lane Queue (ft)	Spillback? (480 ft)	SB Through Left Lane Queue (ft)	Spillback? (600 ft)	SB Through Right Lane Queue (ft)	Spillback? (600 ft)
Mean	43.8	No	58.6	No	223.7	No	198.9	No
Min	0	No	0	No	21	No	42	No
Max	212	No	170	No	509	No	466	No
Std. Dev.	43		41		125		100	
n	47		47		47		47	

4.2.3 MO-13 DCD Conflict and Erratic Maneuver Analysis

This section presents the conflict and erratic maneuver analysis of the MO-13 DCD interchange. The conflict and erratic maneuver analysis will be focused only on the *after* scenario. The results of the *after* conflict and erratic maneuver data collected in the field are illustrated in Table 10.

Analysis of the conflict and erratic maneuver data in Table 10 indicates that the events rate per hour is approximately the same for both crossovers. The north crossover has a slightly higher number of erratic maneuvers per hour, probably due to a larger number of signal compliance events, while the south crossover has a higher number of conflicts per hour, probably due to a higher number of merge conflicts.

Both off-ramp right turns were signalized; however, it was observed that at the south crossover, drivers often turned right on red creating merge conflicts.

Table 10: Conflict and Erratic Maneuver Summary for MO-13 DCD

Summary of Data	After	
	North Crossover	South Crossover
Number of Events Total	55	30
Total Time (hours)	8	4
Rate Events per hour	6.9	7.5
Conflicts vs. Erratic Maneuvers		
Number of Conflicts	7	9
Number of Erratic Maneuvers	48	21
Rate Conflicts/Hour	0.9	2.3
Rate EMs/Hour	6.0	5.3
Event Details		
U-Turn	0	0
Rear-End Conflicts	0	2
Signal Compliance	41	18
Stop	2	0
Merge	9	9
Lane Change	2	1
Backing Up	0	0
Left Turn Head on	0	0
Broke Down	0	0
Illegal Left	0	0
Wrong Way	0	0
Other	1	0

4.2.4 Summary of the MO-13 DCD Analysis

In summary, the *after* analysis of the I-44 and MO-13 DCD interchange in Springfield, MO shows that for the arterial through movements, priority is given to the direction with the heaviest traffic volumes, which results in lower travel times for the prioritized direction of traffic. Also, progression is achieved at the DCD for through the second crossover for many drivers making the arterial through movements.

The arterial through movements operate at LOS D during both peak periods, except for the north-to-south during the PM peak period, which operates at level of service C. The left turn routes operate at LOS A or LOS B for both peak periods. However, the sample size for the left turn routes is very small and any conclusions on its operation should be made with caution. Please note that the level of service analysis follows the methodology of the Highway Capacity Manual 2010, Chapter 22. Average control delay for only the two crossover was used for the level of service analysis.

Peak period queues did not block upstream intersections, except for the maximum queue observed at the north crossover in the southbound direction during the AM peak period. However, the 95th percentile queue for this movement does not block the upstream intersection. Also, queue data show that the leftmost lane carried a higher lane utilization for traffic going *into* the DCD, while the rightmost lane carries a higher lane utilization for the direction *exiting* the DCD.

Conflict and erratic maneuver analysis of *after* conditions indicate that while both crossovers had approximately the same number of events per hour, the north crossover had a slightly higher rate of erratic maneuvers per hour, and the south crossover had a higher rate of conflicts per hour, likely due to the merge conflict created by off-ramp right turn vehicles.

Overall, the DCD interchange at MO-13 operates at acceptable conditions based on analysis of the MOEs presented in this section. Peak period queues did not experience

spillback to the nearest signalized upstream intersections, except for the maximum queue observed at the north crossover in the southbound direction during the AM peak period.

4.3 US-60 and National Avenue DCD Analysis

Data collection at the US-60 and National Avenue DCD in Springfield, MO, only occurred for *after* conditions. Therefore, the contents of this section present an evaluation of the origin-destination travel times, average control delay, average number of stops, level of service, peak period queues, and conflicts and erratic maneuvers for the National Avenue DCD interchange for *after* conditions only. Refer to Appendix E for the field data collected for National Avenue DCD.

Calculation of origin-destination travel times, average control delay and average number of stops was obtained from TravTime software. Level of service was calculated from the methodology presented in Chapter 22 of the Highway Capacity Manual 2010. Note that while the LOS is given for the total route, the calculation only took into account the average delay at each of the two crossovers, not the entire route, as described in the Highway Capacity Manual 2010 methodology. Peak period queues and conflict data was obtained by direct observations in the field. Caution should be exercised with the analysis of the left turns during the AM and PM peak periods due to small sample sizes.

4.3.1 Analysis of Average Travel Time, Average Control Delay, Average Number of Stops, and Level of Service for National Avenue DCD

Figure 20 illustrates the AM and PM peak hour traffic volume counts for the *after* scenario at National Avenue. Table 11 summarizes the origin-destination travel times (in minutes), average control delay (in seconds), the average number of stops, and level of service for the National Avenue DCD site, for the AM peak period and for the PM peak period. MOEs for six routes are presented, south-to-north, north-to-south, south-to-west, west-to-north, north-to-east, and east-to-south.

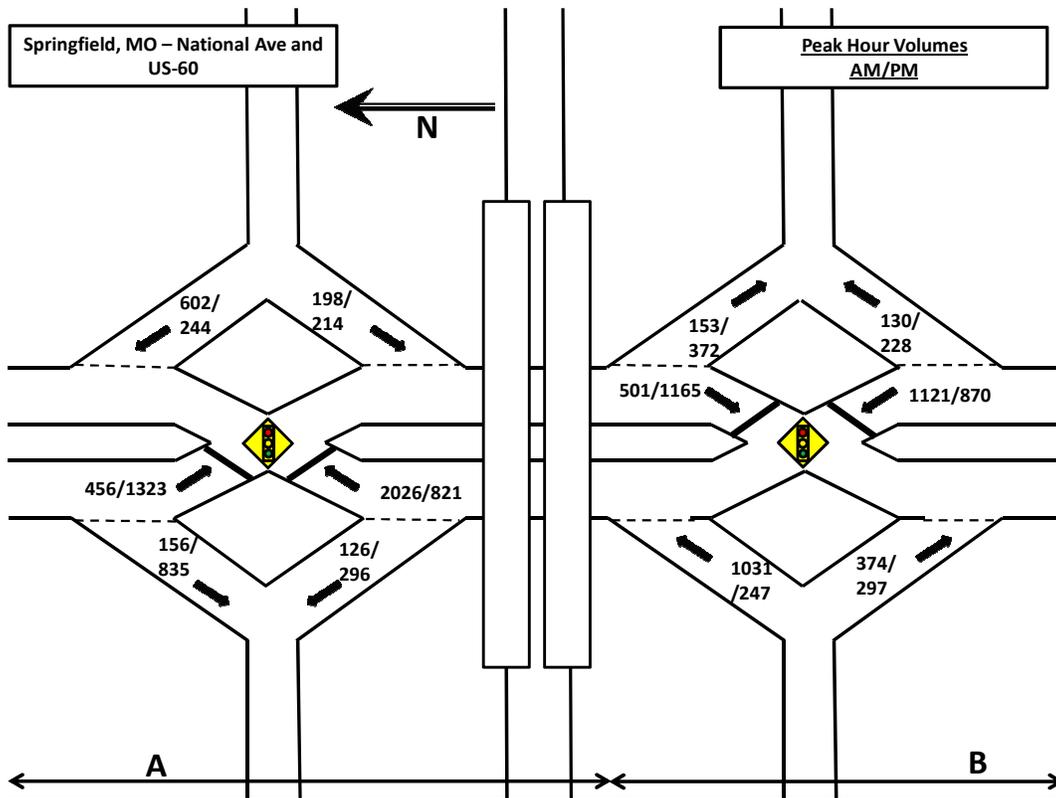


Figure 20: National *After* Traffic Volumes

Table 11: National Avenue DCD AM and PM Peak Period *After* Analysis

Route	After AM Peak						After PM Peak					
	Avg. Travel Time (min)	St. Dev. Travel Time	Avg. Control Delay (s)	Avg. # of Stops	LOS	# Runs	Avg. Travel Time (min)	St. Dev. Travel Time	Avg. Control Delay (s)	Avg. # of Stops	LOS	# Runs
North - South												
Route Start-Primrose St. (0.09 mi)	0.69	0.45	46.2	0.67		6	0.79	0.46	48.6	0.8		15
Primrose St.-North Crossover (0.28 mi)	0.86	0.16	25.8	0.83		6	0.54	0.04	4.8	0.07		15
North Crossover-South Crossover (0.13 mi)	0.35	0.02	0	0		6	0.38	0.24	20.4	0.07		15
South Crossover-Republic Rd. (0.21 mi)	0.88	0.25	27.6	0.83		6	0.78	0.38	31.2	0.6		15
Total North - South (0.71 mi)	2.78	0.74	99.6	2.33	B	6	2.49	0.61	105	1.54	B	15
South - North												
Route Start-Republic Rd. (0.07 mi)	1.03	1.06	20.4	0.67		6	0.64	0.35	37.2	0.77		13
Republic Rd.-South Crossover (0.21 mi)	1	0.81	43.2	0.67		6	1	0.66	43.2	0.77		13
South Crossover-North Crossover (0.13 mi)	0.41	0.29	2.4	0.17		6	0.31	0.03	2.4	0		13
North Crossover-Primrose St. (0.28 mi)	1.16	0.58	60.6	0.83		6	0.65	0.19	17.4	0.46		13
Total South - North (0.69)	3.6	1.36	126.6	2.34	C	6	2.6	0.75	100.2	2	C	13
West - North												
Route Start-South Crossover (0.2 mi)	0.28	N/A	0	0		1	0.94	0.42	45.6	0.75		8
South Crossover-North Crossover (0.12 mi)	0.3	N/A	0	0		1	1.19	0.17	54.6	1		8
North Crossover-Primrose St. (0.28 mi)	0.66	N/A	0	0		1	0.93	0.24	27.6	1		8
Total West - North (0.6 mi)	1.24	N/A	0	0	A	1	3.06	0.54	127.8	2.75	E	8
North - East												
Route Start-Primrose St. (0.09 mi)	0.1	0.03	0	0		2	1.09	0.95	0	1.5		6
Primrose St.-North Crossover (0.28 mi)	0.37	0.04	0	0		2	0.45	0.05	0	0		6
North Crossover-South Crossover (0.12 mi)	0.29	0.01	0	0		2	0.3	0.02	0	0		6
Total North - East (0.49 mi)	0.76	0.06	0	0	A	2	1.84	0.91	0	1.5	A	6
East - South												
Route Start-North Crossover (0.16 mi)	0.33	0.23	20.4	0.5		2	1.05	0.13	56.4	1		4
North Crossover-South Crossover (0.12 mi)	0.94	0.22	0	1.5		2	0.94	0.12	0	1		4
South Crossover-Republic Rd. (0.21 mi)	0.31	0.02	0	0		2	1.27	0.37	69.6	1		4
Total East - South (0.49 mi)	1.58	0.42	20.4	2	B	2	3.26	0.34	126	3	D	4
South - West												
Route Start-Republic Rd. (0.07 mi)	0.21	N/A	0	0		1	0.93	0.29	0	1		6
Republic Rd.-South Crossover (0.21 mi)	1.39	N/A	67.8	1		1	0.7	0.31	0	0.67		6
South Crossover-North Crossover (0.12 mi)	0.27	N/A	0	0		1	0.32	0.03	0	0		6
Total South - West (0.4 mi)	1.87	N/A	67.8	1	D	1	1.95	0.31	0	1.67	A	6

Analysis of the AM peak period indicates that the north-to-south route has a shorter travel time, average control delay, and average number of stops when compared to the south-to-north route. The left turn route with the lowest average travel time, control delay and number of stops is the north-to-east route. Caution should be exercised with the analysis of the left turns during the AM peak period, due to small sample sizes.

Analysis of the PM peak period indicates that the north-to-south route continues to have a shorter travel time and number of stops when compared to the south-to-north route. Average control delay is slightly higher for the southbound direction. The left turn movements experience worse MOEs when compared to the AM peak period, probably due to heavier traffic volumes during the PM peak period.

Analysis of the level of service indicates that the north-to-south movement operates at LOS B during both peak periods, and the south-to-north route operates at LOS C during both peak periods. All left turn movements operate at LOS D or better during both peak periods, except for the west-to-north route during the PM peak period, which operates at LOS E. Caution should be exercised when interpreting these results, due to the small sample size of the left turn movements. Also, the methodology described in Chapter 22 of the Highway Capacity Manual 2010 only includes average control delay at the two intersections of a diamond interchange; therefore, the level of service for each route includes the delay at the crossovers only.

Figures 21 and 22 show the time-space diagram and average number of stops, respectively, for the south-to-north route during the PM peak period. Figures 23 and 24 show the time-space diagram and average number of stops, respectively, for the west-to-north route during the PM peak period.

In the case of the south-to-north route, the time-space diagram shows that very poor progression exist between Republic Road and the south crossover; however, all vehicles do not stop at the north crossover in the northbound direction. Most vehicles will then again stop at Primrose Street.

In the case of the west-to-north route, the time-space diagram shows that most vehicles will stop at the south crossover and again at the north crossover, experiencing very long delays.

Appendix F shows time-space diagrams and average number of stops figures for all routes, during both peak periods, for *after* scenario.

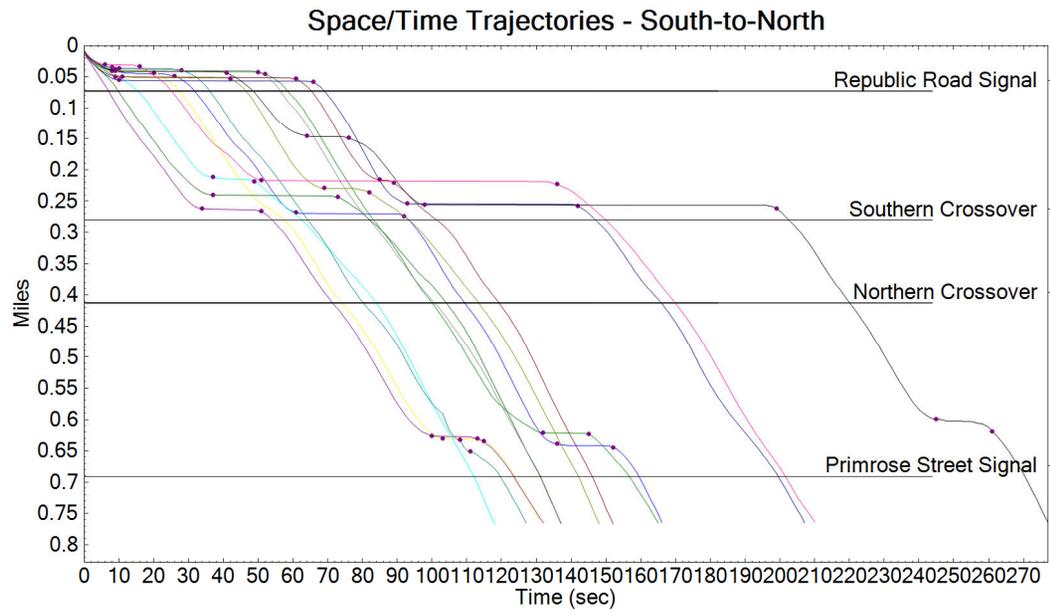


Figure 21: Time-Space Diagram for South-to-North Route – *After* Conditions – PM Peak Period

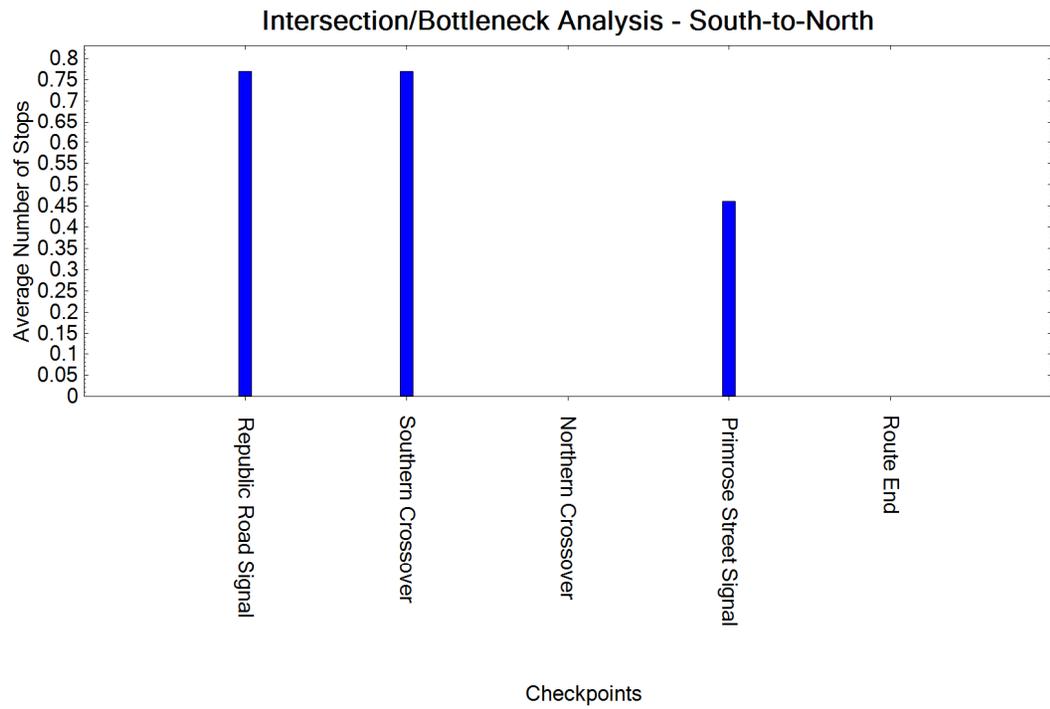


Figure 22: Average Number of Stops South-to-North Route – *After* Conditions – PM Peak Period

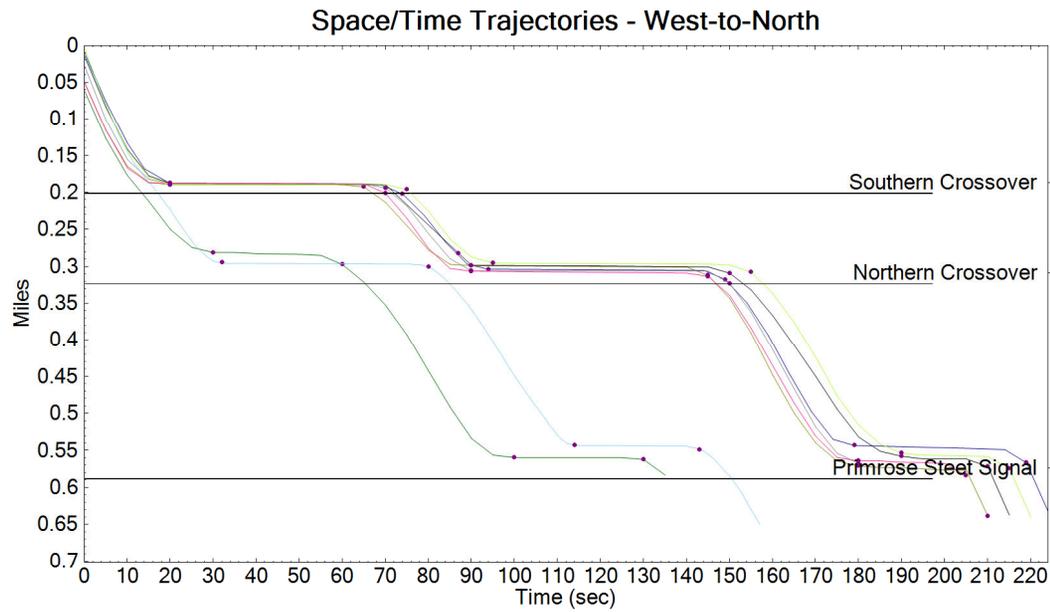


Figure 23: Time-Space Diagram for West-to-North Route – *After* Conditions – PM Peak Period

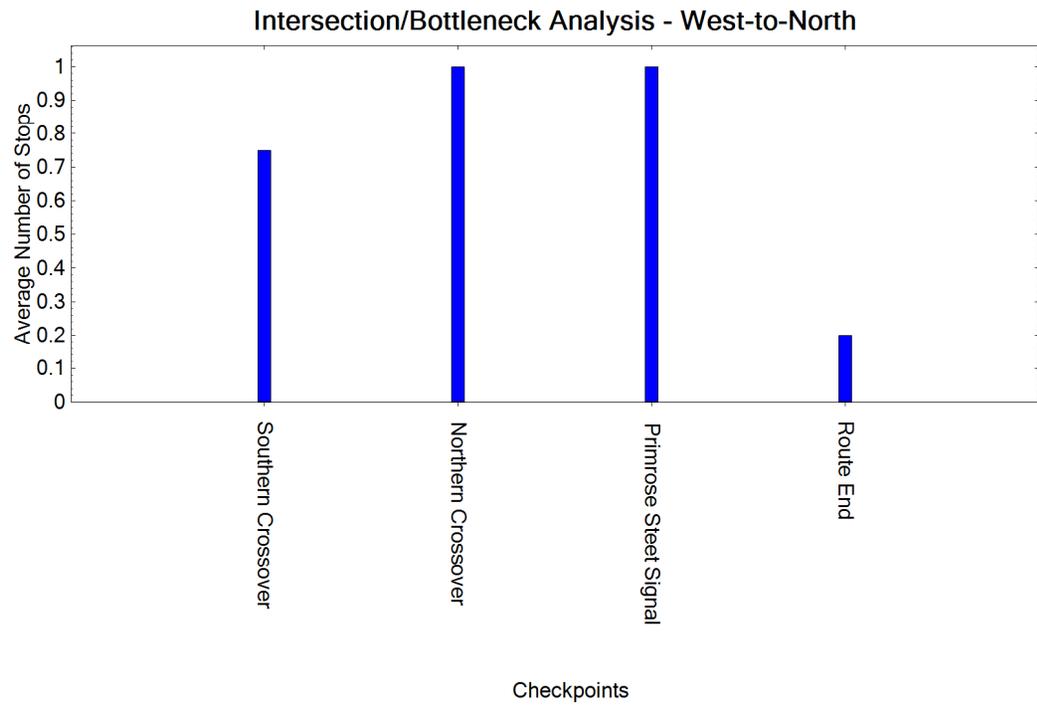


Figure 24: Average Number of Stops West-to-North Route – *After* Conditions – PM Peak Period

4.3.2 Analysis of Peak Period Queues for National Avenue DCD

Refer to Table 12 and Table 13 for a summary of the AM and PM peak period queues for the south crossover and north crossover. The number in parenthesis at the Spillback columns refers to the distance to the closest upstream signalized intersection.

At the south crossover, northbound queues do not experience any spillback to the nearest signalized upstream intersection, located at approximately 1,000 feet, during the AM and PM peak hour. Also, queues show higher lane utilization for the leftmost lane, likely due to the location of the south-to-west on-ramp left turn lane downstream of this intersection. Southbound queues also do not experience any spillback to the north crossover, located at approximately 650 feet. Also, queues show higher lane utilization for the leftmost lane in the southbound direction.

At the north crossover, northbound queues do not experience any spillback to the south crossover, located at approximately 660 feet; however, queues at the leftmost lane do block the south-to-west on-ramp left turn located at approximately 100 feet from the stop bar. Also, queues show higher lane utilization for the rightmost lane. Southbound queues do not experience spillback to the nearest signalized intersection, located at approximately 1,500 feet. Also, queues show higher lane utilization for the leftmost lane in the southbound direction during the PM peak hour, and the southbound center lane during the AM peak hour.

Table 12: National Avenue DCD South Crossover Peak Period Queues

After	AM Peak Hour Queues (ft)									
South Crossover	NB Left Lane	Spillback? (1000 ft)	NB Center Lane	Spillback? (1000 ft)	NB Right Lane	Spillback? (1000 ft)	SB Left Lane	Spillback? (650 ft)	SB Right Lane	Spillback? (650 ft)
Mean	303.4	No	121.0	No	51.7	No	64.5	No	49.0	No
Min	64	No	0	No	0	No	0	No	0	No
Max	721	No	276	No	191	No	148	No	148	No
Std. Dev.	166.5		70.2		44.4		41.2		40.1	
n	48		48		48		48		48	
After	PM Peak Hour Queues (ft)									
South Crossover	NB Left Lane	Spillback? (1000 ft)	NB Center Lane	Spillback? (1000 ft)	NB Right Lane	Spillback? (1000 ft)	SB Left Lane	Spillback? (650 ft)	SB Right Lane	Spillback? (650 ft)
Mean	355.7	No	152.5	No	28.9	No	105.4	No	91.9	No
Min	127	No	21	No	0	No	21	No	21	No
Max	848	No	530	No	106	No	233	No	191	No
Std. Dev.	214.4		112.5		31.3		60.5		45.6	
n	36		36		36		36		36	

Table 13: National Avenue DCD North Crossover Peak Period Queues

After	AM Peak Hour Queues (ft)											
North Crossover	SB Left Lane	Spillback? (1500 ft)	SB Center Lane	Spillback? (1500 ft)	SB Right Lane	Spillback? (1500 ft)	NB Left Lane	Spillback? (660 ft)	NB Center Lane	Spillback? (660 ft)	NB Right Lane	Spillback? (660 ft)
Mean	81.9	No	97.8	No	55.0	No	187.1	No	149.2	No	229.9	No
Min	0	No	21	No	0	No	42	No	42	No	85	No
Max	212	No	254	No	170	No	466	No	466	No	466	No
Std. Dev.	54.3		46.2		37.6		115.4		84.6		93.2	
n	52		52		52		52		52		52	
After	PM Peak Hour Queues (ft)											
North Crossover	SB Left Lane	Spillback? (1500 ft)	SB Center Lane	Spillback? (1500 ft)	SB Right Lane	Spillback? (1500 ft)	NB Left Lane	Spillback? (660 ft)	NB Center Lane	Spillback? (660 ft)	NB Right Lane	Spillback? (660 ft)
Mean	170.2	No	130.5	No	85.9	No	66.9	No	79.2	No	89.3	No
Min	0	No	21	No	21	No	0	No	21	No	21	No
Max	424	No	424	No	212	No	170	No	170	No	212	No
Std. Dev.	97.8		77.5		46.0		42.3		36.4		48.6	
n	38		38		38		38		38		38	

4.3.3 National Avenue DCD Conflict and Erratic Maneuver Analysis

This section present the preliminary results of the conflict and erratic maneuver analysis of the National DCD interchange. The conflict and erratic maneuver analysis will be focused only on the *after* scenario. The results of the *after* conflict and erratic maneuver data collected in the field are illustrated in Table 14. Note that for this site, conflicts and erratic maneuvers were collected by two different observers at each crossover; therefore, interpretation of what constituted an erratic maneuver and a conflict could have varied slightly for each of the observers.

Analysis of the conflict and erratic maneuver data in Table 14 indicates that the events rate per hour is higher for the north crossover. The north crossover has a higher number of erratic maneuvers per hour, probably due to a larger number of signal compliance events, while the south crossover has a higher number of conflicts per hour, probably due to a higher number of merge conflicts.

Table 14: Conflict and Erratic Maneuver Summary for National Avenue DCD

Summary of Data	After	
	North Crossover	South Crossover
Number of Events Total	29	15
Total Time (hours)	8	8
Rate Events per hour	3.6	1.9
Conflicts vs. Erratic Maneuvers		
Number of Conflicts	3	11
Number of Erratic Maneuvers	26	4
Rate Conflicts/Hour	0.4	1.4
Rate EMS/Hour	3.3	0.5
Event Details		
U-Turn	1	0
Rear-End Conflicts	1	1
Signal Compliance	19	5
Stop	1	0
Merge	4	6
Lane Change	3	3
Backing Up	0	0
Left Turn Head on	0	0
Broke Down	0	0
Illegal Left	0	0
Wrong Way	0	0
Other	0	0

4.3.4 Summary of the National Avenue DCD Analysis

In summary, the *after* analysis of the US Route 60 and National Avenue DCD interchange in Springfield, MO shows that for the arterial through movements, the south-to-north movement experience worse MOEs during both analysis periods. Also, once a vehicle enters the DCD, it will progress through the next crossover without stopping, as shown in Figure 21.

The north-to-south through movement operates at LOS B during both peak periods, and the south-to-north route operates at LOS C during both peak periods.. All left turn movements operate at LOS D or better during both peak periods, except for the west-to-north route during the PM peak period, which operates at LOS E. Caution should be exercised when interpreting these results, due to the small sample size of the left turn movements. Also, the methodology described in Chapter 22 of the Highway Capacity Manual 2010 only includes average control delay at the two intersections of a diamond interchange; therefore, the level of service for each route includes the delay at the crossovers only. Peak period queues do not block upstream intersections. Also, queues show that the leftmost lane carries a higher lane utilization for traffic going *into* the DCD, while the rightmost lane carries a higher lane utilization for the direction *exiting* the DCD in the north crossover, while the south crossover shows a higher lane utilization for the center and left lanes.

Conflict and erratic maneuver analysis of *after* conditions indicate that the north crossover has a higher rate of total events and erratic maneuvers, while the south crossover has a higher rate of conflicts per hour.

Overall, the National Avenue DCD site showed good operational performance for the arterial through movements and for the left turn movements. However, small sample size for the left turn should be taken into consideration when analyzing the data.

4.4 I-270 and Dorsett Road DCD, Maryland Heights, MO Analysis

Data collection at the I-270 and Dorsett Road DCD, in Maryland Heights, MO, only occurred for *after* conditions. Therefore, the contents of this section present an evaluation of the origin-destination travel times, average control delay, average number of stops, level of service, peak period queues, and conflicts and erratic maneuvers for the Dorsett Road DCD interchange for *after* conditions only. Refer to Appendix G for the field data collected for Dorsett Road DCD.

Calculation of origin-destination travel times, average control delay and average number of stops was obtained from TravTime software. Level of service was calculated from the methodology presented in Chapter 22 of the Highway Capacity Manual 2010. Note that while the LOS is given for the total route, the calculation only took into account the average delay at each of the two crossovers, not the entire route, as described in the Highway Capacity Manual 2010 methodology. Peak period queues and conflict data was obtained by direct observations in the field. Caution should be exercised with the analysis of the left turns during the AM and PM peak periods due to small sample sizes.

4.4.1 Analysis of Average Travel Time, Average Control Delay, Average Number of Stops, and Level of Service for Dorsett Road DCD

Figure 25 illustrates the AM and PM peak hour traffic volume counts for the *after* scenario at Dorsett Road. Table 15 summarizes the origin-destination travel times (in minutes), average control delay (in seconds), the average number of stops, and level of service for the Dorsett Road DCD site, for the AM peak period and for the PM peak period. MOEs for six routes are presented, west-to-east, east-to-west, west-to-north, north-to-east, east-to-south, and south-to-east.

Analysis of the arterial through routes indicates that the east-to-west route has a shorter travel time, and average number of stops when compared to the west-to-east route, during

both peak periods. However, the east-to-west route has a higher average control delay, when compared to the west-to-east route.

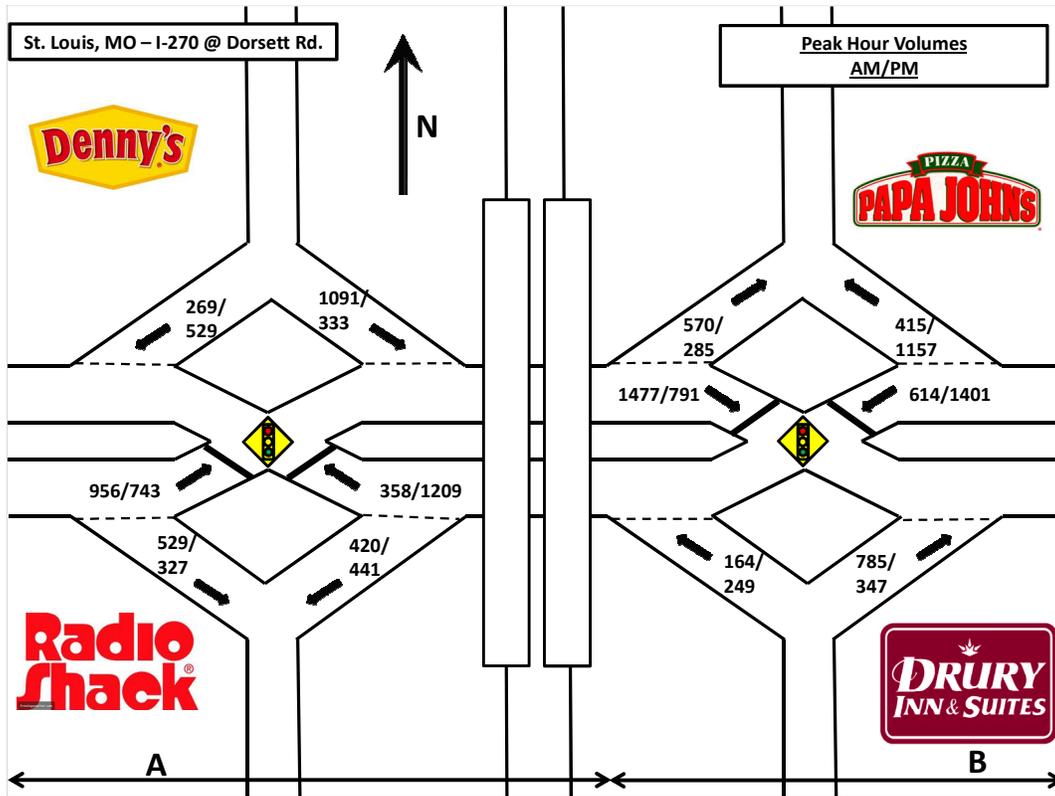


Figure 25: Dorsett After Traffic Volumes

Table 15: Dorsett Road DCD AM and PM Peak Period *After* Analysis

Route	After AM Peak						After PM Peak					
	Avg. Travel Time (min)	St. Dev. Travel Time	Avg. Control Delay (s)	Avg. # of Stops	LOS	# Runs	Avg. Travel Time (min)	St. Dev. Travel Time	Avg. Control Delay (s)	Avg. # of Stops	LOS	# Runs
East - West												
Route Start-Weldon Pkwy. (0.05 mi)	0.09	0.02	6	0		18	0.16	0.08	16.2	0.25		16
Weldon Pkwy.-Old Dorsett Rd. (0.24 mi)	0.53	0.19	19.8	0.17		18	0.77	0.27	18.6	1.06		16
Old Dorsett Rd.-East Crossover (0.04 mi)	0.41	0.2	28.2	0.78		18	0.28	0.4	62.4	0.25		16
East Crossover-West Crossover (0.09 mi)	0.23	0.03	8.4	0.06		18	0.22	0.11	27.6	0.06		16
West Crossover-McKelvey Rd. (0.29 mi)	0.68	0.19	28.2	0.33		18	0.72	0.25	35.4	0.44		16
Total East - West (0.71 mi)	1.94	0.4	90.6	1.34	C	18	2.15	0.68	160.2	2.06	E	16
West - East												
Route Start-McKelvey Rd. (0.11 mi)	0.73	0.43	37.2	0.81		16	0.62	0.43	34.2	0.75		16
McKelvey Rd.-West Crossover (0.29 mi)	0.85	0.24	24.6	0.69		16	0.9	0.32	30.6	0.69		16
West Crossover-East Crossover (0.09 mi)	0.58	0.1	23.4	1		16	0.4	0.42	64.2	0.19		16
East Crossover-Old Dorsett Rd. (0.04 mi)	0.1	0.01	0	0		16	0.16	0.29	6	0.12		16
Old Dorsett Rd.-Weldon Pkwy. (0.24 mi)	0.52	0.13	3.6	0.12		16	0.62	0.24	12	0.38		16
Total West - East (0.77 mi)	2.78	0.56	88.8	2.62	C	16	2.7	0.87	147	2.13	E	16
East - South												
Route Start-Weldon Pkwy. (0.05 mi)	0.1	0.02	0	0		4	0.18	0.07	21.6	0.29		7
Weldon Pkwy.-Old Dorsett Rd. (0.24 mi)	0.63	0.27	33	0.5		4	1.61	0.8	43.2	2.14		7
Old Dorsett Rd.-East Crossover (0.04 mi)	0.55	0.15	0	1		4	0.37	0.48	0	0.29		7
East Crossover-West Crossover (0.08 mi)	0.21	0.03	0	0		4	0.2	0.02	0	0		7
Total East - South (0.41 mi)	1.49	0.36	33	1.5	A	4	2.36	0.91	64.8	2.72	A	7
South - West												
Route Start-East Crossover (0.16 mi)	0.45	0.12	12.6	0.75		4	0.48	0.26	18.6	0.5		6
East Crossover-West Crossover (0.07 mi)	0.66	0.05	33	1		4	0.58	0.22	35.4	0.5		6
West Crossover-McKelvey Rd. (0.29 mi)	0.53	0.01	31.2	0		4	0.8	0.45	33	0.5		6
Total South - West (0.52 mi)	1.64	0.24	76.8	1.75	C	4	1.86	0.59	87	1.5	C	6
West - North												
Route Start-McKelvey Rd. (0.11 mi)	0.84	0.3	44.4	0.88		8	0.43	0.31	21	0.57		7
McKelvey Rd.-West Crossover (0.29 mi)	0.77	0.2	18.6	0.75		8	1.15	0.63	43.8	0.86		7
West Crossover-East Crossover (0.07 mi)	0.17	0.02	0	0		8	0.18	0.02	0	0		7
Total West - North (0.47 mi)	1.78	0.37	63	1.63	B	8	1.76	0.79	64.8	1.43	C	7
North - East												
Route Start-West Crossover (0.18 mi)	2.02	1.35	30.6	3.33		6	0.77	0.32	31.2	0.83		6
West Crossover-East Crossover (0.07 mi)	0.23	0.06	0	0		6	0.74	0.32	43.2	0.83		6
East Crossover-Old Dorsett Rd. (0.04 mi)	0.21	0.13	16.2	0.33		6	0.12	0.01	0	0		6
Old Dorsett Rd.-Weldon Pkwy. (0.24 mi)	0.6	0.09	0	0.17		6	0.68	0.29	12.6	0.67		6
Total North - East (0.53 mi)	3.06	1.36	46.8	3.83	C	6	2.31	0.75	87	2.33	D	6

During the AM peak period, the left turn route with the lowest average travel time, average control delay, level of service, and number of stops is the east-to-south route. During the PM peak period, the left turn route with the lowest average travel time, average control delay, level of service, and number of stops is the west-to-north route. Caution should be exercised with the analysis of the left turns, due to small sample size.

Analysis of the level of service indicates that during the AM peak period, the arterial through movements operate at LOS C. During the PM peak period, the arterial through movements operate at LOS E. All left turn movements operate at LOS D or better during both peak periods. Caution should be exercised when interpreting these results, due to the small sample size of the left turn movements. Also, the methodology described in Chapter 22 of the Highway Capacity Manual 2010 only includes average control delay at the two intersections of a diamond interchange; therefore, the level of service for each route includes the delay at the crossovers only.

Figures 26 and 27 show the time-space diagram and average number of stops, respectively, for the east-to-west route during the PM peak period. Figures 28 and 29 show the time-space diagram and average number of stops, respectively, for the south-to-west route during the PM peak period.

In the case of the east-to-west route, the time-space diagram shows that most of the vehicles progress through both crossovers, while about half of all vehicles progress through Old Dorsett Road, the east crossover and the west crossover. Fourteen out of sixteen vehicles will stop at least once during the east-to-west.

In the case of the south-to-west route, the time-space diagram shows that half of all vehicles will stop at the west crossover after entering the DCD.

Appendix H shows time-space diagrams and average number of stops figures for all routes, during both peak periods, for *after* scenario.

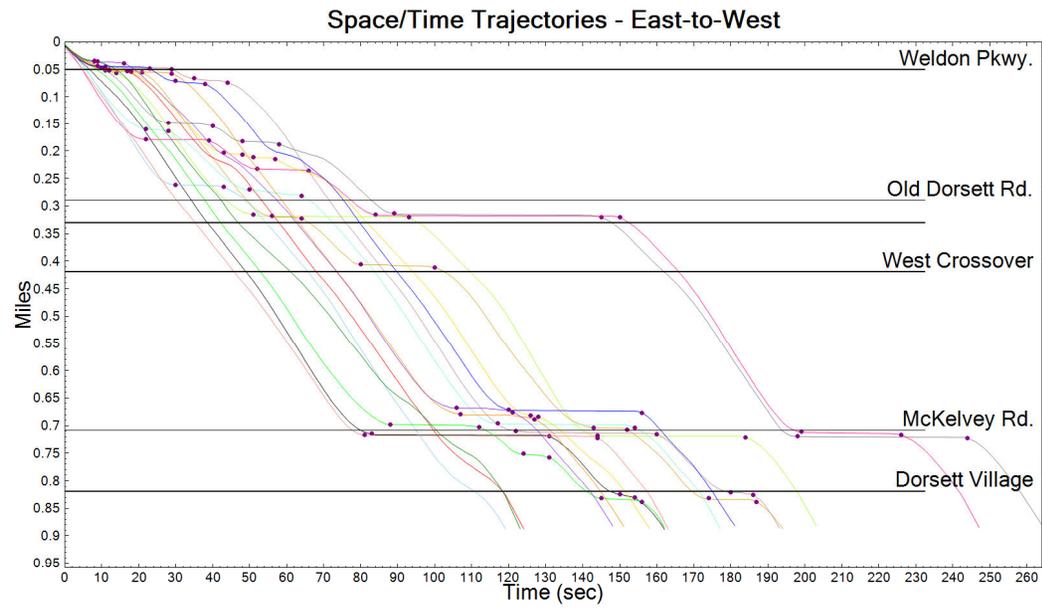


Figure 26: Time-Space Diagram for East-to-West Route – *After* Conditions – PM Peak Period

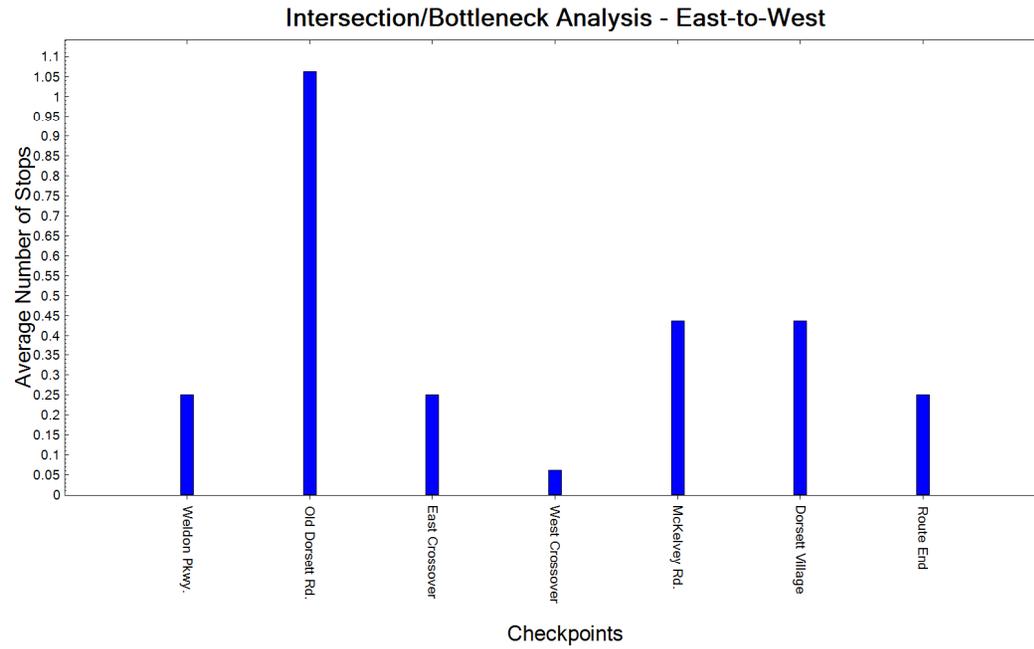


Figure 27: Average Number of Stops East-to-West Route – *After* Conditions – PM Peak Period

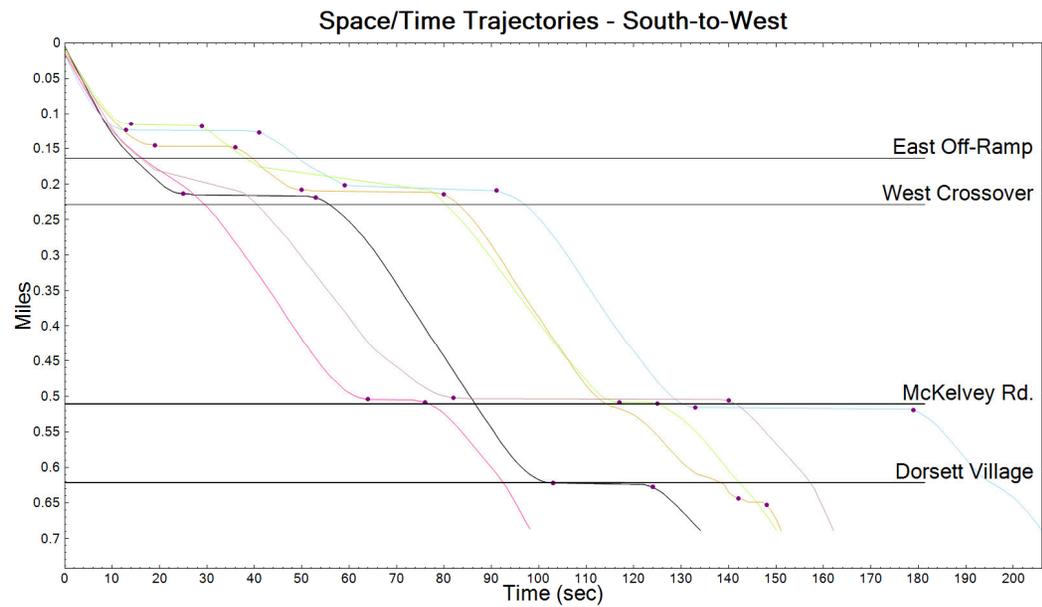


Figure 28: Time-Space Diagram for South-to-West Route – *After* Conditions – PM Peak Period

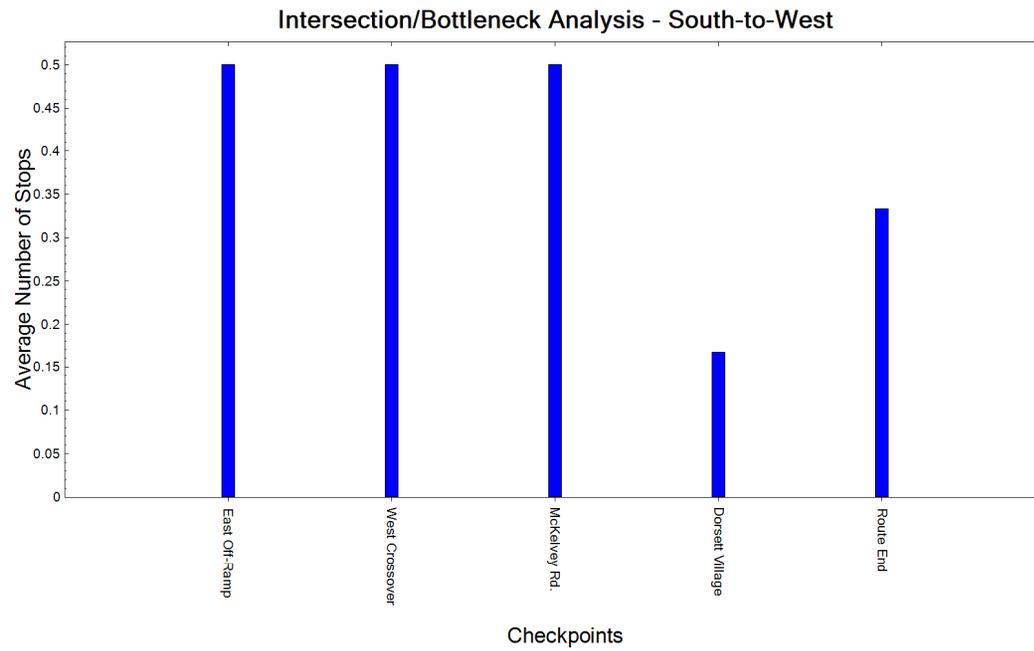


Figure 29: Average Number of Stops South-to-West Route – *After* Conditions – PM Peak Period

4.4.2 Analysis of Peak Period Queues for Dorsett Road DCD

Refer to Table 16 and Table 17 for a summary of the AM and PM peak period queues for the west crossover and east crossover. The number in parenthesis at the spillback columns refers to the distance to the closest upstream signalized intersection.

At the west crossover, eastbound queues do not experience any spillback to the west crossover, located at approximately 450 feet. Also, queues show higher lane utilization for the rightmost lane during the AM peak period, and the leftmost lane during the PM peak period. Westbound queues do not experience spillback to the nearest signalized intersection, located at approximately 900 feet. Also, queues show higher lane utilization for the leftmost lane during the both peak periods.

At the east crossover, eastbound queues do not experience any spillback to the nearest signalized upstream intersection, located at approximately 1,500 feet, during the AM and PM peak hour. Also, queues show higher lane utilization for the leftmost lane, likely due to the location of the west-to-north on-ramp left turn lane downstream of this intersection. Westbound queues also do not experience any spillback to the east crossover, located at approximately 450 feet. Also, queues show higher lane utilization for the rightmost lane in the southbound direction.

Table 16: Dorsett Road DCD West Crossover Peak Period Queues

After	AM Peak Hour Queues (ft)									
West Crossover	EB Left Lane	Spillback? (1500 ft)	EB Center Lane	Spillback? (1500 ft)	EB Right Lane	Spillback? (1500 ft)	WB Left Lane	Spillback? (450 ft)	WB Right Lane	Spillback? (450 ft)
Mean	167.2	No	57.1	No	52.2	No	22.6	No	30.6	No
Min	0	No	0	No	0	No	0	No	0	No
Max	424	No	148	No	170	No	106	No	127	No
Std. Dev.	75.1		34.2		37.8		21.2		26.5	
n	108		108		108		108		108	
After	PM Peak Hour Queues (ft)									
West Crossover	EB Left Lane	Spillback? (1500 ft)	EB Center Lane	Spillback? (1500 ft)	EB Right Lane	Spillback? (1500 ft)	WB Left Lane	Spillback? (450 ft)	WB Right Lane	Spillback? (450 ft)
Mean	194.8	No	98.1	No	73.7	No	148.4	No	217.5	No
Min	0	No	0	No	0	No	0	No	21	No
Max	318	No	339	No	276	No	424	No	424	No
Std. Dev.	76.7		55.4		49.1		80.5		91.4	
n	59		59		59		58		58	

Table 17: Dorsett Road DCD East Crossover Peak Period Queues

After	AM Peak Hour Queues (ft)									
East Crossover	WB Left Lane	Spillback? (900 ft)	WB Center Lane	Spillback? (900 ft)	WB Right Lane	Spillback? (900 ft)	EB Left Lane	Spillback? (450 ft)	EB Right Lane	Spillback? (450 ft)
Mean	117.1	No	23.5	No	42.2	No	92.5	No	143.9	No
Min	0	No	0	No	0	No	0	No	0	No
Max	360	No	127	No	212	No	318	No	339	No
Std. Dev.	82.8		26.9		45.1		64.3		99.6	
n	82		82		108		108		108	
After	PM Peak Hour Queues (ft)									
East Crossover	WB Left Lane	Spillback? (900 ft)	WB Center Lane	Spillback? (900 ft)	WB Right Lane	Spillback? (900 ft)	EB Left Lane	Spillback? (450 ft)	EB Right Lane	Spillback? (450 ft)
Mean	210.4	No	52.2	No	112.3	No	132.0	No	122.4	No
Min	0	No	0	No	0	No	0	No	0	No
Max	424	No	148	No	424	No	212	No	254	No
Std. Dev.	134.7		37.7		73.7		46.3		56.2	
n	67		67		67		66		66	

4.4.3 Dorsett Road DCD Conflict and Erratic Maneuver Analysis

This section present the preliminary results of the conflict and erratic maneuver analysis of the Dorsett Road DCD interchange. The conflict and erratic maneuver analysis will be focused only on the *after* scenario. The results of the *after* conflict and erratic maneuver data collected in the field are illustrated in Table 18.

Analysis of the conflict and erratic maneuver data in Table 18 indicates that the events rate per hour is higher for the east crossover. The east crossover has a higher number of conflicts per hour, probably due to a larger number of merge conflicts, while the west crossover has a higher number of erratic maneuvers per hour, probably due to a higher number of signal compliance events.

Table 18: Conflict and Erratic Maneuver Summary for Dorsett Road DCD

Summary of Data	After	
	West Crossover	East Crossover
Number of Events Total	14	15
Total Time (hours)	8	6
Rate Events per hour	1.8	2.5
Conflicts vs. Erratic Maneuvers		
Number of Conflicts	4	10
Number of Erratic Maneuvers	10	5
Rate Conflicts/Hour	0.5	1.7
Rate EMS/Hour	1.3	0.8
Event Details		
U-Turn	0	0
Rear-End Conflicts	1	1
Signal Compliance	7	2
Stop	1	2
Merge	2	7
Lane Change	3	2
Backing Up	0	1
Left Turn Head on	0	0
Broke Down	0	0
Illegal Left	0	0
Wrong Way	0	0
Other	0	0

4.4.4 Summary of the Dorsett Road DCD Analysis

In summary, the *after* analysis of the I-270 and Dorsett Road DCD interchange in Maryland Heights, MO shows that for the arterial through movements, the east-to-west route has a shorter travel time, and average number of stops when compared to the west-to-east route, during both peak periods. However, the east-to-west route has a higher average control delay, when compared to the west-to-east route. Also, once a vehicle enters the DCD, it will progress through the next crossover without stopping, as shown in Figure 21.

The arterial through movements operate at LOS C during the AM peak period. During the PM peak period, the arterial through movements operate at LOS E. The left turn routes operate at level of service D or better during both peak periods. However, the sample size for the left turn routes is very small and any conclusions on its operation should be taken with caution.

Peak period queues do not block upstream intersections. Also, queues show that the leftmost lane carries a higher lane utilization for traffic going *into* the DCD, while the rightmost lane carries a higher lane utilization for the direction *exiting* the DCD in the west crossover, while the east crossover shows a higher lane utilization for the rightmost lane during the AM peak period, and the leftmost lane during the PM peak period.

Conflict and erratic maneuver analysis of *after* conditions indicate that the east crossover has a higher rate of total events and conflicts per hour, while the west crossover has a higher rate of erratic maneuvers per hour.

Overall, the Dorsett Road DCD site showed acceptable operational performance during the AM peak period for all movements; while the arterial through movements operated at excessive level of service during the PM peak hour.

4.5 Calibration and Validation of SimTraffic Model of DCD Interchanges

The contents of this section present the results of the process used to calibrate and validate the DCD interchange model in SimTraffic.

Synchro 7 was used to build the *after* scenario of the MO-13 DCD site. SimTraffic is a microsimulation tool directly linked to Synchro, from which all relevant data is imported into the SimTraffic model. The SimTraffic model is a time-based, stochastic simulation of individual vehicles. Measures of effectiveness (MOE's) are computed for each vehicle in the model for every time step of model simulation. SimTraffic has the ability to collect system wide measurements, as well as intersection and corridor related statistics.

The DCD interchange of I-44 and MO-13 in Springfield, MO was chosen to perform the SimTraffic model validation. This site was selected since all data required to build the SimTraffic model was collected in the field and was available at the time of this analysis. Further, the MO-13 DCD site had been opened for approximately two years at the time of the data collection effort and it is assumed that drivers at this site are the most familiar with this type of interchange.

4.5.1 Calibration of SimTraffic Model

The input parameters utilized for calibration of the SimTraffic model were collected in the field, as described in section 3.3 of this thesis. Data items collected in the field that were used as input in SimTraffic include 15-minute traffic volumes shown in Figure 15, saturation flow rates shown in Table 1, and signal timing and phasing as illustrated in Table 19 below.

Synchro was used to optimize offsets and signal phasing at the adjacent intersections, since the signal timing data for Evergreen Street (Wal-Mart Entrance) and Norton Road was not collected in the field.

The offsets are defined for the northbound through phases at each of the intersections in the study area. The master intersection is the south crossover. For the intersection of the north crossover, the offsets are 85 seconds in the AM peak period and 5 seconds in the PM peak period. For the intersection of Evergreen Street (Wal-Mart Entrance), the offsets are 6 seconds in the AM peak period and 7 seconds in the PM peak period. For the intersection of Norton Road, the offsets are 5 seconds in the AM peak period and 15 seconds in the PM peak period.

Base free flow speed in the SimTraffic model was 35 mph, and lane geometry and distance between intersections in the SimTraffic model was the same as measured in the field. Therefore, the SimTraffic model was calibrated from the following data collected in the field: base free flow speed, lane geometry, intersection distance, saturation flow rates (only for crossovers), 15-minute traffic volumes, and signal timings (only for crossovers).

Table 19: Signal Timing and Phasing for MO-13

Through Movement	AM Peak				PM Peak			
	Green	Yellow	All-Red	Offset	Green	Yellow	All-Red	Offset
North Crossover SB	46	4	3	85	61	4	3	5
North Crossover NB	60	4	3		65	4	3	
South Crossover NB	26	4	3	0	53	4	3	0
South Crossover SB	80	4	3		73	4	3	

Once the model was built in SimTraffic, ten 1-hour simulations were recorded for each of the peak periods, preceded by a 15 minute seeding interval. However, during the AM peak period, four out of the ten recordings were discarded, since the SimTraffic model

produced unrealistically congested models. During the PM peak period, two of the recordings were considered valid for this analysis.

The unrealistically congested simulations appeared to be related to the unique geometry of the DCD interchange in the SimTraffic model, in which some vehicles stalled in the middle of the crossovers, blocking all other movements for the remaining of the simulation period. It appears that the SimTraffic DCD model produces long intersection areas at the crossovers that vehicles do not clear entirely during the yellow phase, and when a downstream queue does not allow them to clear the intersection, the entire system is blocked.

4.5.2 *Model Validation and Analysis*

The measure of effectiveness (MOE) used for validation of the SimTraffic model were origin-destination travel time, average control delay, and level of service, as defined in Chapter 22 of the Highway Capacity Manual 2011. These MOEs were also used in the evaluation of *after* conditions described previously. Refer to Appendix I for the SimTraffic analysis outputs.

Tables 20 and 21 illustrate the SimTraffic outputs and the MOEs as collected in the field, for the AM and PM peak periods, respectively. On the right side of the tables, the difference between SimTraffic outputs and field data is shown as total value and percent difference. A negative value indicates that the SimTraffic model produces a worse MOE than the actual field data.

As illustrated in Table 20, the SimTraffic model produces very similar MOEs for the south-to-north route during the AM and PM peak periods. This is the route with the largest sample size collected in the field. The north-to-south route for the SimTraffic model produces worse MOEs than those observed in the field, both in terms of travel time and average control delay, during both peak periods.

In the case of the left turn movements during the AM peak period, the travel time values are higher for the SimTraffic model in all routes, except for the south-to-west route. Average control delay is higher for all left turn routes in the SimTraffic model, although this may be related to the small sample size of the field data collected. Based on the methodology presented in the HCM 2011, SimTraffic produces LOS D or better for all routes, except for the north-to-south and north-to-east routes, although this is caused by the segment from Norton Road to the north crossover.

During the PM peak period described in Table 21, the south-to-west and west-to-north routes in the SimTraffic model have lower travel time than those collected in the field; while the north-to-east and east-to-south MOEs are higher for the SimTraffic model. Similarly to the AM peak period, control delay at the simulated left turn routes is higher than the field observed data; however, this is likely caused by the small sample size of the data collected. Based on the methodology presented in the HCM 2011, SimTraffic produces LOS D or better only for south-to-north, south-to-west, and west-to-north routes.

In summary, the SimTraffic model produces very similar results for only the south-to-north route during both peak periods. However, the north-to-south route in SimTraffic has higher MOEs than those observed in the field. For both of these routes the sample size is much higher than for the left turn routes.

For the left turn routes, the south-to-west route during both peak periods and the west-to-north during the PM peak period, have lower travel time values in SimTraffic when compared to field data collected. All other left turn routes had a higher travel time in SimTraffic. While the SimTraffic model averaged the travel time and control delay for all vehicles in a specific route, the field data collect only consisted in fewer than six runs for the left turn routes during both peak periods. Therefore, caution should be exercised when comparing the SimTraffic outputs to the left turn routes as collected in the field. It is recommended to increase the sample size for data collected in the field, and also to

compare the MOEs from SimTraffic and the field data to different traffic simulation software, such as VISSIM.

Table 20: Comparison of SimTraffic and Field Collected data for MO-13 DCD AM Peak

Route	SimTraffic AM Peak				Field Data AM Peak				Difference	% Diff.	Difference	% Diff.
	Avg. Travel Time (min)	Avg. Control Delay (s)	LOS	# Runs	Avg. Travel Time (min)	Avg. Control Delay (s)	LOS	# Runs	Avg. Travel Time (min)	Avg. Control Delay (s)		
South - North												
Evergreen St.-South Crossover (0.17 mi)	0.85	34			1.36	58.8		10	0.51	60.00%	24.8	72.94%
South Crossover-North Crossover (0.1 mi)	0.89	44.1			0.25	0		10	-0.64	-71.91%	-44.1	-100.00%
North Crossover-Norton Rd. (0.13 mi)	0.38	9.4			0.6	32.4		10	0.22	57.89%	23	244.68%
Total South - North (0.4 mi)	2.12	87.5	D	N/A	2.21	91.2	D	10	0.09	4.25%	3.7	4.23%
North - South												
Norton Rd. - North Crossover (0.13 mi)	1.95	102.8			0.41	24.6		9	-1.54	-78.97%	-78.2	-76.07%
North Crossover-South Crossover (0.1 mi)	0.22	25.2			0.38	30.6		9	0.16	72.73%	5.4	21.43%
South Crossover-Evergreen St. (0.17 mi)	0.55	15.8			0.64	26.4		9	0.09	16.36%	10.6	67.09%
Total North - South (0.4 mi)	2.72	143.8	F	N/A	1.43	81.6	D	9	-1.29	-47.43%	-62.2	-43.25%
South - West												
Evergreen St.-South Crossover (0.17 mi)	0.85	34			1.14	0		1	0.29	34.12%	-34	-100.00%
South Crossover-North Crossover (0.09 mi)	0.34	12.2			0.44	0		1	0.1	29.41%	-12.2	-100.00%
Total South - West (0.26 mi)	1.19	46.2	C	N/A	1.58	0	A	1	0.39	32.77%	-46.2	-100.00%
West - North												
I-44 Off-Ramp-South Crossover (0.15 mi)	0.42	10.5			0.25	0		1	-0.17	-40.48%	-10.5	-100.00%
South Crossover-North Crossover (0.08 mi)	0.83	42.4			0.12	0		1	-0.71	-85.54%	-42.4	-100.00%
North Crossover-Norton Rd. (0.13 mi)	0.38	9.4			0.38	0		1	0	0.00%	-9.4	-100.00%
Total West - North (0.36 mi)	1.63	62.3	C	N/A	0.75	0	A	1	-0.88	-53.99%	-62.3	-100.00%
North - East												
Norton Rd.-North Crossover (0.13 mi)	1.95	102.8			0.69	0		2	-1.26	-64.62%	-102.8	-100.00%
North Crossover-South Crossover (0.08 mi)	0.47	19.6			0.19	0		2	-0.28	-59.57%	-19.6	-100.00%
Total North - East (0.21 mi)	2.42	122.4	F	N/A	0.88	0	A	2	-1.54	-63.64%	-122.4	-100.00%
East - South												
I-44 Off-Ramp-North Crossover (0.13 mi)	0.95	44.5			0.47	22.2		1	-0.48	-50.53%	-22.3	-50.11%
North Crossover-South Crossover (0.08 mi)	0.31	11.5			0.14	0		1	-0.17	-54.84%	-11.5	-100.00%
South Crossover-Evergreen St. (0.17 mi)	0.55	15.8			0.95	0		1	0.4	72.73%	-15.8	-100.00%
Total East - South (0.38 mi)	1.81	71.8	D	N/A	1.56	22.2	B	1	-0.25	-13.81%	-49.6	-69.08%

Table 21: Comparison of SimTraffic and Field Collected data for MO-13 DCD PM Peak

Route	SimTraffic PM Peak				Field Data PM Peak				Difference	% Diff.	Difference	% Diff.
	Avg. Travel Time (min)	Avg. Control Delay (s)	LOS	# Runs	Avg. Travel Time (min)	Avg. Control Delay (s)	LOS	# Runs	Avg. Travel Time (min)	Avg. Control Delay (s)		
South - North												
Evergreen St.-South Crossover (0.17 mi)	1.05	46			1.12	52.8		13	0.07	6.67%	6.8	14.78%
South Crossover-North Crossover (0.1 mi)	0.39	14.3			0.29	7.8		13	-0.1	-25.64%	-6.5	-45.45%
North Crossover-Norton Rd. (0.13 mi)	0.49	15.4			0.47	18.6		13	-0.02	-4.08%	3.2	20.78%
Total South - North (0.4 mi)	1.93	75.7	D	N/A	1.88	79.2	D	13	-0.05	-2.59%	3.5	4.62%
North - South												
Norton Rd. - North Crossover (0.13 mi)	1.73	89.3			0.69	31.8		13	-1.04	-60.12%	-57.5	-64.39%
North Crossover-South Crossover (0.1 mi)	0.82	39			0.28	0		13	-0.54	-65.85%	-39	-100.00%
South Crossover-Evergreen St. (0.17 mi)	0.36	10.1			0.42	5.4		13	0.06	16.67%	-4.7	-46.53%
Total North - South (0.4 mi)	2.91	138.4	F	N/A	1.39	37.2	C	13	-1.52	-52.23%	-101.2	-73.12%
South - West												
Evergreen St.-South Crossover (0.17 mi)	1.05	46			1.44	0		3	0.39	37.14%	-46	-100.00%
South Crossover-North Crossover (0.09 mi)	0.24	6.4			0.27	0		3	0.03	12.50%	-6.4	-100.00%
Total South - West (0.26 mi)	1.29	52.4	C	N/A	1.71	0	A	6	0.42	32.56%	-52.4	-100.00%
West - North												
I-44 Off-Ramp-South Crossover (0.15 mi)	0.53	17			0.25	6.6		6	-0.28	-52.83%	-10.4	-61.18%
South Crossover-North Crossover (0.08 mi)	0.32	12.3			0.41	0		6	0.09	28.13%	-12.3	-100.00%
North Crossover-Norton Rd. (0.13 mi)	0.49	15.4			1.35	0		6	0.86	175.51%	-15.4	-100.00%
Total West - North (0.36 mi)	1.34	44.7	B	N/A	2.01	6.6	A	6	0.67	50.00%	-38.1	-85.23%
North - East												
Norton Rd.-North Crossover (0.13 mi)	1.73	89.3			0.59	0		5	-1.14	-65.90%	-89.3	-100.00%
North Crossover-South Crossover (0.08 mi)	0.51	21.7			0.29	0		5	-0.22	-43.14%	-21.7	-100.00%
Total North - East (0.21 mi)	2.24	111	E	N/A	0.88	0	A	5	-1.36	-60.71%	-111	-100.00%
East - South												
I-44 Off-Ramp-North Crossover (0.13 mi)	1.08	52.7			0.28	0		4	-0.8	-74.07%	-52.7	-100.00%
North Crossover-South Crossover (0.08 mi)	1.11	59.5			0.61	0		4	-0.5	-45.05%	-59.5	-100.00%
South Crossover-Evergreen St. (0.17 mi)	0.36	10.1			0.48	0		4	0.12	33.33%	-10.1	-100.00%
Total East - South (0.38 mi)	2.55	122.3	E	N/A	1.37	0	A	4	-1.18	-46.27%	-122.3	-100.00%

5.0 DOUBLE CROSSOVER DIAMOND CAPACITY GUIDANCE

This section presents the contents of a paper prepared concurrently with the research project described in the previous sections. This section will focus on the body of the paper and sections already described previously in this thesis will be omitted.

5.1 Introduction

Signal timing and capacity calculations are an important part of the DCD operational research being conducted. It is the objective of this paper to analyze the queue interaction, movement capacities, and signal spacing at each of the DCD interchange crossovers.

Queue interaction and movement capacities at closely spaced intersections, such as conventional diamond interchanges, have been analyzed in previous research. However, due to innovations in interchange design, previous analysis models need to be adapted to new signal and lane configurations. This paper will analyze the case of double crossover diamond (DCD) interchanges (also known as diverging diamond interchanges - DDI), in terms of queue interaction, movement capacities, and signal spacing. Previous analysis models were limited to a single upstream traffic approach and negligible midblock traffic generation/absorption. However, a DCD has two upstream approaches (one of which may or may not be signalized) and considerable midblock traffic absorption. The model presented in this paper took these factors into account and expanded previous models to the case of DCD interchanges. Finally, the model was applied to an existing DCD interchange in Maryland Heights, Missouri.

5.2 Literature Review

Rouphail and Akçelik (11) introduced in 1992 a model for analyzing queue interaction at paired intersections. This model utilizes an iterative method to calculate a reduced saturation flow rate at the upstream intersection, when downstream queues prevent

discharge of the upstream intersection at its full saturation flow rate. The iterative process is applied until an equilibrium state is achieved between the length of the downstream queue and the discharge characteristics of the upstream platoon. This model was limited to one upstream and one downstream traffic stream. Further, the effective green period of the upstream signal is not changed.

Prosser and Dunne (12) continued research on queue interaction and presented a graphical model to calculate movement capacities by reducing the effective green at the upstream approach, when a downstream queue interferes with the upstream traffic discharge. This model allows for multiple upstream traffic streams, and it is the model most closely applicable to the DCD interchange case. This paper will utilize the Prosser and Dunne model as the basis for the DCD interchange queue interaction model.

Xu, Liu and Tian (8) developed a method to calculate control delay at DCD interchanges using a newly developed analytical model. The model was first created to calculate control delay of external movements at conventional diamond interchanges, while later added a function to calculate to calculate delay of internal movements under a DCD configuration.

Finally, the recently published Highway Capacity Manual 2010 (16) introduced a method in the Interchange Ramp Terminals chapter to calculate lost time at closely spaced intersections. The methodology takes into consideration the duration of common green times between various phases at the two intersections.

5.3 Paired Intersections

Closely spaced paired intersections are defined by Rouphail and Akçelik (11) as a system where:

- Upstream arrivals follow a random (Poisson) distribution, and are unimpeded prior to reaching the upstream approach,

- Downstream arrivals occur primarily in platoons, which are formed due to queuing at the upstream approach,
- Platoon dispersion is negligible, due to the close proximity of the two intersections,
- Midblock generation/absorption is considered negligible,
- Limited number of vehicles are allowed to queue at the downstream approach, due to proximity of the two intersections. When the downstream demand exceeds the queuing capacity, blockage occur at the upstream approach.

Queue interaction exists when, under a combination of queue space, demand level and signal control parameters, the upstream approach experiences a reduction in saturation flow rate, effective green time, and platoon speed due to the blocking downstream queue.

5.3.1 Queue Interaction at DCD Interchanges

Signalized intersections at DCD interchanges generally follow the same characteristics described in the previous section for paired intersections. The main difference is that midblock absorption could be significant due to the left turn to the freeway. Also, arrivals at the upstream through approach could be random or in platoons, depending on the location of the closest signalized intersection.

As described before, queue interaction occurs when the upstream approach experiences a reduction in saturation flow rate, effective green time and platoon speed, due to the presence of a blocking downstream queue. In the case of a DCD interchange, two upstream movements will be affected by the downstream blocking queue: the upstream through movement from the surface street, and the upstream left turn movement from the freeway off-ramp. The size of the blocking queue will be smaller for the upstream off-ramp left turn, since the left turn stop bar is located closer to the downstream stop bar.

of the queue is denoted t_q . The intersignal travel time is denoted as t_f , and any vehicle which discharges from the upstream approach after a time t_2 , equal to t_f seconds before the end of the downstream green period, is going to stop at the downstream stop bar. Vehicles are allowed to discharge from the upstream approach after t_2 , until the maximum number of vehicles in the intersignal queue is reached, denoted N_{max} . The end of the clear period is denoted t_3 . Refer to Figure 31 for illustration of the clear period at a paired intersection. For this example, the intersignal storage capacity for movement 1 is 24 vehicles, the common cycle time is 120 seconds and the offset between the upstream and downstream signals is 20 seconds.

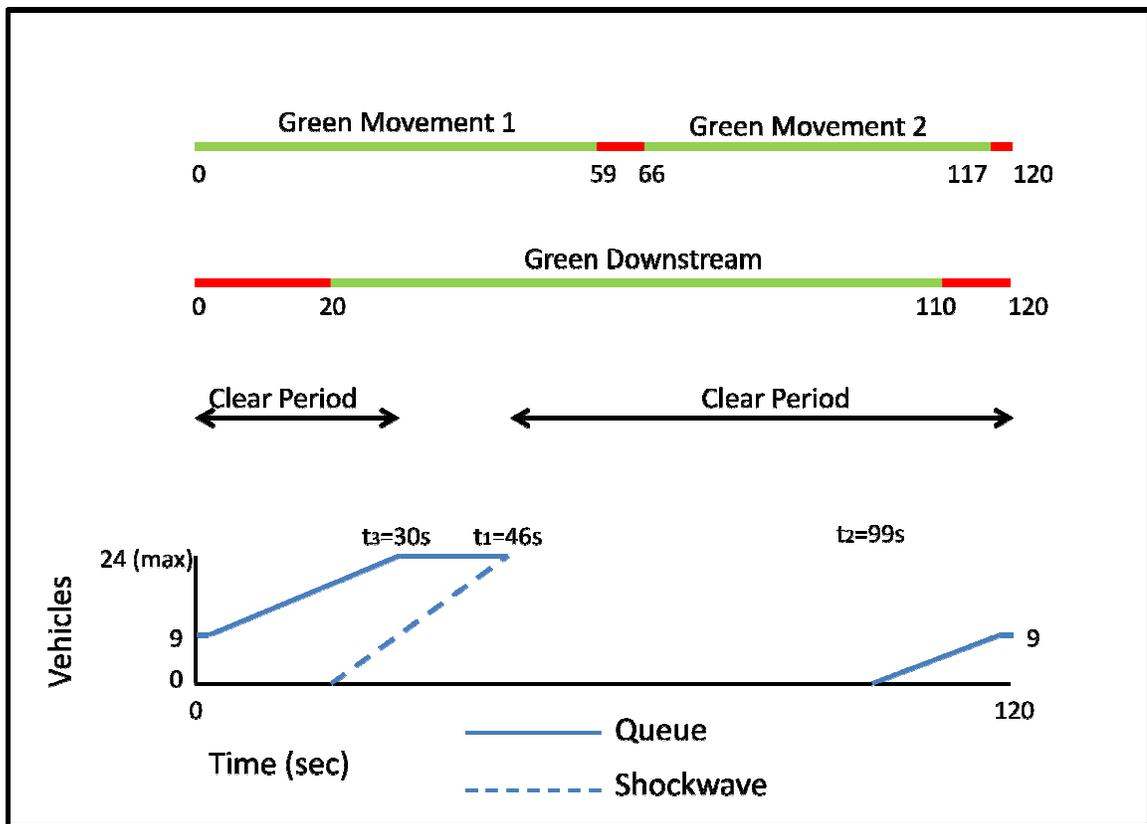


Figure 31: Clear Period Illustration

While the Prosser and Dunne model assumes equal intersignal travel time t_f , and maximum intersignal queue N_{max} for both upstream approaches; in the case of DCD interchanges, each of the upstream approaches could have a different intersignal travel

time t_f , and maximum intersignal queue N_{max} , since the length of the intersignal block will always be shorter for the upstream left turn off-ramp.

5.4 Capacity Analysis From Queue Interaction

The capacity analysis model for DCD interchanges will be based on the model developed by Prosser and Dunne (12), adapted to the lane geometry of DCD interchanges. This model considers two upstream traffic streams discharging to a common downstream approach. In the case of a DCD interchange, one of the upstream approaches will be a through movement *into* the DCD, denoted movement 1; and the second upstream approach will be a left turn off-ramp *into* the DCD, denoted as movement 2. The downstream approach will always be a through movement *out* of the DCD, denoted as movement d .

While the left turn off-ramp could operate under signalized or unsignalized conditions, this paper will consider this movement to be signalized under all conditions. Also, the off-ramp left turn movement from the upstream intersection will not be considered to make a “U-Turn” (making a new left turn on-ramp at the downstream intersection). Further, the upstream through movement will be assumed to utilize an exclusive leftmost lane when making a left turn on-ramp prior to the downstream signal, therefore not being included in the queue interaction analysis. Further, the length of the intersignal segment L_i , will be always shorter for movement 2, due to geometric characteristics of DCD interchanges.

5.4.1 Capacity Analysis Parameters

To analyze the capacity for upstream movements 1 and 2 from queue interaction, the following input parameters will be defined:

V_i : Volume for upstream movement i (vph)

PHF: Peak hour factor for upstream intersection

- s_i : Saturation flow rate for upstream movement i (veh/sec)
- s_d : Saturation flow rate for downstream movement (veh/sec)
- α_i : Start lost for upstream movement i (sec)
- β_i : End gain for upstream movement i (sec)
- L_i : Length of intersignal segment i (ft)
- C : Common cycle time (sec)
- G_i : Green period for movement i (sec)
- O : Offset between upstream and downstream signals (sec) (based on arterial through movements)
- O_u : Offset between upstream movement 1 and 2 signals (sec)
- Y_i : Yellow time for movement i (sec)
- R_i : All-red time for movement i (sec)
- N_{li} : Number of lanes for upstream movement i (lanes)
- L_j : Vehicle spacing at rest (ft)

Alternatively, vehicle spacing at rest can be calculated using the car spacing at rest, the heavy vehicle spacing at rest, and percentage of heavy traffic:

L_c : Car spacing at rest (ft)

L_{hv} : Heavy vehicle spacing at rest (ft)

$\%_{hv}$: Percentage of heavy traffic

Therefore,
$$L_j = \left(1 - \frac{\%_{hv}}{100}\right) L_c + \frac{\%_{hv}}{100} L_{hv}$$

From the previous input parameters, the following parameters are calculated:

q_i : Arrival flow rate for upstream movement i (veh/sec) = $\frac{V_i}{PHF * 3600 * N_{li}}$

g_i : Effective green period for movement i (sec) = $G_i - \alpha_i + \beta_i$

t_i^s : Saturation flow period for movement i (sec) = $\frac{q_i(C-g_i)}{s_i-q_i}$

K_j : Jam concentration (veh/ft) = $\frac{1}{L_j}$

K_m : Saturation flow concentration (veh/ft) = $\frac{K_j}{2}$ (Based on the model developed by Greenshields (17))

L_q : Length of blocking queue (veh)

t_q : Blocking queue clearing time (sec) = $L_q \frac{(K_j-K_m)}{s_d}$

t_{fi} : Intersignal travel time for movement i (sec) = $K_m \frac{L_i}{s_d}$

$N_{\max i}$: Intersignal storage capacity for upstream movement i (veh) = $L_i K_j$

t_1 : Time at which clear period begins (sec)

t_2 : Time at which blocking queue begins to form (sec)

t_3 : Time at which clear period ends

5.4.2 Capacity Analysis Methodology

Determining capacity of the upstream movements from queue interaction by the method proposed in this paper, could be reached by applying a graphical method, as described by Prosser and Dunne (12), or it could be programmed into an Excel spreadsheet or other similar software. The steps of this methodology are described below:

Step 1: Determine the parameters of the signal timing scenario to be analyzed.

This includes the upstream and downstream green periods G_i , start lost for upstream movement i α_i , end gain for upstream movement i β_i , offset between upstream and downstream signals O , offset between upstream movement 1 and 2 signals O_u , yellow time for movement i Y_i , all-red time for movement i R_i , and common cycle time C . The offset between upstream movement 1 and 2 signals O_u is necessary since a significant

distance could be present between movement 1 and 2 stop bars. This offset O_u could be incorporated into the yellow plus all-red period $Y+AR_j$.

Step 2: Determine the saturation flow rate for each of the upstream movements s_i , and the saturation flow rate for the downstream movement s_d .

Step 3: Calculate the intersignal storage capacity for each of the upstream movements by the formula $N_{\max i} = L_i \times K_j$. The length of intersignal segment L_i could be obtained by field measurement or from design, while the jam concentration K_j could be obtained by the formula presented in the previous section.

Step 4: Calculate the time at which blocking queue begins to form by the formula

$t_2 = 0 + G_d - t_{fi}$. The intersignal travel time t_{fi} could be calculated by the formula described in the previous section or by field measurement. If the end of green for the downstream movement occurs before the beginning of the upstream green for movement 2, the intersignal travel time for movement 1 will be utilized; otherwise, the intersignal travel time for movement 2 will be utilized. The intersignal travel time will always be shorter for the upstream left turn movement.

Step 5: Calculate the number of vehicles stopped at the downstream signal at the end of the upstream green phase for which t_2 applies. This could be calculated by the formula $N_{\text{veh } i} = [(End \text{ of } G_i + \beta_i) - t_2] \times s_i$. When $N_{\text{veh } i} = N_{\max i}$, blockage has occurred, marking the end of the clear period, denoted t_3 (note that $N_{\text{veh } i} > N_{\max i}$ can never occur). Three special cases apply:

1. If $N_{\text{veh } 2} < N_{\max 2}$, the number of vehicles stopped will carry over to the beginning of green for movement 1. At this stage, $N_{\max 1} - N_{\text{veh}2}$

vehicles will be able to discharge from upstream movement 1, until blockage occurs.

2. If $N_{veh\ 2} = N_{max\ 2}$, blockage has occurred for movement 2, but not for movement 1. This case is the same as case 1, $N_{max\ 1} - N_{max\ 2}$ vehicles will be able to discharge from upstream movement 1, until blockage occurs.
3. If $N_{veh\ 1} \geq N_{max\ 2}$, blockage has occurred for movement 2.

Step 6: Calculate the time at which the clear period begins by the formula $t_1 = 0 + t_q$. The blocking queue clearing time t_q , could be calculated by the formula described in the previous section. Note that when applying t_q for movement 2, the length of blocking queue L_q can be longer than $N_{max\ 2}$. In that case, L_q must be calculated by the formula $L_q' = L_q + (L_q - N_{max\ 2})$.

Step 7: Once t_1 , t_2 , and t_3 have been determined, the clear period length can be determined from t_1 to t_3 .

Step 8: Calculate the effective green g_i for each of the upstream movements. The effective green will equal the time when an upstream green phase is concurrent with the clear period. Start loss α_i and end gain β_i should also be considered for each of the green phases. Further, at the beginning of the clear period, a start loss α_i should also be applied to the corresponding upstream green phase. No end gain will occur at the end of the clear period, since it is related to blockage, not signal phasing.

Step 9: Calculate the vehicle hourly capacity for each of the upstream movements by the formula $\frac{s_i \times 3600 \times g_i \times N_{it}}{c}$.

5.4.3 *Limitations of This Methodology*

The methodology described in the previous section assumes that no interfering queue will be present downstream of the DCD downstream intersection. In other words, the DCD downstream intersection through movement will experience no blocking during the entire cycle length. However, field data has suggested that queue interaction will also occur between a crossover signal at a DCD interchange and the adjacent signals at nearby intersections.

Another limitation in this methodology is that movement 2 is always considered to be signalized. While this is common in several DCD interchanges constructed in the United States, this is not always the norm, such as in the Alcoa, TN DCD interchange. When movement 2 (left turn off-ramp) operates under unsignalized conditions, the queue interaction methodology must take into account the random vehicle arrivals that will be added to the downstream queue when applicable.

Also, the methodology presented in this paper has omitted vehicles traveling through the upstream intersection but making a left turn at the on-ramp before the downstream intersection. In some instances, this assumption is valid, as long as the left turn on-ramp has an exclusive turn lane; however, this is not always the case, such as in the Alcoa, TN DCD interchange also. In the latter case, the blocking queue will also include vehicles that will make a left turn before the downstream intersection.

Finally, this model assumes oversaturated conditions for both upstream movements. However, if the duration of the saturation flow period is less than the corresponding green phase, it is possible that the upstream movement being served will not discharge any vehicles during parts of its effective green time. This situation will reduce the length of the downstream queue. In this case, the methodology provided in this paper assumes a conservative scenario for movement capacity calculations.

5.5 Application of the Model to the Maryland Heights, MO DCD Interchange

5.5.1 Site Description

The Maryland Heights DCD interchange is located at the intersection of Dorsett Road and I-270, in Maryland Heights, Missouri. This DCD interchange opened during October 2010, and was one of the first five DCD interchanges to be constructed in the United States. The surrounding area consists of residential neighborhoods to the west and commercial and retail areas to the east. Figure 32 for illustrates of the general layout of the Maryland Heights DCD interchange.

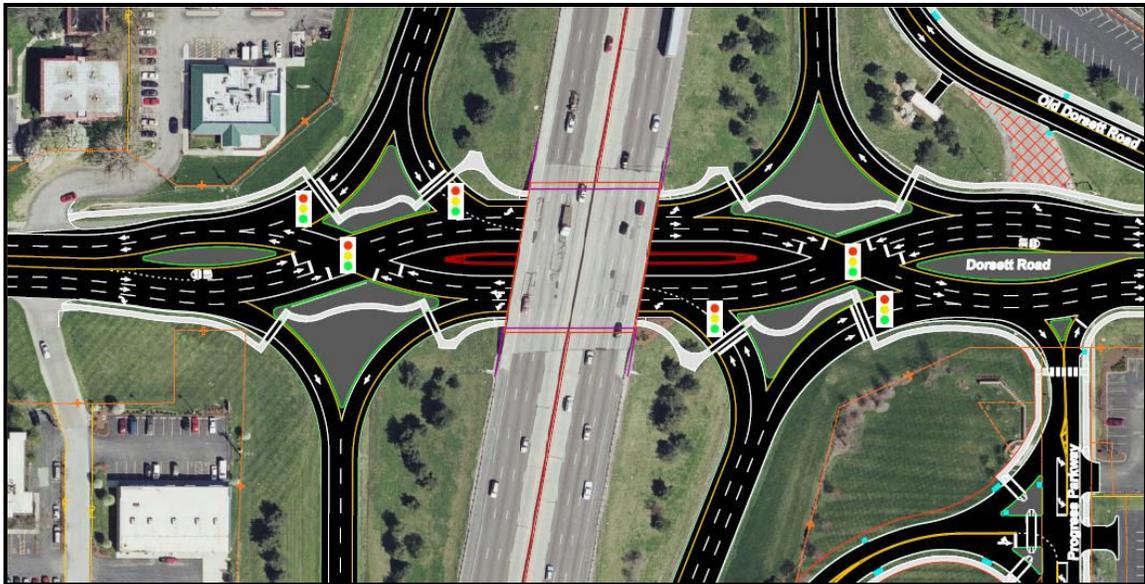


Figure 32: DCD at Maryland Heights, MO

5.5.2 Analysis Parameters

This paper analyzed both crossovers. Movement 1 will be defined as the through movement at the upstream approach, and movement 2 will be defined as the left turn off-ramp at the upstream approach. The downstream approach corresponds to the through movement at the downstream crossover.

As previously explained, the leftmost through lane on both directions along Dorsett Road will be considered independent from the queue interaction analysis. Also, left turn off-

ramp traffic making a “U-Turn” at the downstream left turn on-ramp, will be considered negligible. Finally, the left turn off-ramps are signalized, as shown in Figure 19.

The author of this paper was part of a research team that visited the site in August 2011, and collected traffic volumes, saturation flow rates, and travel times at all approaches. The time period under study will be the AM peak hour. A summary of the input parameters needed for the DCD capacity analysis is summarized in Tables 22 and 23.

Signal timing parameters for the west and east crossovers were calculated for this paper using the methodology described in the Highway Capacity Manual 2000, Chapter 16, Appendix B Signal Timing Design (18).

5.5.3 Analysis Results and Sensitivity Analysis

The capacity analysis results were obtained by utilizing the methodology described in section 5.4.2 of this report for the eastbound and westbound directions. In addition to analyzing capacity for the conditions given in Tables 22 and 23, a sensitivity analysis of the offset between through movements was performed to evaluate the different conditions of queue interaction.

Tables 24 and 24 summarize the analysis results for 10-second offset increments. Also, results for volume to capacity (v/c) ratios at the different scenarios are provided. Figure 33 and Figure 34 present graphical representations of the analysis results from Tables 24 and 25.

The capacity analysis results show that an offset equal 0 seconds produces the maximum capacity for both crossovers combined, while an offset of 40 seconds produces the minimum capacity for both crossovers combined. Also, at offsets between 0 and 40 seconds, the v/c ratio for movement 2 at the west crossover is under 1. Offsets of between 40 seconds and 110 seconds provide capacity/c ratios over 1 for movement 2 at the west crossover. All other movements have a v/c ratio under 1 for all offset scenarios.

Field data indicated that the offset between crossovers during the AM peak hour is approximately 20 seconds.

Table 22: Input Parameters West Crossover

AM Peak Hour	Upstream 1 (EBT)	Upstream 2 (SBL)	Downstream (EBT)
Volume (vph)	386	1091	1477
Sat. Flow Rate (vphpl)	1823	1795	1676
N. Lanes (lanes)	2	2	2
PHF	0.91	0.91	0.89
Green Period (sec)	59	51	90
Start Loss (sec)	2	2	2
End Gain (sec)	1	1	1
Yellow + All-Red (sec)	3	3	3
Offset 1 and 2 (sec)	N/A	4	N/A
Common Cycle Time (sec)	120	120	120
Length of intersignal Segment (ft)	510	365	N/A
Intersignal Storage Capacity (veh)	24	17	N/A
Intersignal Travel Time (sec)	12	9	N/A

Table 23: Input Parameters East Crossover

AM Peak Hour	Upstream 1 (WBT)	Upstream 2 (NBL)	Downstream (WBT)
Volume (vph)	194	164	358
Sat. Flow Rate (vphpl)	1851	1883	1833
N. Lanes (lanes)	2	2	2
PHF	0.89	0.89	0.91
Green Period (sec)	24	86	55
Start Loss (sec)	2	2	2
End Gain (sec)	1	1	1
Yellow + All-Red (sec)	3	3	3
Offset 1 and 2 (sec)	N/A	4	N/A
Common Cycle Time (sec)	120	120	120
Length of intersignal Segment (ft)	505	323	N/A
Intersignal Storage Capacity (veh)	24	15	N/A
Intersignal Travel Time (sec)	12	8	N/A

Table 24: Analysis Results West Crossover

Offset (sec)	Capacity 1 (vph)	Capacity 2 (vph)	Total Capacity (vph)	V/C ₁	V/C ₂
0	1337	1466	2803	0.289	0.744
10	1337	1496	2833	0.289	0.729
20	1337	1496	2833	0.289	0.729
30	1337	1496	2833	0.289	0.729
40	1398	1286	2684	0.276	0.848
50	1701	987	2688	0.227	1.105
60	1762	957	2719	0.219	1.140
70	1762	927	2689	0.219	1.177
80	1762	927	2689	0.219	1.177
90	1762	927	2689	0.219	1.177
100	1762	1017	2779	0.219	1.073
110	1580	1137	2717	0.244	0.960

Table 25: Analysis Results East Crossover

Offset (sec)	Capacity 1 (vph)	Capacity 2 (vph)	Total Capacity (vph)	V/C ₁	V/C ₂
0	494	1381	1875	0.393	0.119
10	710	1067	1777	0.273	0.154
20	710	942	1652	0.273	0.174
30	710	942	1652	0.273	0.174
40	710	973	1683	0.273	0.169
50	710	1098	1808	0.273	0.149
60	710	1193	1903	0.273	0.137
70	710	1193	1903	0.273	0.137
80	494	1381	1875	0.393	0.119
90	494	1349	1843	0.393	0.122
100	494	1349	1843	0.393	0.122
110	494	1349	1843	0.393	0.122

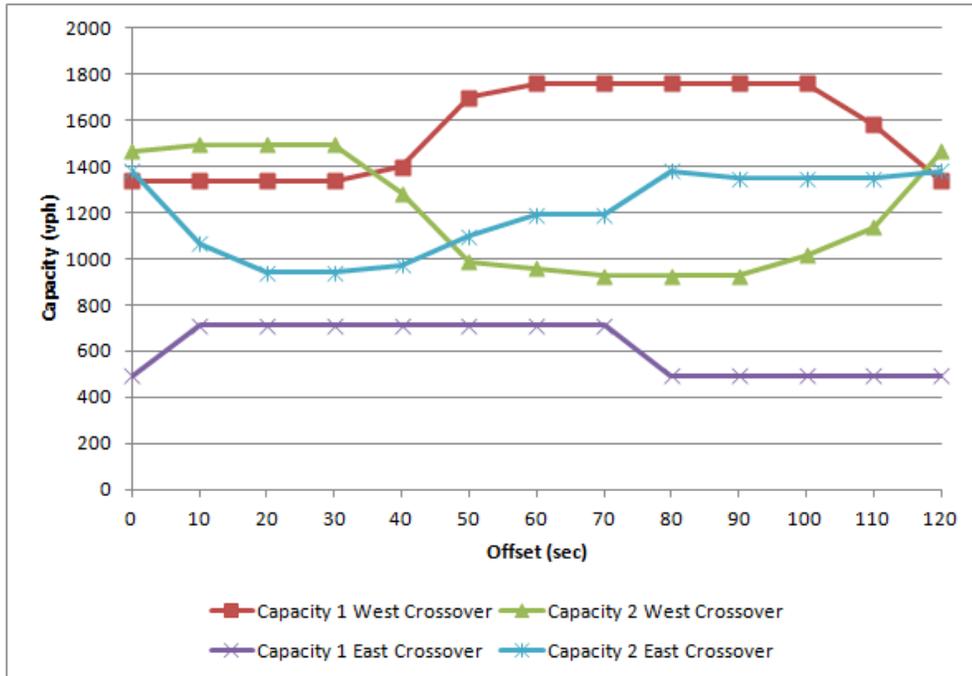


Figure 33: Sensitivity Analysis of Movement Capacities Based on Offset Modification

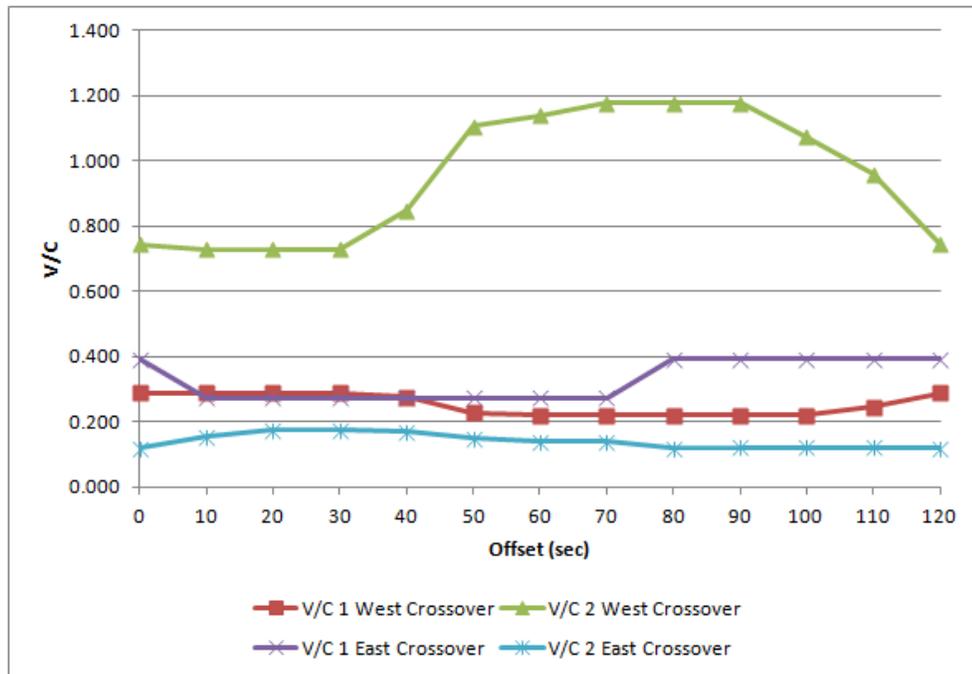


Figure 34: Sensitivity Analysis of Movement V/C Ratios Based on Offset Modification

5.6 Conclusions of the DCD Capacity Guidance Model

This section presented a methodology to calculate movement capacity at DCD interchanges while taking into account queue interaction. While previous research has analyzed paired intersections and the effect that interfering queues have on the capacity of their approaches, the unique geometry of DCD interchanges required a variation of previous models to this type of interchange design.

A step by step methodology was presented, and its input parameters were described. The analysis method could be performed either by graphical representation or by utilizing an Excel spreadsheet or other similar software.

Limitations of this method include the assumption that the downstream approach will not be affected by any other queue outside of the DCD interchange. Also, the upstream left turn off-ramp movement was assumed to be signalized under all cases; however, this is not always the case. Further, traffic making a left turn onto the on-ramp prior to the downstream intersection was assumed to utilize a separate lane and not interfering with the blocking queue. Also, both upstream approaches were assumed to operate under oversaturated conditions, overestimating the actual queues that could be present at the downstream approach.

Finally, this paper presented an example of the methodology described for a DCD interchange recently constructed in Maryland Heights, Missouri. An analysis of different offset values and their effect on the capacity of the upstream approaches was provided. The capacity analysis results show that an the ideal offset is 0 seconds, since it produces the maximum capacity for both crossovers combined, and the second lowest average v/c ratio. Also, at offsets between 0 and 40 seconds, the v/c ratio for movement 2 at the west crossover is under 1. Offsets of between 40 seconds and 110 seconds provide capacity/c ratios over 1 for movement 2 at the west crossover. All other movements have a v/c ratio under 1 for all offset scenarios. Field data indicated that the offset between crossovers during the AM peak hour is approximately 20 seconds. This is within the offset range of

0 to 40 seconds, which produces high capacity and vehicle-to-capacity (v/c) ratios under 1 for all movements.

6.0 CONCLUSIONS

6.1 Research

This project investigated the operational impacts of four Double Crossover Diamond Interchanges (DCD- also known as “Diverging Diamond Interchange – DDI”) recently built in the United States, as part of a project commissioned by the Federal Highway Administration (FHWA). The DCD interchanges under study are located at US Route 129 and Bessemer Street in Alcoa, TN; I-44 and MO-13 in Springfield, MO; US Route 60 and National Avenue in Springfield, MO; and I-270 and Dorsett Road in Maryland Heights, MO. Also, this thesis presented a model to estimate capacity at DCD interchanges based on queue interaction.

While the DCD concept originated in France in the 1970s, this type of interchange started to be studied in the early 2000s by Chlewicki and others (2). The I-44 and MO-13 DCD interchange in Springfield, MO, was the first of its type in the United States, opened in June 2009. The Alcoa, TN DCD interchange was one of the latest to be constructed in the United States, at the beginning of this research project, opened in January 2011. At the time of the writing of this thesis, there were a total of six DCDs operating in the United States, and several others in late stages of construction.

The main part of this thesis compared *transition* to *after* data in the case of the Alcoa DCD, and evaluated *after* data in the MO-13, National Avenue, and Dorsett Road DCD interchanges. Also, a calibration and validation of a SimTraffic DCD model was included. *Transition* period is defined as within two weeks of opening at the DCD site under the new traffic configuration. The *after* period is defined between three and six months of opening at the DCD site under the new traffic configuration.

Of the four interchanges analyzed in this thesis, the Alcoa DCD was the only one where *transition* data were collected. *Before* data collection was not possible at any sites, since

at the time of the beginning of the project, all interchanges had been constructed. Similarly, *transition* data collection was not possible at the MO-13, National Avenue and Dorsett Road DCDs either. *After* data were collected at all sites, and included peak period queue lengths, origin-destination travel times, saturation flow rates, erratic maneuvers, and field observed conflicts. From the origin-destination travel time raw data, it was possible to obtain average control delay and average number of stops at each approach. The methodology described in the Highway Capacity Manual 2010, Chapter 22, was utilized to calculate level of service.

The *transition* to *after* analysis of average travel time, average control delay and average number of stops for Alcoa, TN DCD, showed that in most of the routes and peak periods under study, traffic operation performance improved after six months of the opening of the DCD interchange, even after traffic volumes increased during the same period. Further, level of service indicated that for both scenarios, the interchange operated at LOS D or LOS C for the arterial through movements, and at LOS C or better for the left turn movements. It should be noted that the sample size for the left turn routes was small, and analysis of the operational results for the left turn movements should be taken with caution.

The peak period queues also showed an improvement, although less significant than the other MOEs. In terms of conflicts and erratic maneuvers, there was a significant decrease in total conflicts per hour and erratic maneuvers between *transition* and *after* conditions. However, different observers were involved in the conflict data collection, and defining a conflict or an erratic maneuver involves a considerable amount of engineering judgment, that could possibly vary between observers. Overall, the Alcoa DCD site operated under acceptable traffic conditions during the *transition* and *after* periods.

The *after* analysis of the I-44 and MO-13 DCD interchange in Springfield, MO shows that for the arterial through movements, priority is given to the direction with the heaviest traffic volumes, which results in lower travel times for the prioritized direction of traffic.

Also, progression is achieved at the DCD for through the second crossover for many drivers making the arterial through movements. The arterial through movements operate at LOS D during both peak periods, except for the north-to-south during the PM peak period, which operates at level of service C. The left turn routes operate at LOS A or LOS B for both peak periods. However, the sample size for the left turn routes is very small and any conclusions on its operation should be made with caution.

Peak period queues did not block upstream intersections, except for the maximum queue observed at the north crossover in the southbound direction during the AM peak period. However, the 95th percentile queue for this movement does not block the upstream intersection. Conflict and erratic maneuver analysis of *after* conditions indicate that while both crossovers had approximately the same number of events per hour, the north crossover had a slightly higher rate of erratic maneuvers per hour, and the south crossover had a higher rate of conflicts per hour, likely due to the merge conflict created by off-ramp right turn vehicles. Overall, the DCD interchange at MO-13 operates at acceptable conditions based on analysis of the MOEs presented in this section.

The *after* analysis of the US Route 60 and National Avenue DCD interchange in Springfield, MO shows that for the arterial through movements, the south-to-north movement experience worse MOEs during both analysis periods. Also, once a vehicle enters the DCD, it will progress through the next crossover without stopping. The north-to-south through movement operates at LOS B during both peak periods, and the south-to-north route operates at LOS C during both peak periods.. All left turn movements operate at LOS D or better during both peak periods, except for the west-to-north route during the PM peak period, which operates at LOS E. Caution should be exercised when interpreting these results, due to the small sample size of the left turn movements.

Peak period queues do not block upstream intersections. Conflict and erratic maneuver analysis of *after* conditions indicate that the north crossover has a higher rate of total events and erratic maneuvers, while the south crossover has a higher rate of conflicts per

hour. Overall, the National Avenue DCD site showed good operational performance for the arterial through movements and for the left turn movements. However, small sample size for the left turn should be taken into consideration when analyzing the data.

The *after* analysis of the I-270 and Dorsett Road DCD interchange in Maryland Heights, MO shows that for the arterial through movements, the east-to-west route has a shorter travel time, and average number of stops when compared to the west-to-east route, during both peak periods. However, the east-to-west route has a higher average control delay, when compared to the west-to-east route. Also, once a vehicle enters the DCD, it will progress through the next crossover without stopping. The arterial through movements operate at LOS C during the AM peak period. During the PM peak period, the arterial through movements operate at LOS E. The left turn routes operate at level of service D or better during both peak periods. However, the sample size for the left turn routes is very small and any conclusions on its operation should be taken with caution.

Peak period queues do not block upstream intersections. Conflict and erratic maneuver analysis of *after* conditions indicate that the east crossover has a higher rate of total events and conflicts per hour, while the west crossover has a higher rate of erratic maneuvers per hour. Overall, the Dorsett Road DCD site showed acceptable operational performance during the AM peak period for all movements; while the arterial through movements operated at excessive level of service during the PM peak hour.

Analysis of the SimTraffic DCD model indicates that the simulation produces very similar results to field data collected only for the south-to-north route during both peak periods. However, the north-to-south route in SimTraffic has higher MOEs than those observed in the field. For both of these routes the sample size is much higher than for the left turn routes. Caution should be exercised when comparing the SimTraffic outputs to the left turn routes as collected in the field. Also, the SimTraffic model created unrealistically congested conditions that appeared to be related to the unique geometry of the DCD interchange. In the simulation, some vehicles stalled in the middle of the

crossovers, blocking all other movements for the remaining of the simulation period. It appears that the SimTraffic DCD model produces long intersection areas at the crossovers that vehicles do not clear entirely during the yellow phase, and when a downstream queue does not allow them to clear the intersection, the entire system is blocked.

Finally, the DCD capacity guidance chapter showed a model for estimating capacity at a DCD interchange based on queue interaction. This model was based on previous research that studied queue interaction and capacity at closely spaced intersections; however, the DCD presents some unique characteristics and an adapted model was developed. A step by step methodology was presented, and its input parameters were described. The analysis method could be performed either by graphical representation or by utilizing an Excel spreadsheet or other similar software. Finally, this paper presented an example of the methodology described for the DCD interchange recently constructed in Maryland Heights, Missouri. An analysis of different offset values and their effect on the capacity of the upstream approaches was provided. It was found that maximum capacity for both crossovers is achieved at an offset of 0 seconds; while acceptable v/c ratios for all movements occur at offsets between 0 seconds and 30 seconds approximately.

6.2 Recommendations

Agencies should continue investigating viable sites to install DCD interchanges. This type of interchange has shown that it can handle left turns better than a traditional diamond interchange, as well as providing opportunity for progression along the arterial through movements. However, it should be noted that as shown in the Alcoa DCD site, even when traffic volumes are low, emphasis should be given to signal timing design, otherwise progression could be impacted.

Double Crossover Diamond interchanges present a series of benefits to traditional diamond interchanges where left turns are heavy. Limited conflict data in this report also

showed that drivers adapt to the DCD positively, reducing the number of conflicts and erratic maneuvers significantly in an approximately six-month period.

6.3 Future Research Needs

While this thesis presented limited data on conflicts and erratic maneuvers at DCD interchanges, future research should expand the safety assessment of this type of interchange. While no major safety issues were observed in the field, unfamiliarity with this type of design during the *transition* period in Alcoa, TN created a large number of erratic maneuvers. Also, Alcoa, TN *transition* field data showed that access management conditions should be taken into consideration when designing a DCD interchange and the nearby roadway infrastructure.

More research is needed with regards to signal timing at DCD interchanges. As with any type of interchanges, poor signal design could offset any positive impact that the DCD may offer to a specific site. Also, simulation of DCD interchanges should be expanded to other software, in addition to the one provided in this thesis. Concurrently with this thesis, a FHWA commissioned study is underway at the Institute for Transportation Research and Education at North Carolina State University (ITRE), that will include a *before* scenario simulation of each DCD in VISSIM, which is considered a more robust traffic simulation software than SimTraffic.

Walkability and bikeability assessments of DCD interchanges should be taken into consideration for future research, since they were outside of the scope of this thesis. Geometric design considerations were also not included in this thesis, but future research in this area is also needed, since currently there are no uniform guidelines on the geometric design of DCD interchanges.

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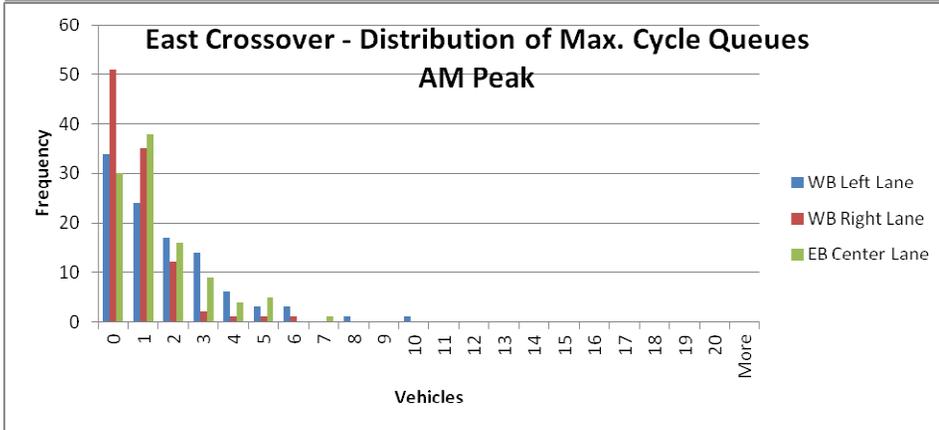
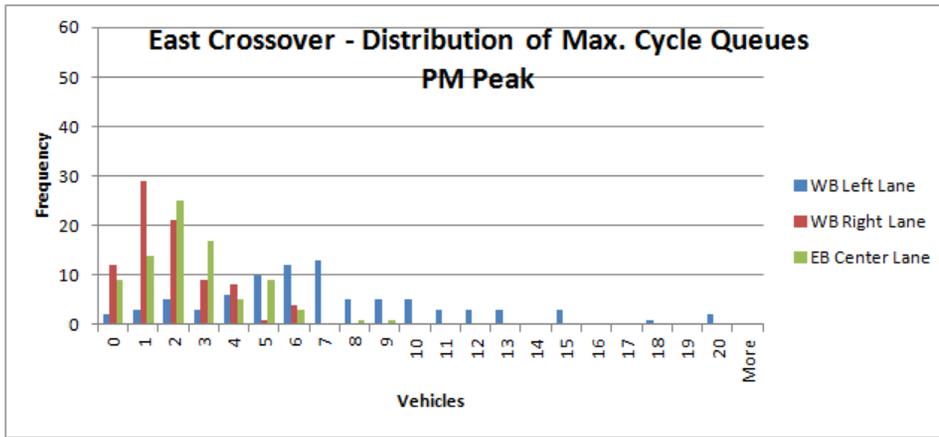
APPENDICES

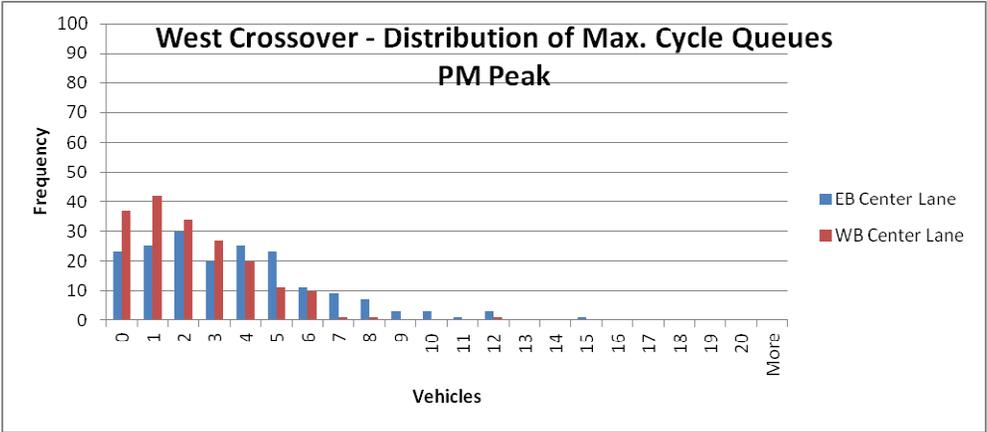
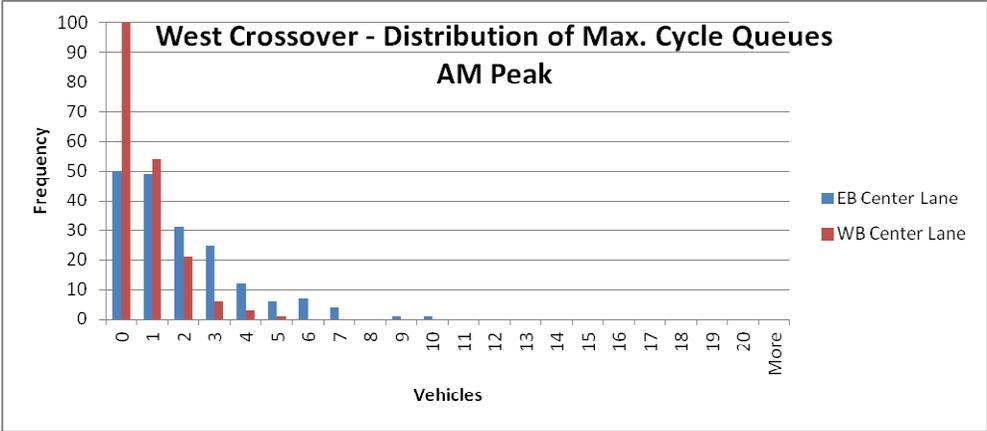
APPENDIX A:
Field Data from Alcoa, TN

Peak Period Queues – Transition

West Crossover	AM Peak - 6:30am - 8:30am		AM Off Peak - 9:00am-11:00am		Lunch Peak - 11:45am-1pm		PM Off Peak - 2:45-4:00pm	
	EB	WB	EB	WB	EB	WB	EB	WB
Mean	1.9	0.7	1.8	1.4	2.4	1.5	3.6	2.2
Min	0	0	0	0	0	0	0	0
Max	10	5	7	8	10	7	15	12
Std.Dev	1.94	0.97	1.47	1.58	2.01	1.43	2.87	1.96
n	186	186	110	110	72	72	184	184

East Crossover	AM Peak - 6:30am - 8:30am			PM Off Peak - 2:45-4:00pm			PM Peak - 4:00pm-6:00pm		
	EB	WB - L1	WB-L2	EB	WB - L1	WB-L2	EB	WB - L1	WB-L2
Mean	1.4	1.7	0.8	2.9	5.8	1.5	2.6	7.2	1.9
Min	0	0	0	0	0	0	0	0	0
Max	7	10	6	8	21	8	9	20	6
Std.Dev	1.46	1.89	1.07	1.93	3.97	1.48	1.82	4.16	1.51
n	103	103	103	131	131	123	84	84	84

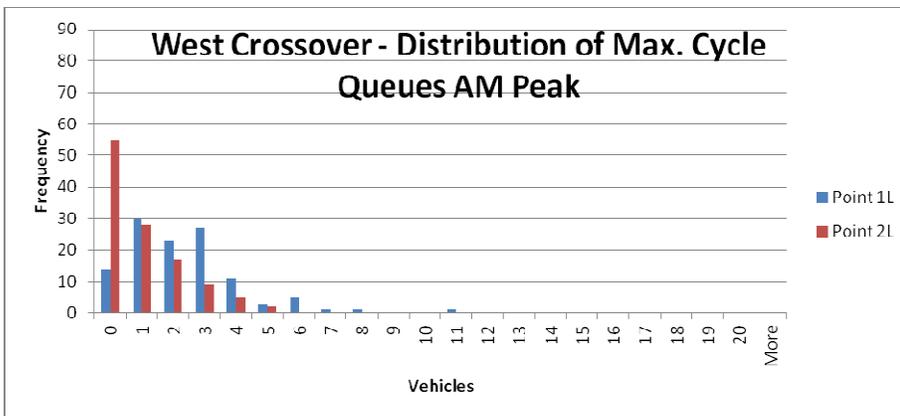
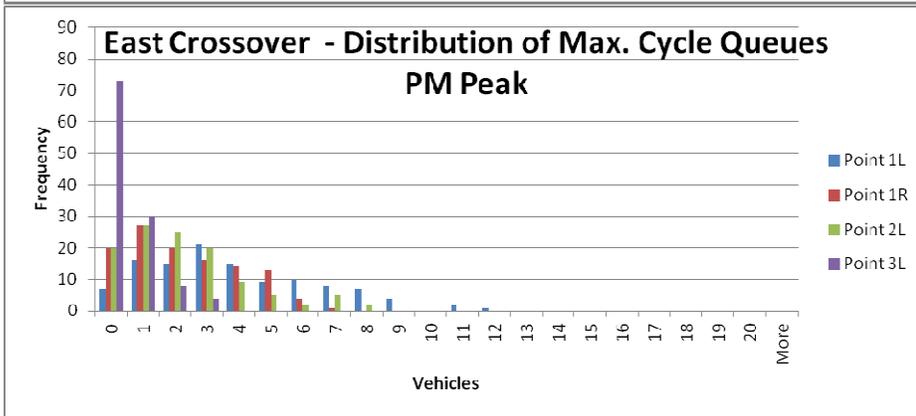
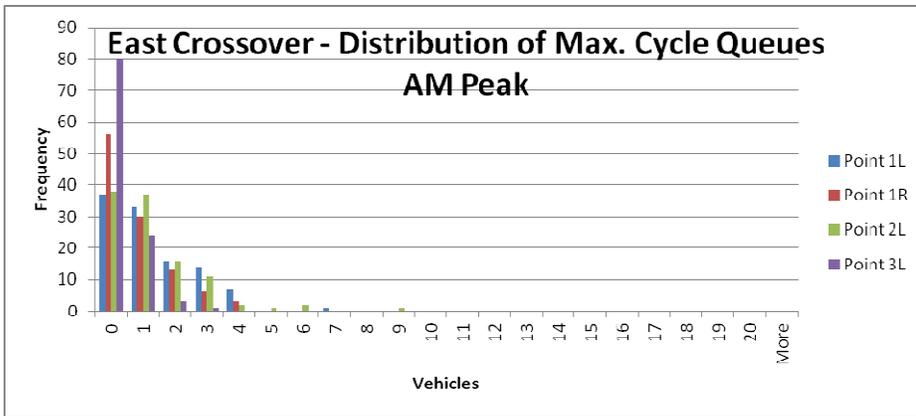


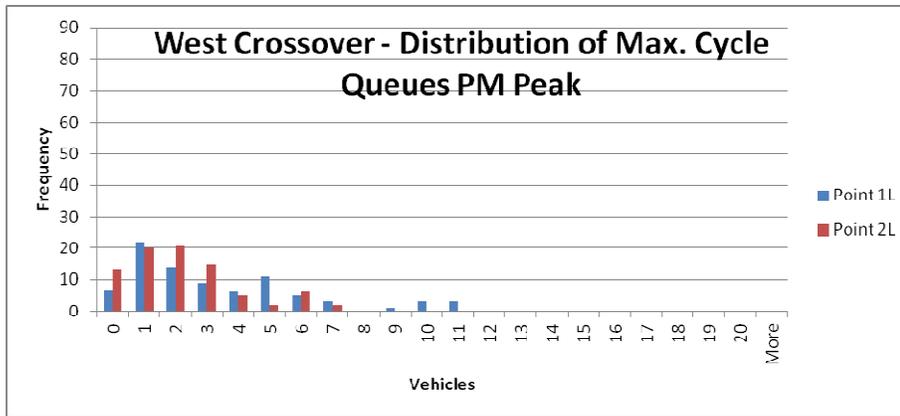


Peak Period Queues – After

Side A	AM Peak – 7:00 am – 9:00 am		PM Peak – 4:30 pm – 6:30 pm	
	Point 1 – EB Left Lane	Point 2 –WB Left Lane	Point 1 – EB Left Lane	Point 2 –WB Left Lane
Mean	2.3	1.0	3.3	2.2
Min	0	0	0	0
Max	11	5	11	7
Std. Dev.	1.8	1.3	2.9	1.8
n	116	116	84	84

Side B	AM Peak – 7:00 am – 9:00 am				PM Peak – 4:30 pm – 6:30 pm			
	Point 1 – WB Left Lane	Point 1 – WB Right Lane	Point 2 – EB Left Lane	Point 3 – NB Left Lane	Point 1 – WB Left Lane	Point 1 – WB Right Lane	Point 2 – EB Left Lane	Point 3 – NB Left Lane
Mean	1.3	0.8	1.3	0.3	4.0	2.3	2.3	0.5
Min	0	0	0	0	0	0	0	0
Max	7	4	9	3	12	7	8	3
Std. Dev.	1.4	1.0	1.5	0.6	2.7	1.8	1.9	0.8
n	108	108	108	108	115	115	115	115





Saturation Flow Rate Calculation – After

Saturation Flow Rate Calculation for West Crossover EB Through Movement

Obs No.	Time (seconds) between 4th vehicle and...			
	7th veh.	8th veh.	9th veh.	10th veh.
1				15.35
2	7.4			
3	5.78			
4	6.47			
5			10.38	
6				9.94
7				11.03
8				11.31
9				9.94
10				11.34
11				11.5
12				11.6
13			10.69	
14	7.34			
15				11.5
16				12.06
17				11
18				14.13
19				11.9
20				10.72
21	5.78			
22	5.34			
23	8.88			
Sum	46.99	0	21.07	163.32
Total Observations = 23		Mean Sat Flow (vph) = 1758		

Saturation Flow Rate Calculation for West Crossover WB Through Movement

Obs No.	Time (seconds) between 4th vehicle and...			
	7th veh.	8th veh.	9th veh.	10th veh.
1	6.57			
2	6.09			
3	7.28			
4	5.28			
Sum	25.22	0	0	0
Total Observations = 4		Mean Sat Flow (vph) = 1713		

Saturation Flow Rate Calculation for East Crossover WB Through Movement

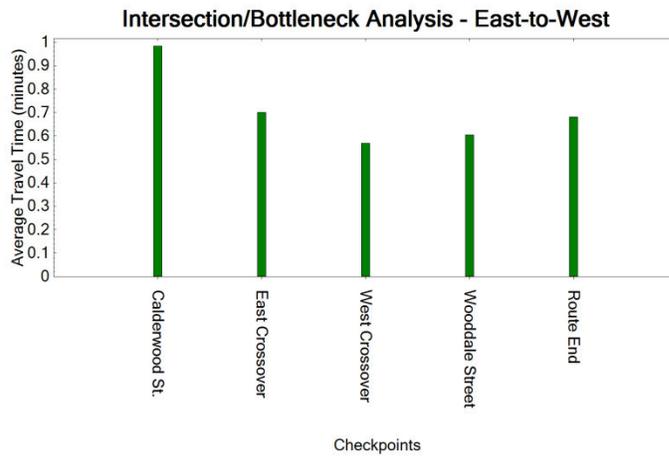
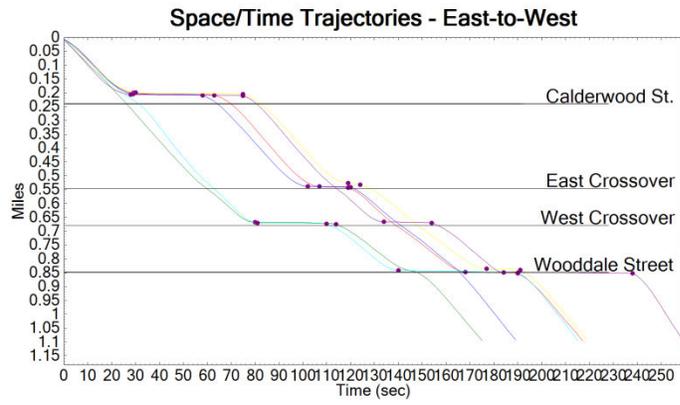
Obs No.	Time (seconds) between 4th vehicle and...			
	7th veh.	8th veh.	9th veh.	10th veh.
1	5.86			
2		8.03		
3	6.42			
4				13.53
5				9.9
6				12.04
7	5.59			
8	6.85			
9				12.03
10	5.97			
11	6.69			
12	5.03			
13	4.91			
14		7.68		
15			10	
16		6.25		
17		9.84		
18	5.9			
19			9.4	
20	6.19			
21		9.15		
22			9.15	
23				10.81
24			9.46	
25		8.53		
26	6.09			
27			10.9	
28	9.16			
29				12.79
Sum	74.66	49.48	48.91	71.1
Total Observations = 29		Mean Sat Flow (vph) = 1773		

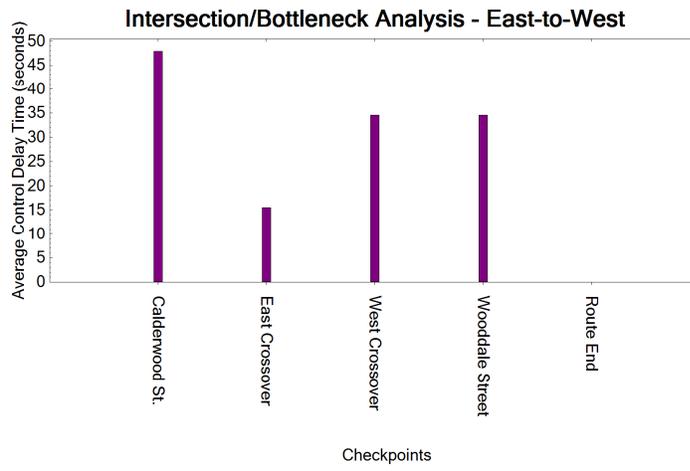
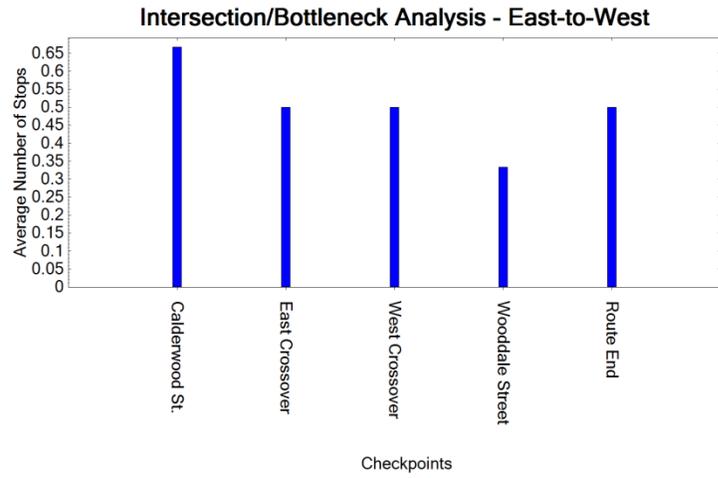
Saturation Flow Rate Calculation for East Crossover EB Through Movement

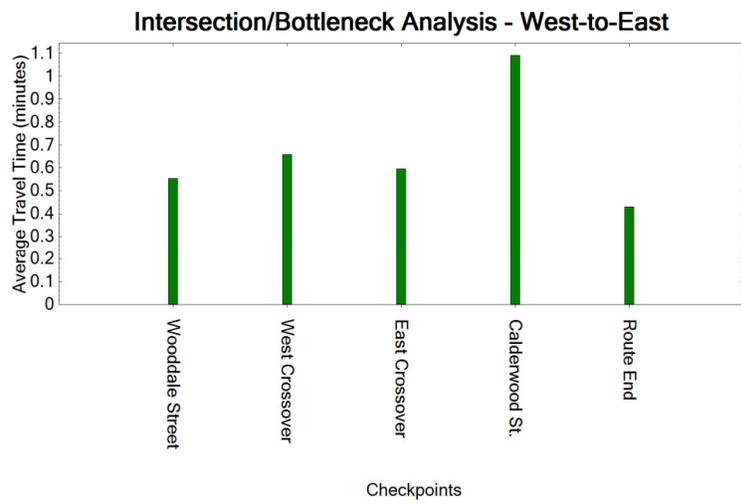
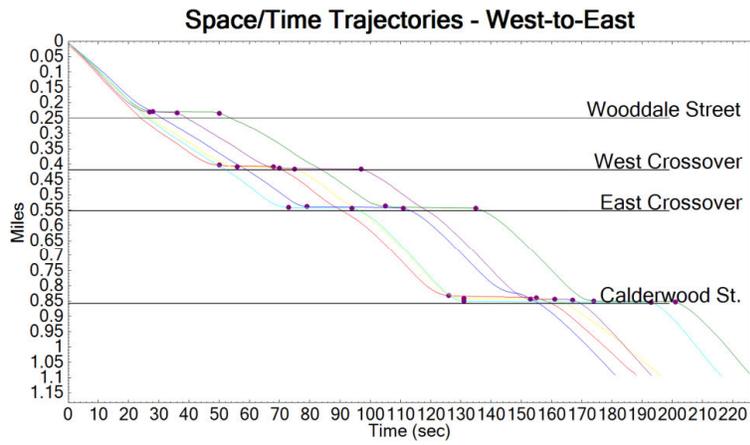
Obs No.	Time (seconds) between 4th vehicle and...			
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1			11.03	
2		9.01		
3		7.53		
4	5.56			
5	6.16			
6			9.41	
7			11.88	
8	6			
9	5.93			
10	7.72			
11		8.59		
12	6.36			
13	6.03			
14				11.65
15	6.47			
Sum	50.23	25.13	32.32	11.65
Total Observations = 15		Mean Sat Flow (vph) = 1718		

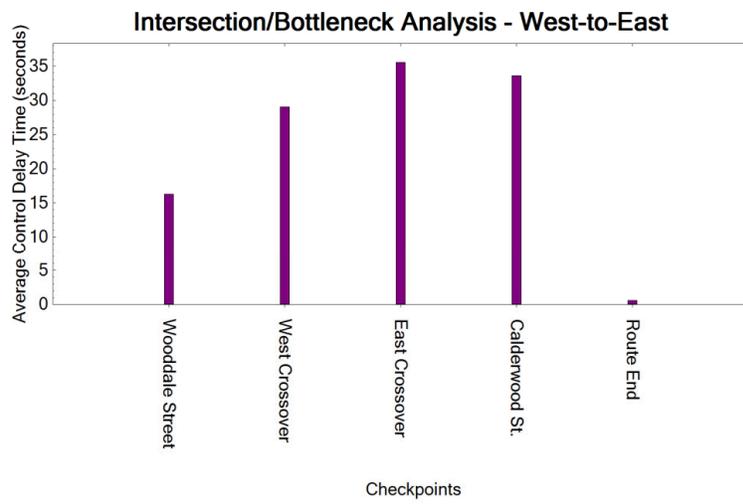
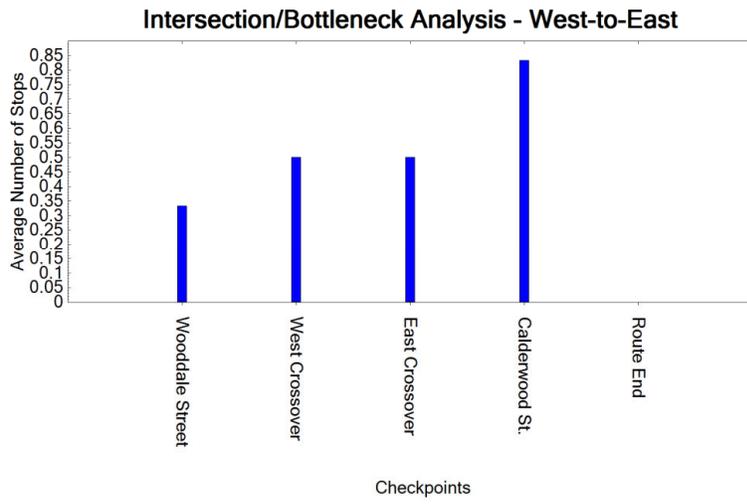
APPENDIX B:
Travel Time Data from Alcoa, TN

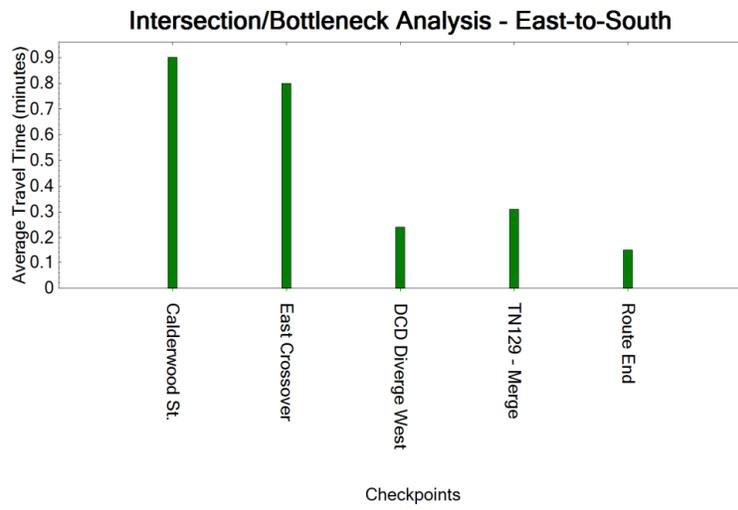
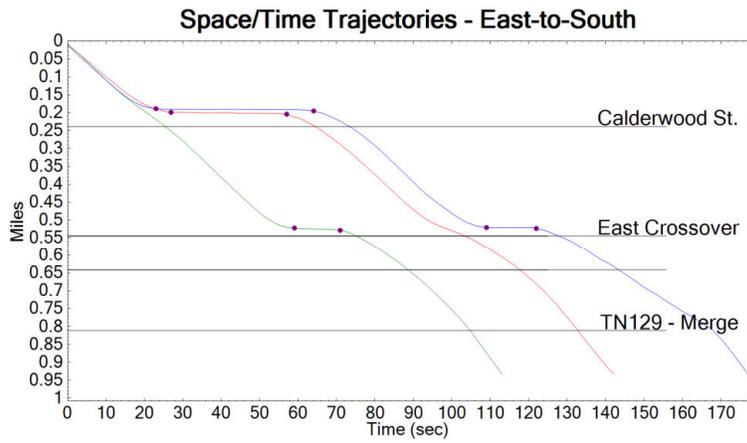
AM Peak Period Transition Data



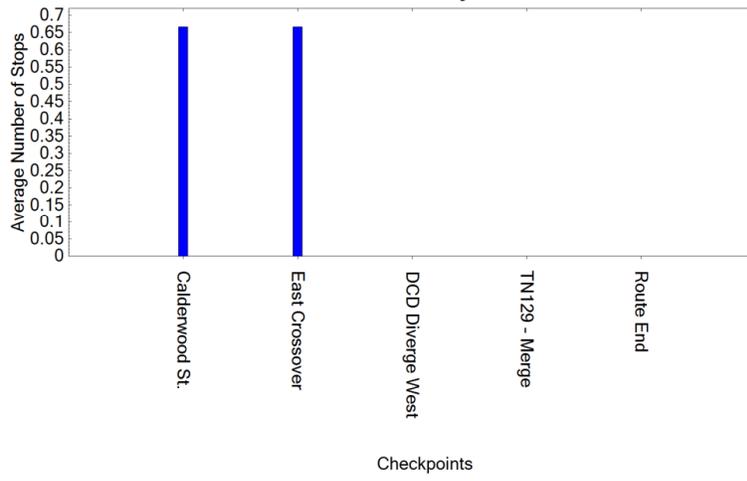




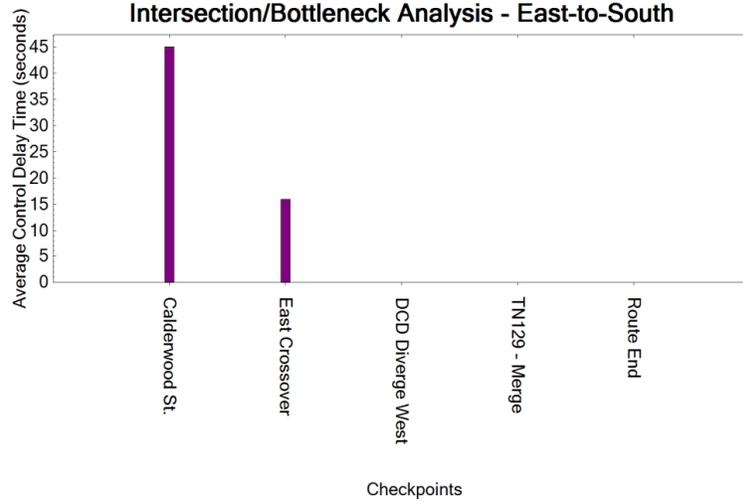


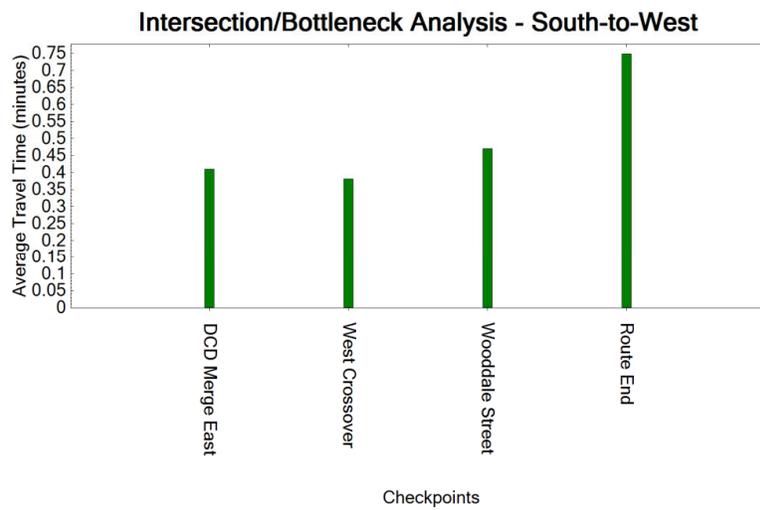
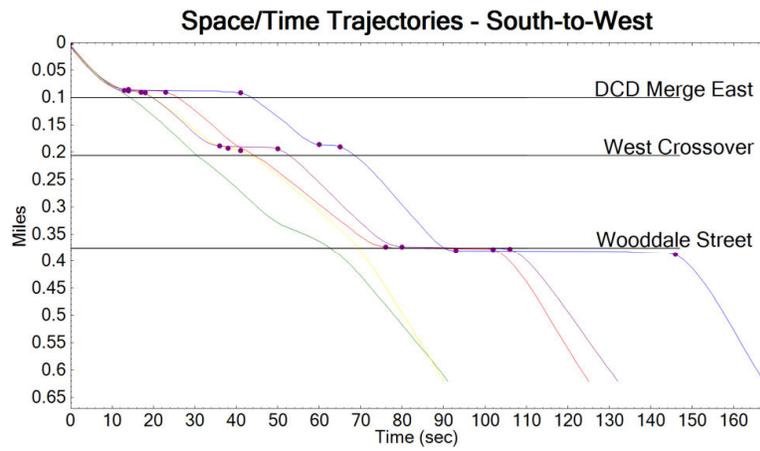


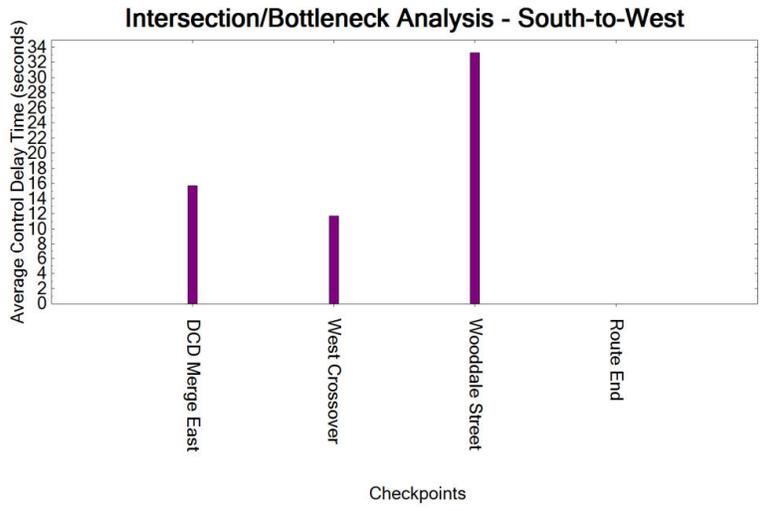
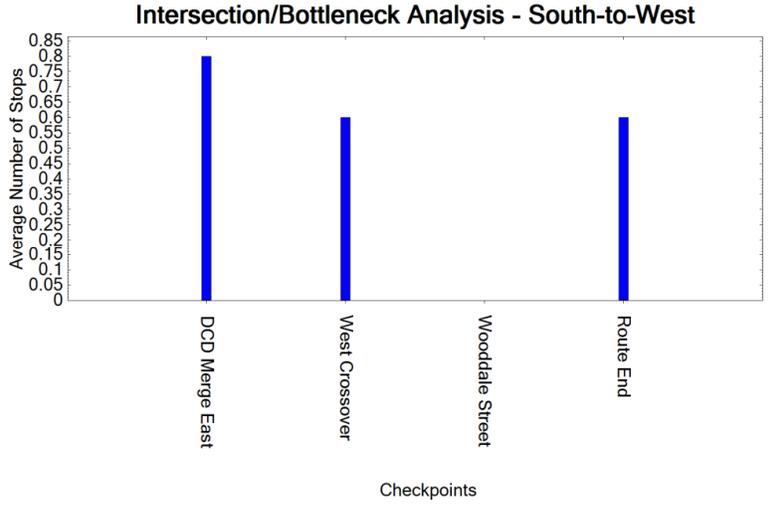
Intersection/Bottleneck Analysis - East-to-South

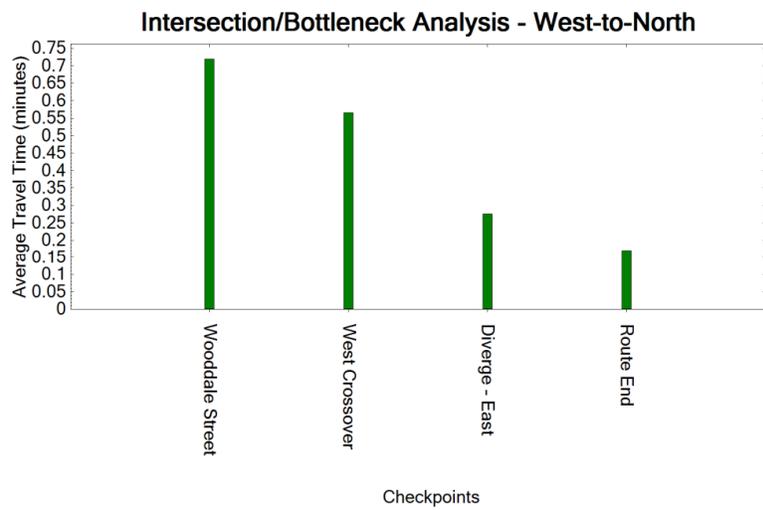
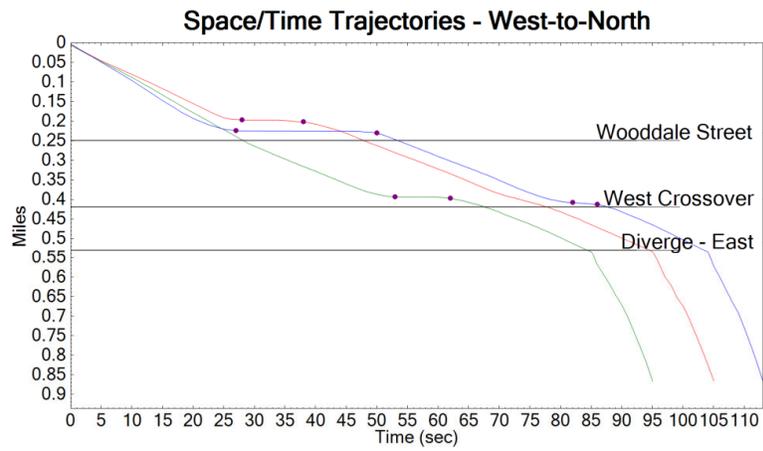


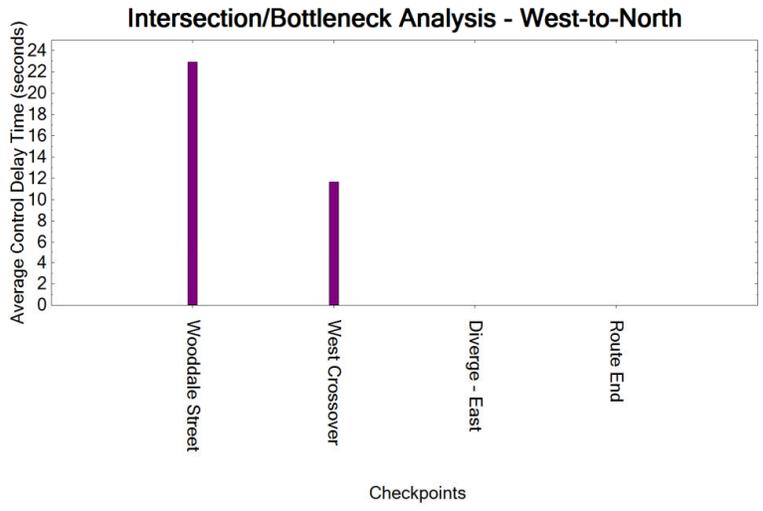
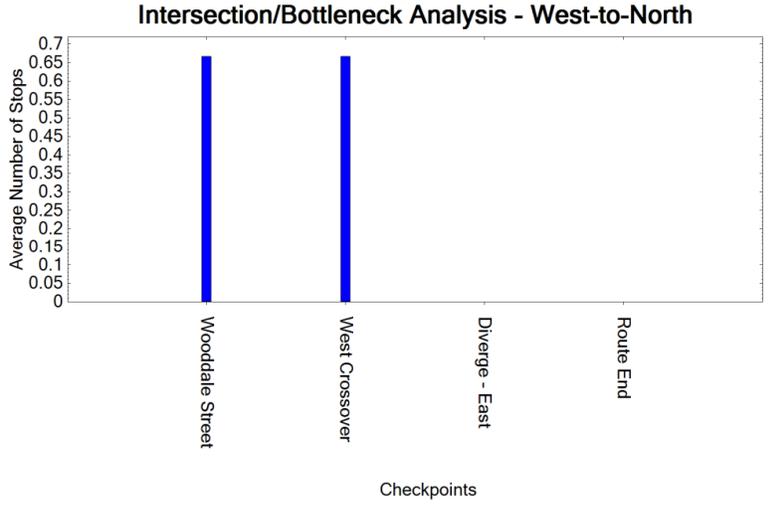
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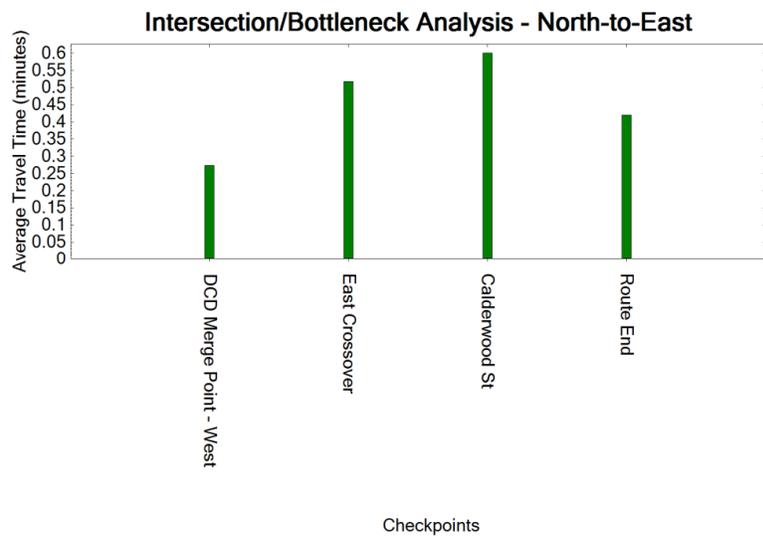
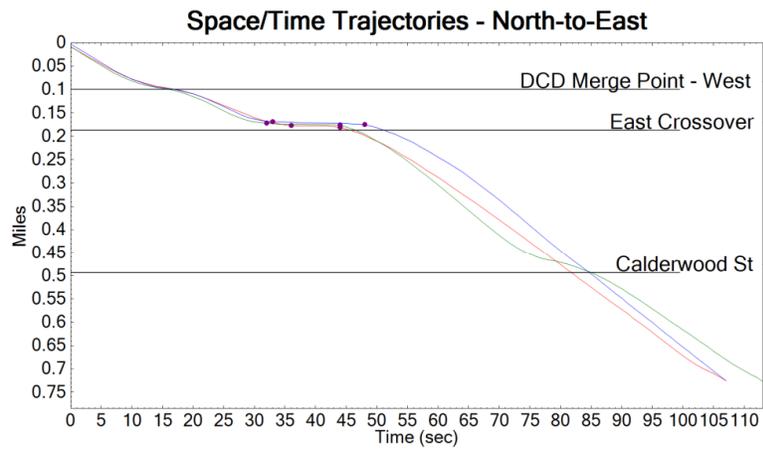




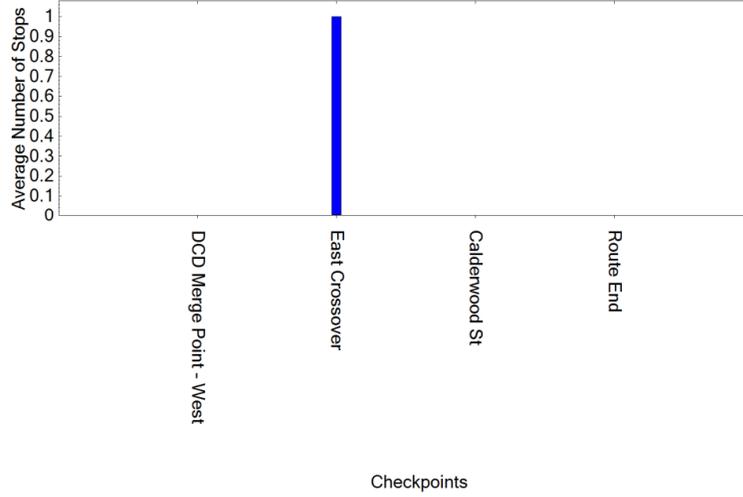




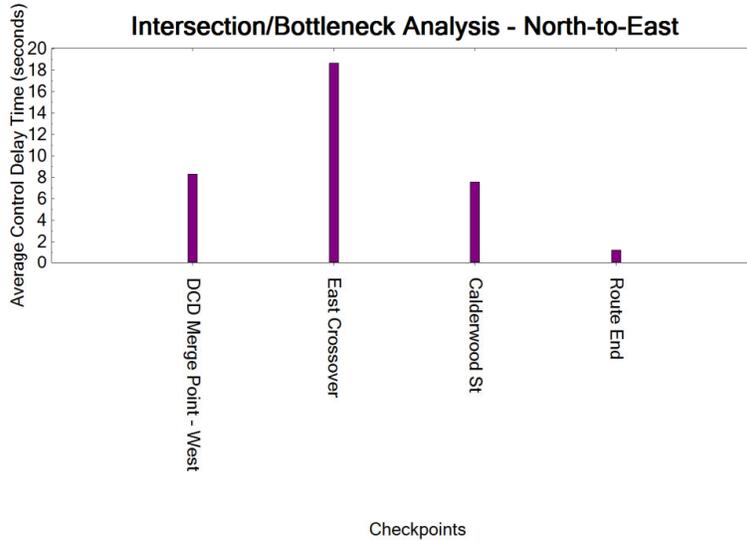




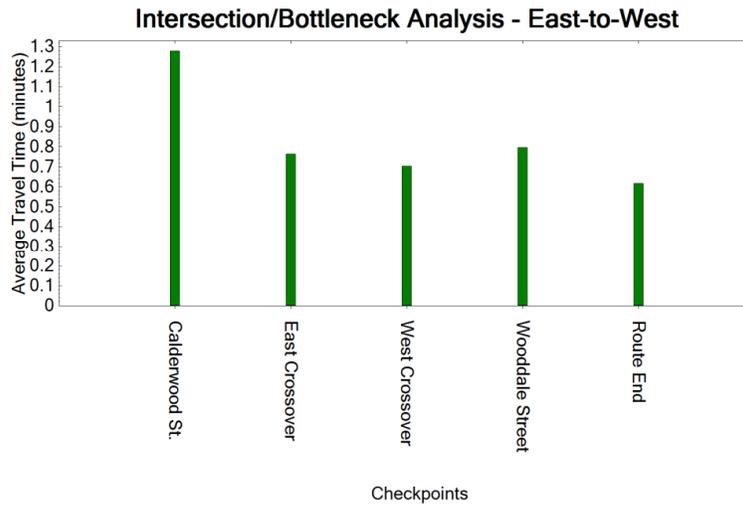
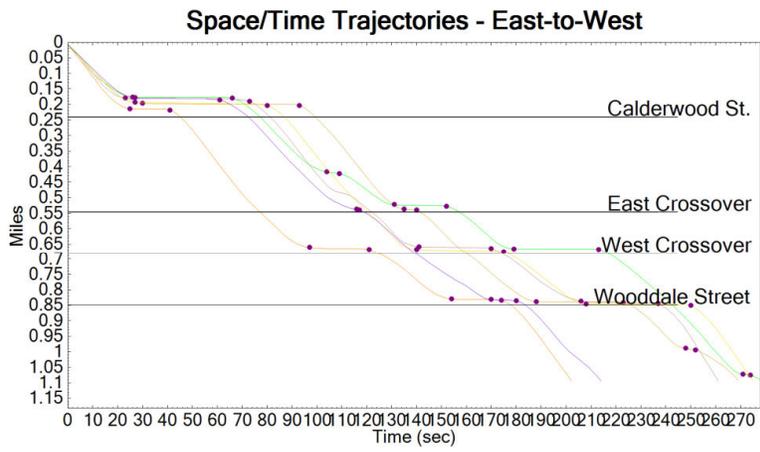
Intersection/Bottleneck Analysis - North-to-East

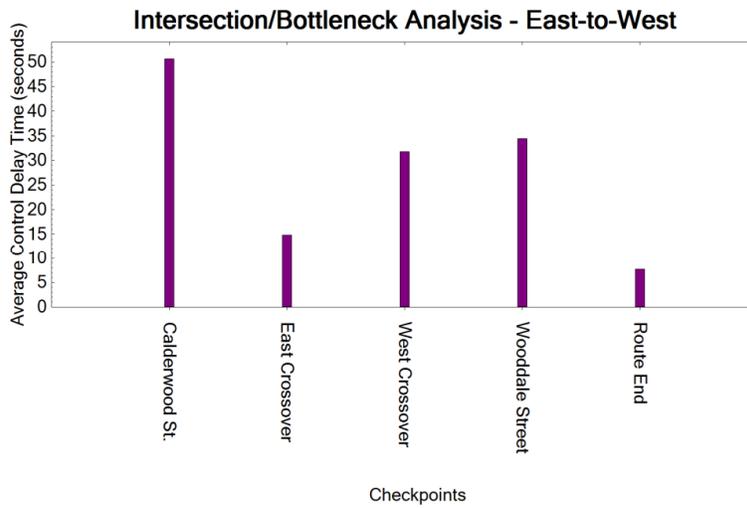
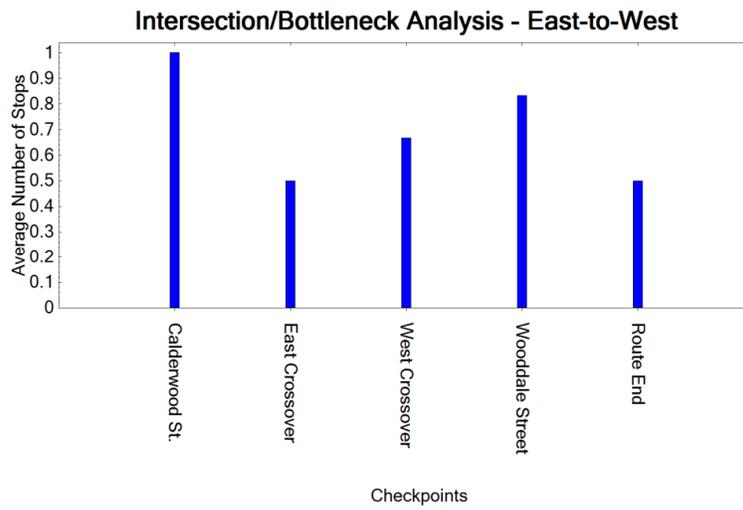


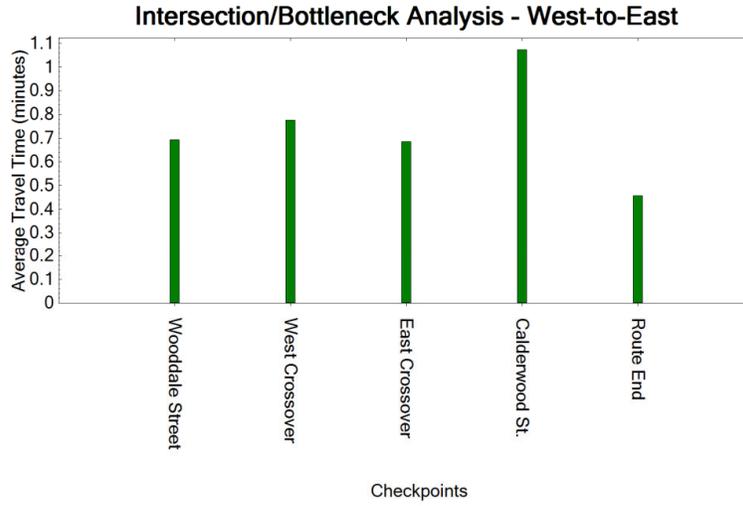
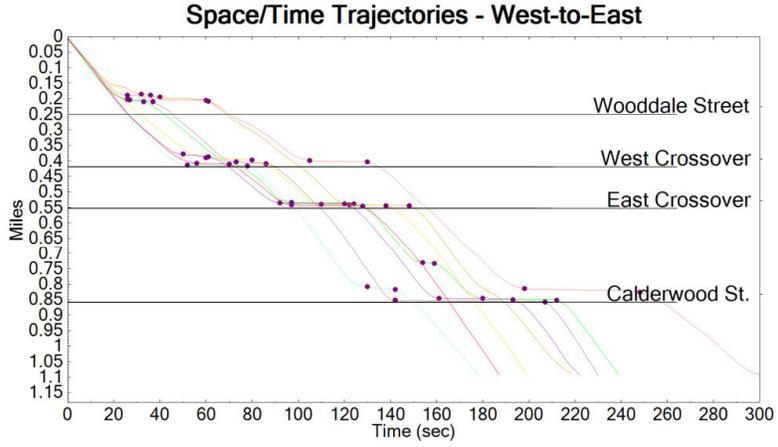
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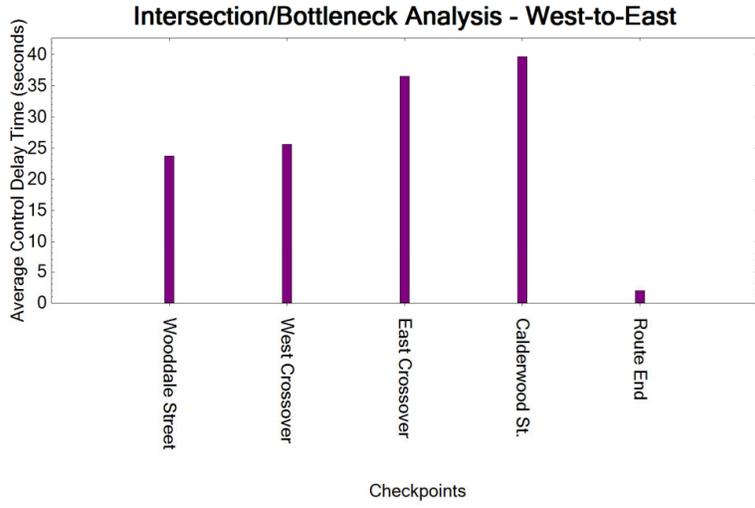
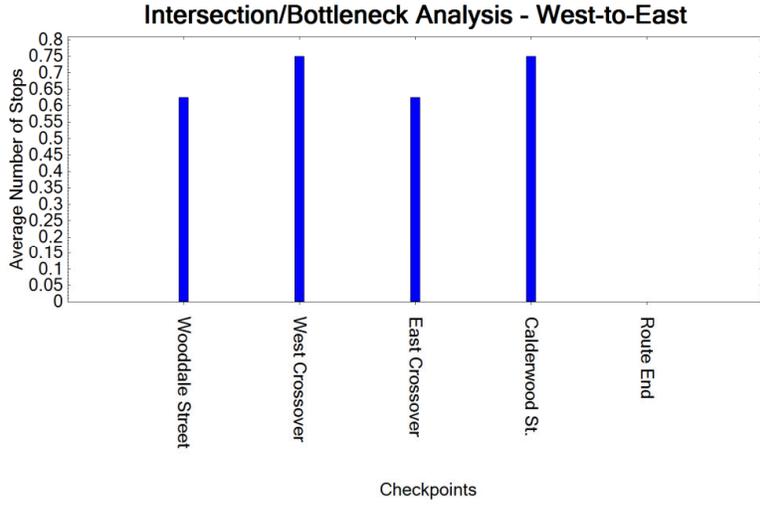


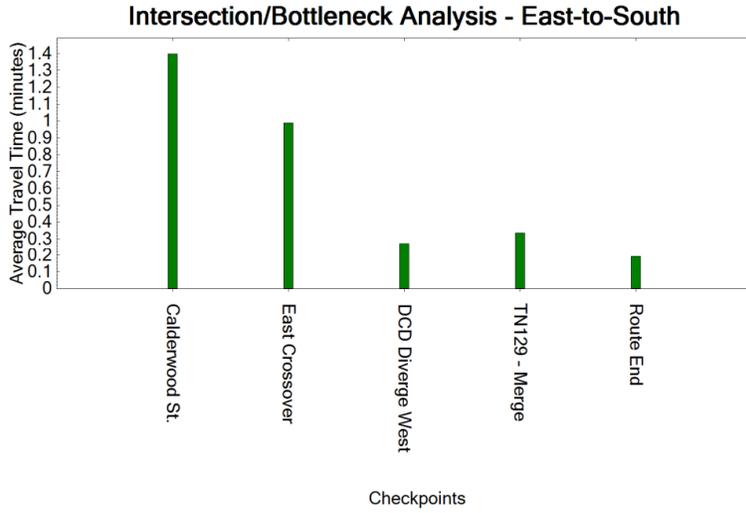
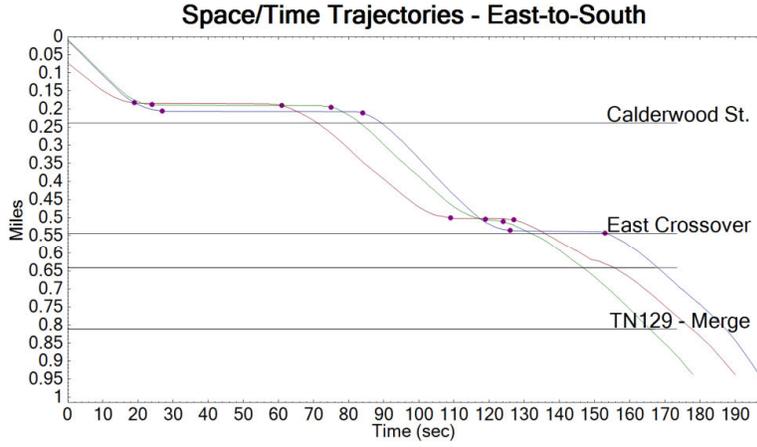
PM Peak Period Transition Data



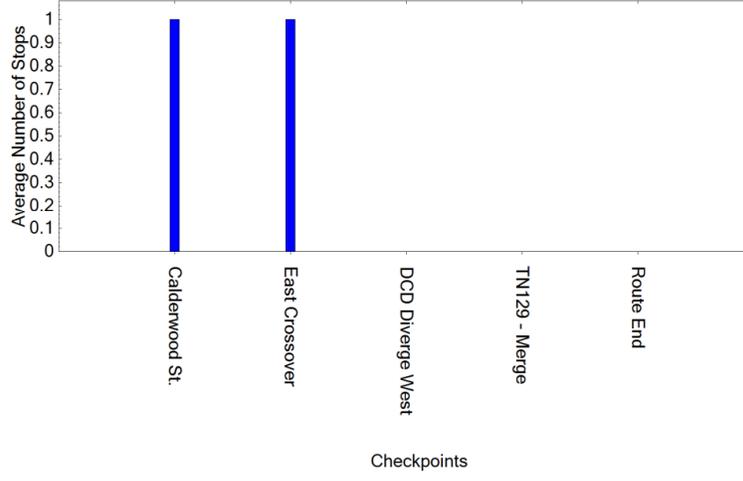




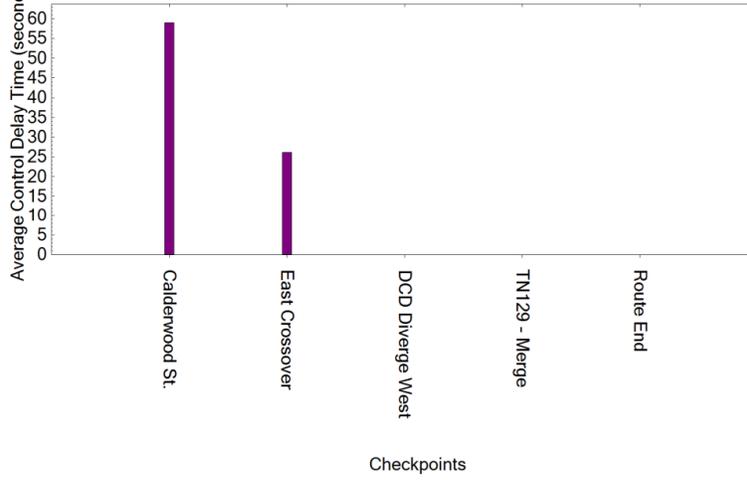


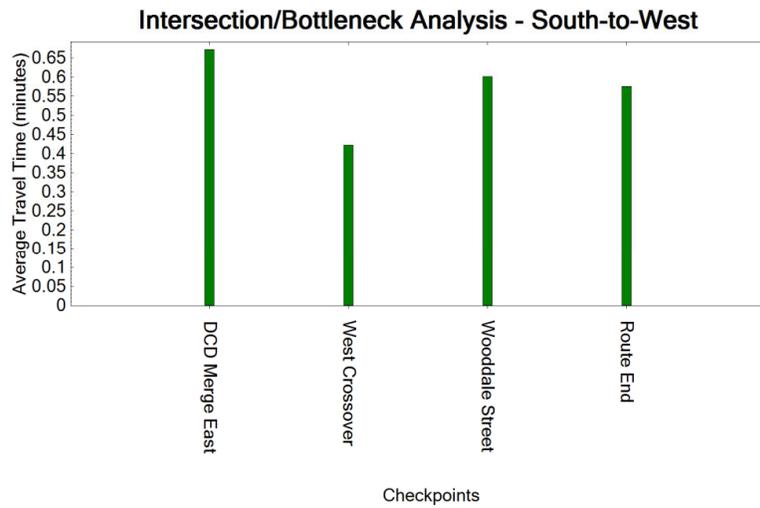
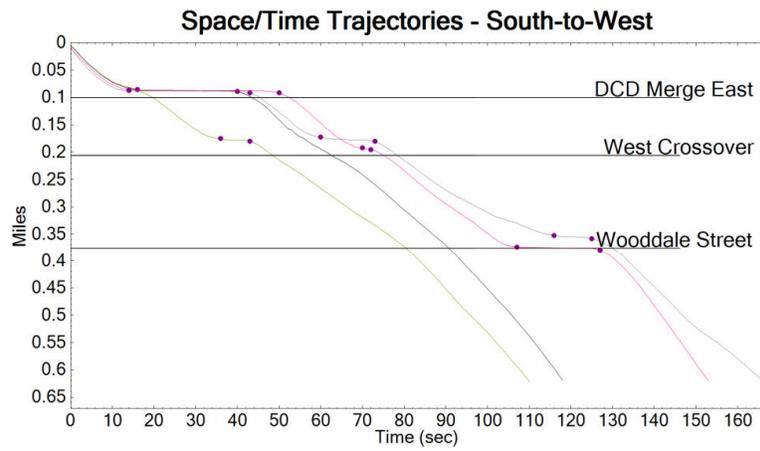


Intersection/Bottleneck Analysis - East-to-South

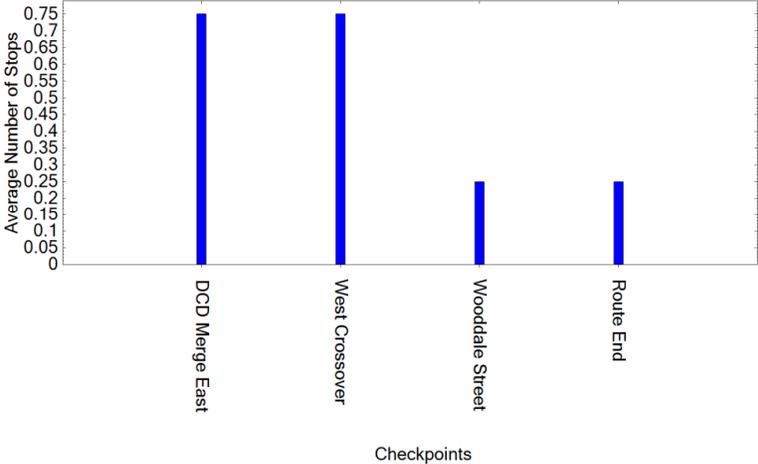


Intersection/Bottleneck Analysis - East-to-South

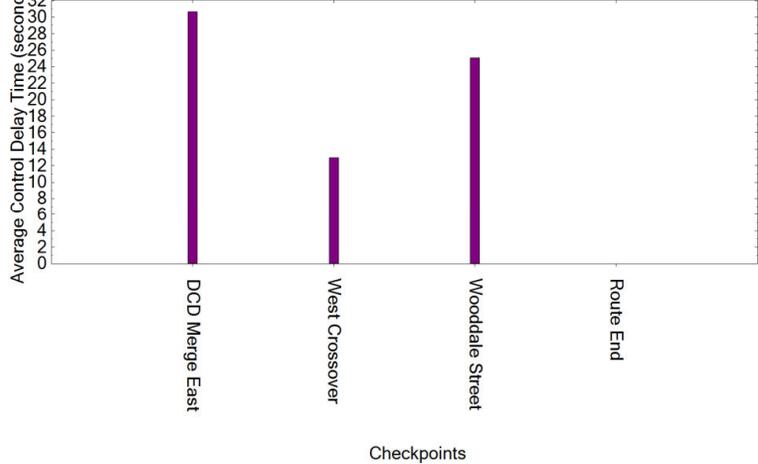


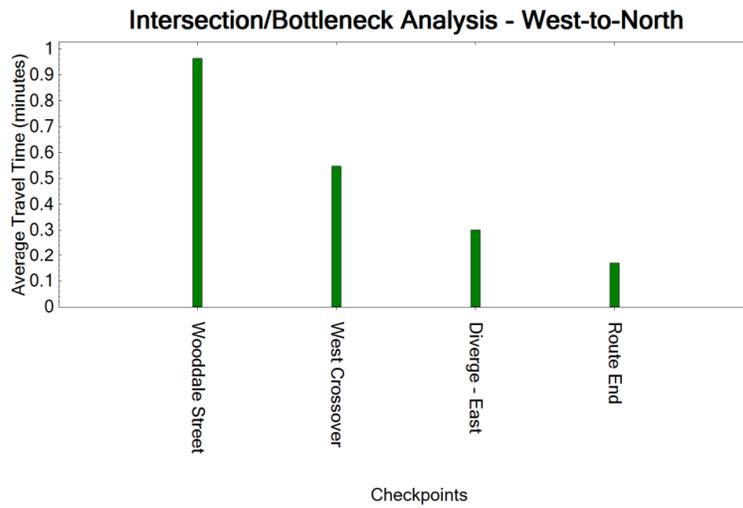
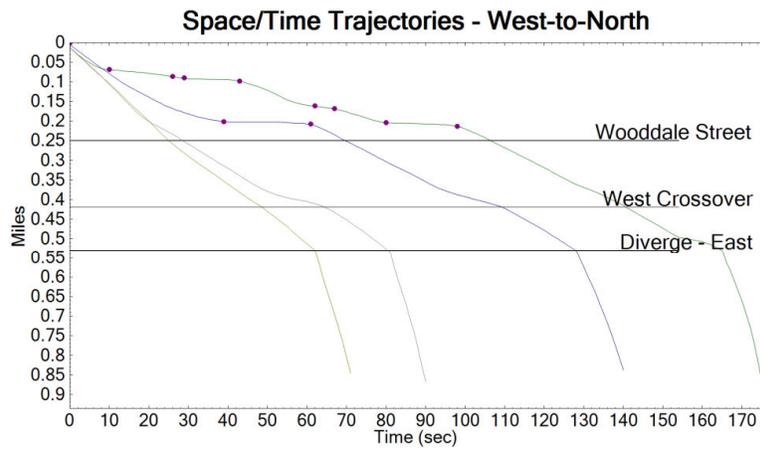


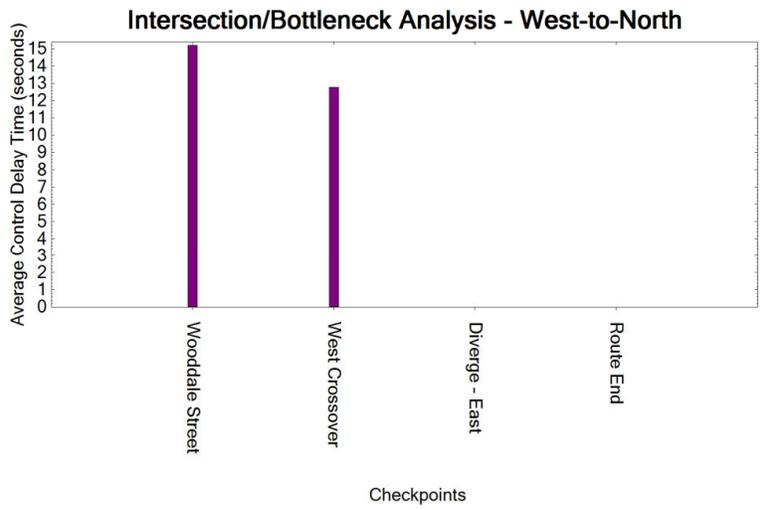
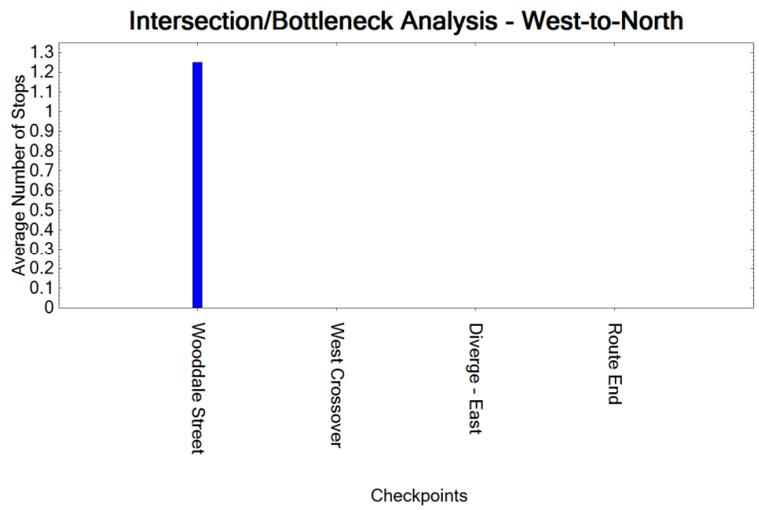
Intersection/Bottleneck Analysis - South-to-West

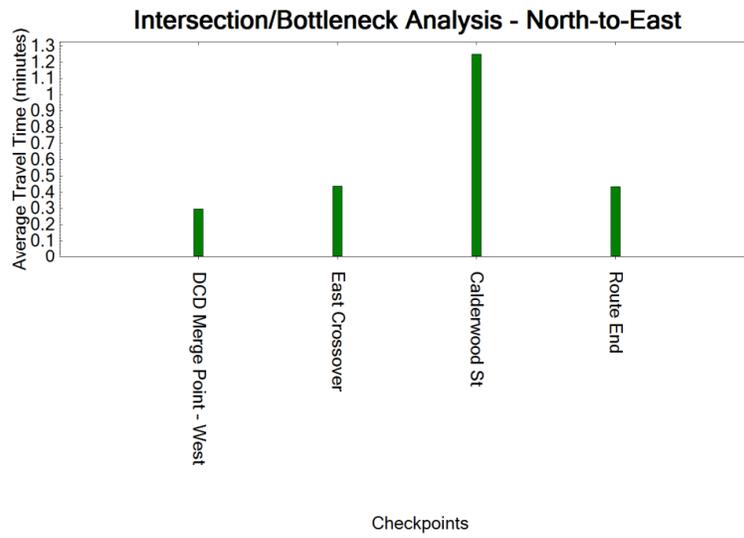
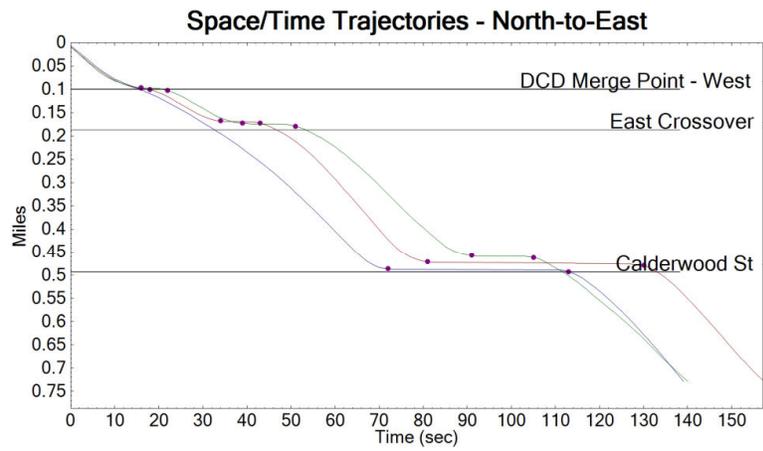


Intersection/Bottleneck Analysis - South-to-West

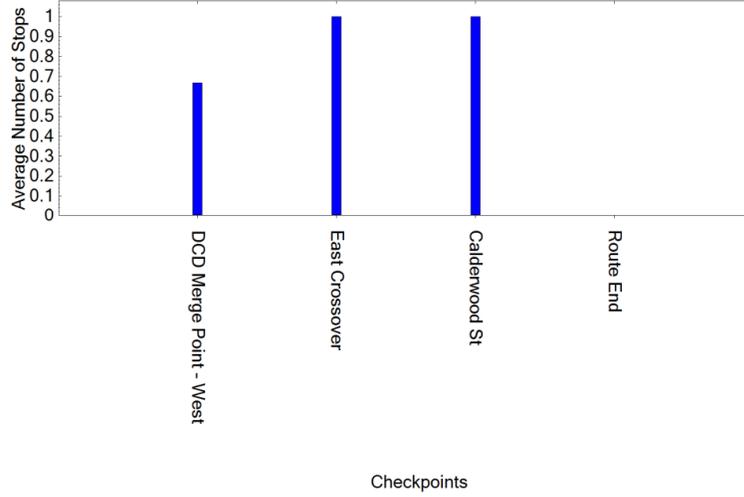




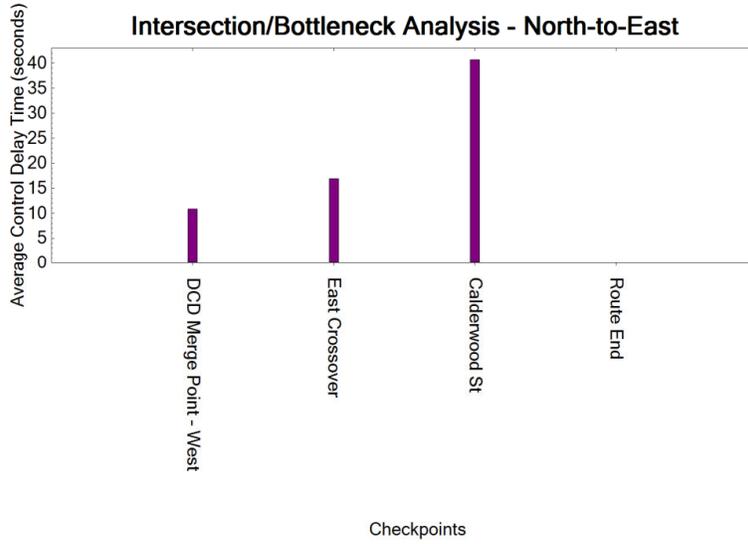




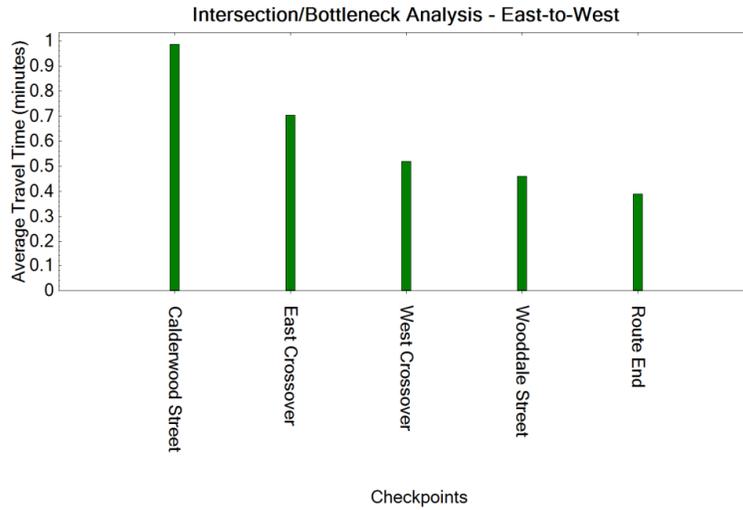
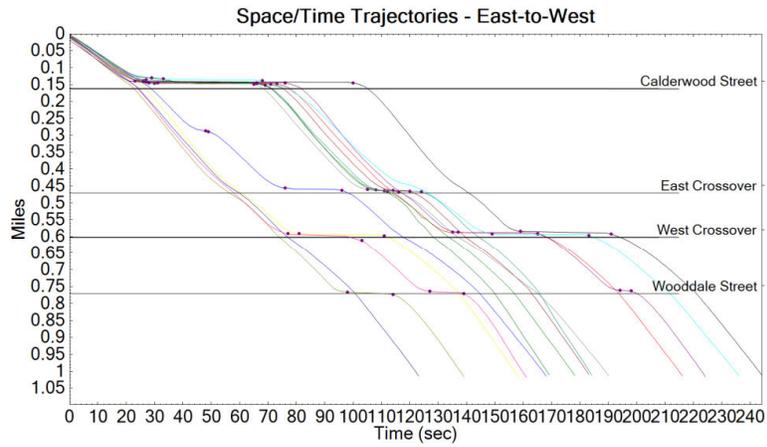
Intersection/Bottleneck Analysis - North-to-East

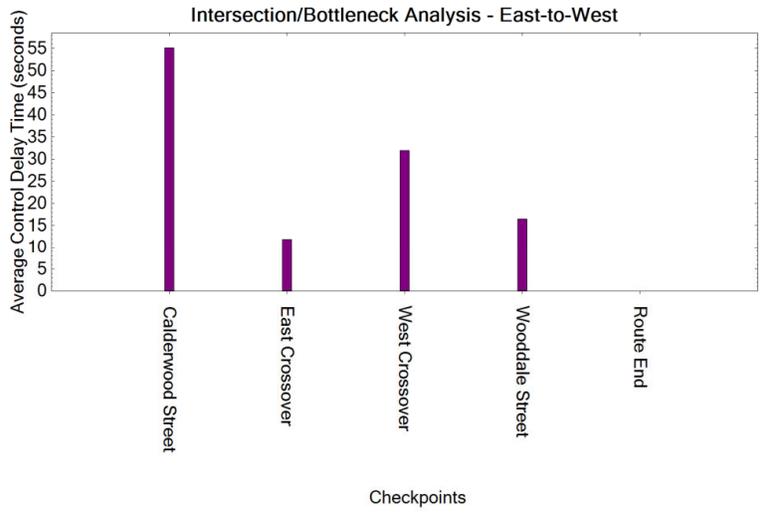
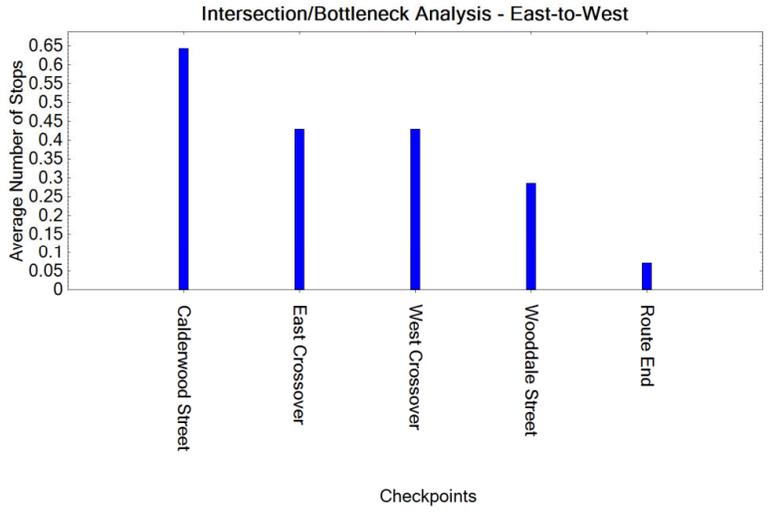


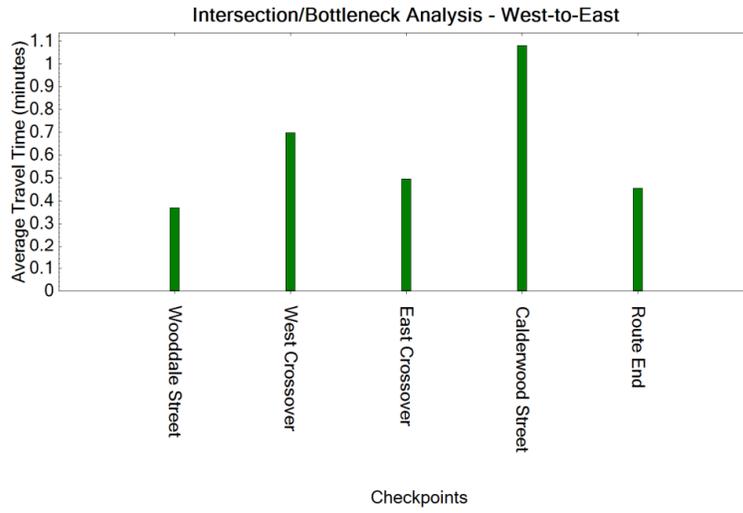
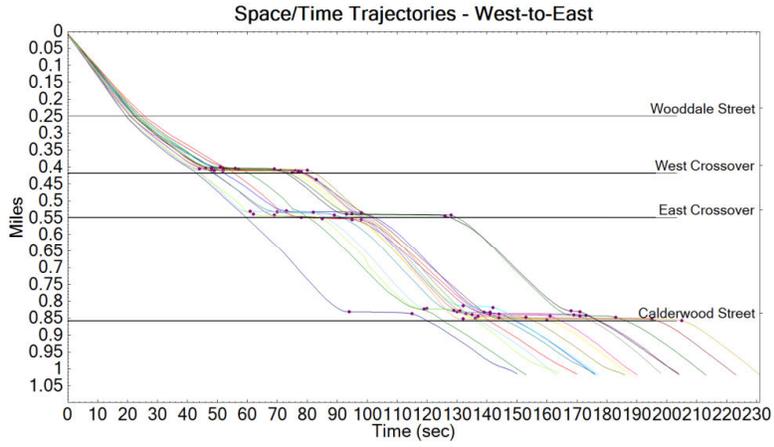
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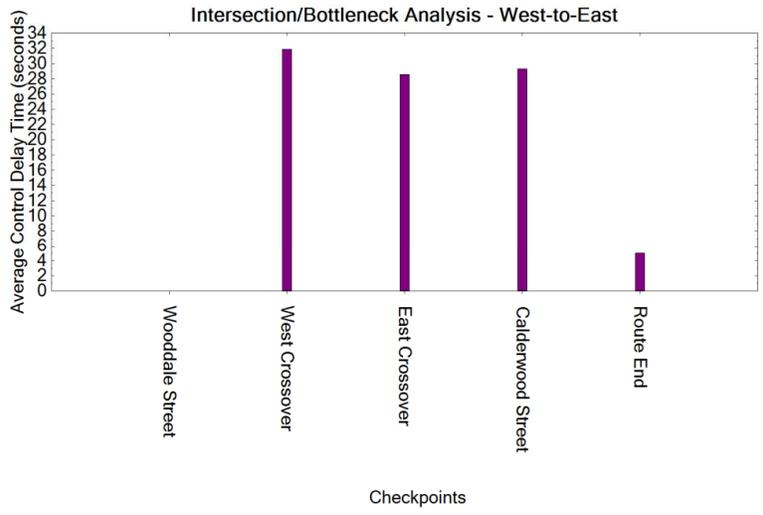
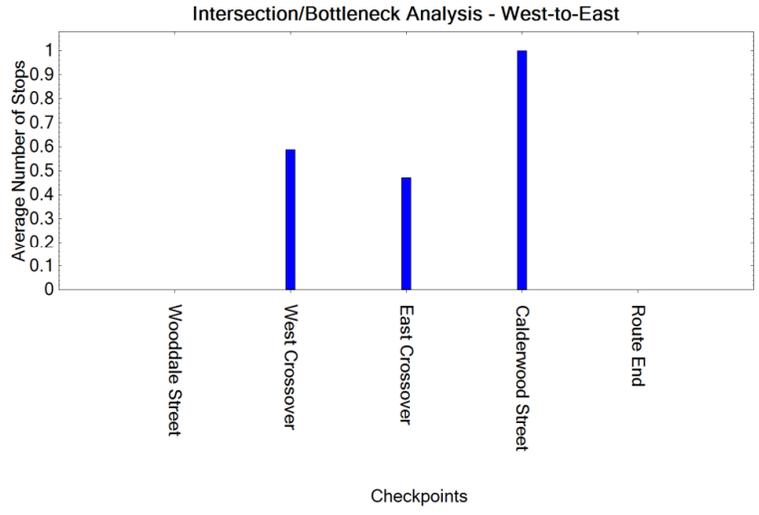


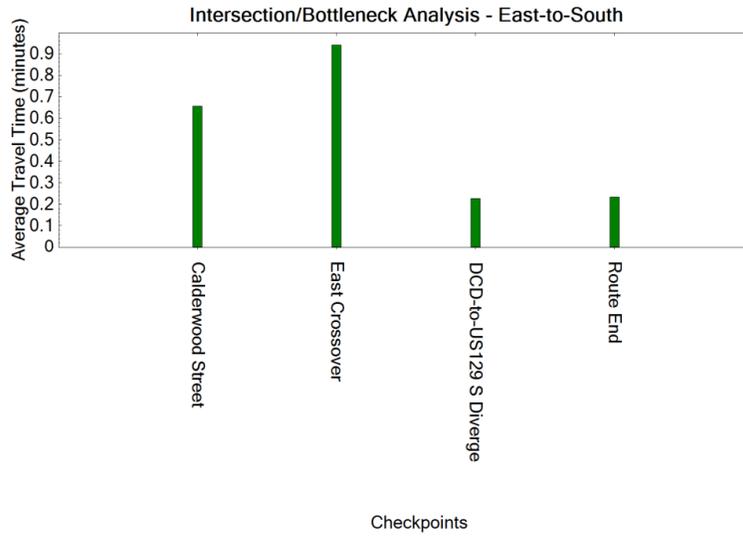
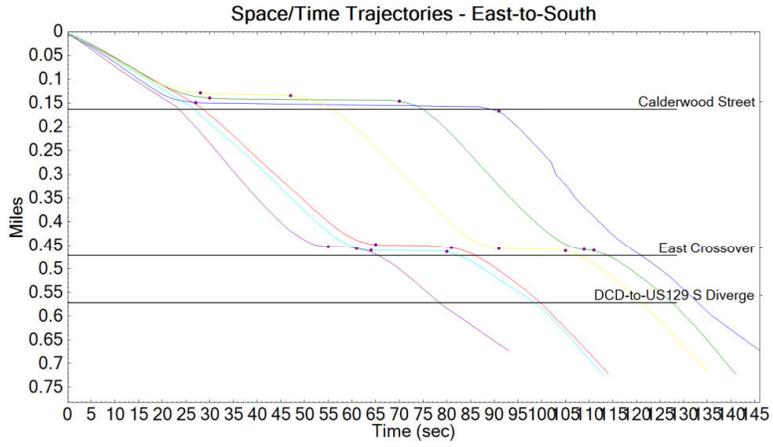
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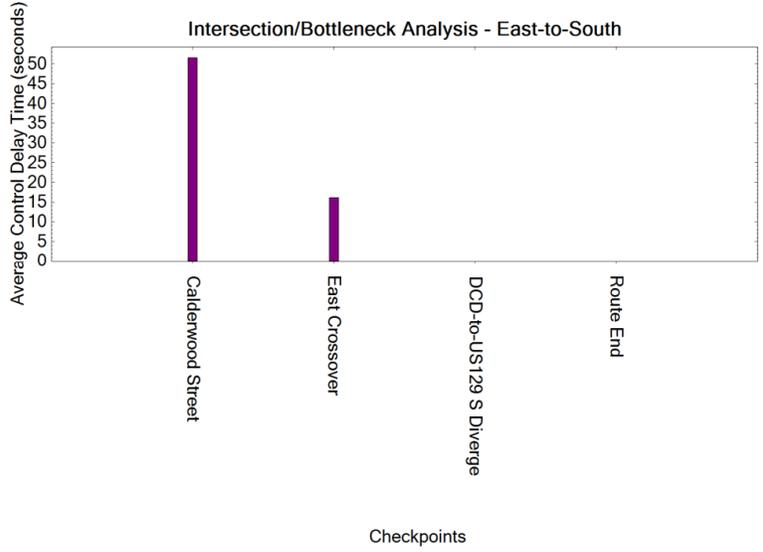
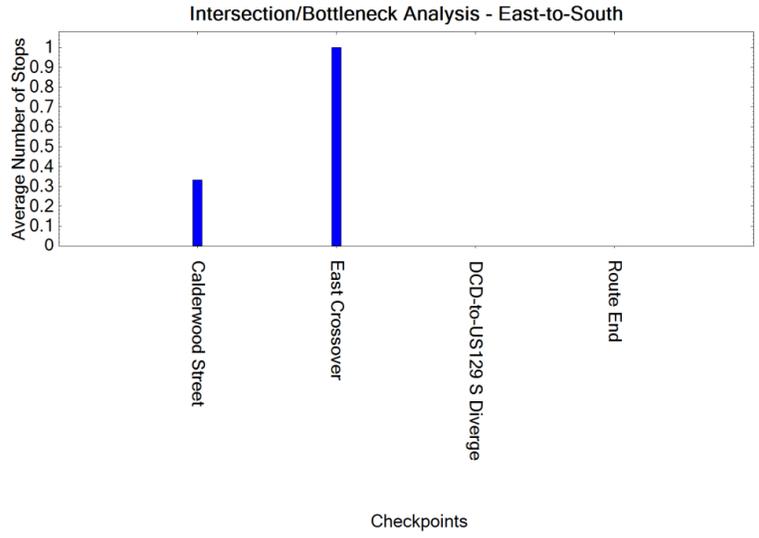


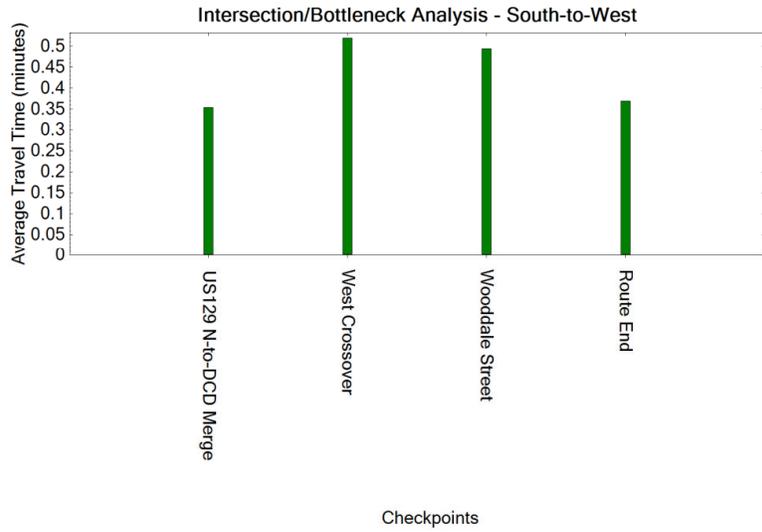
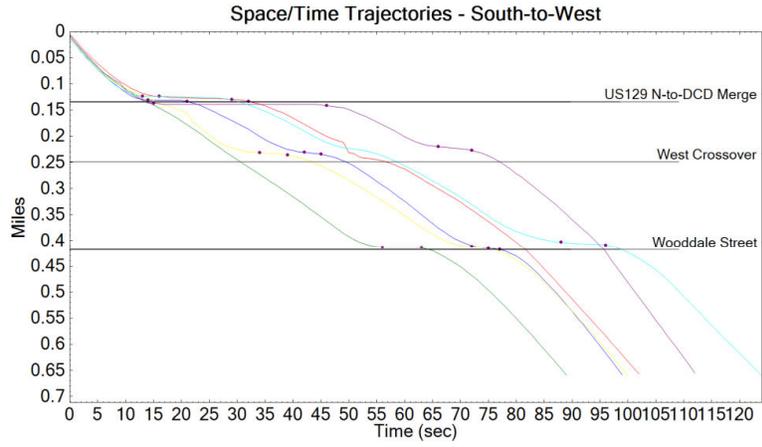


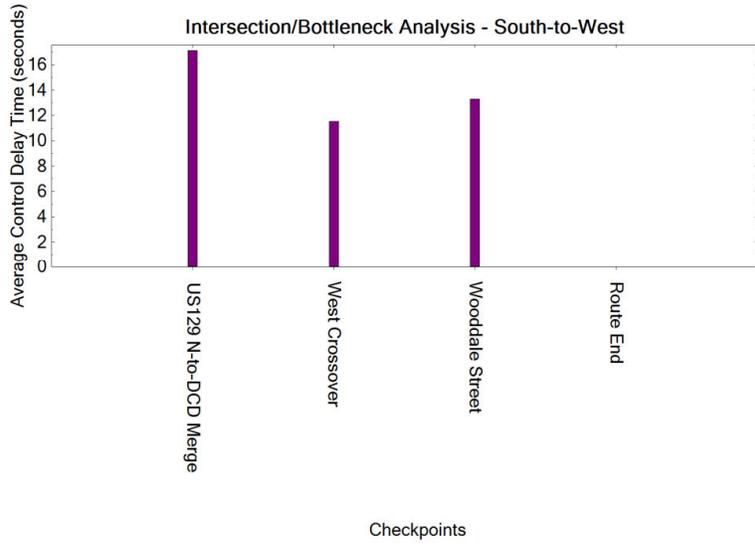
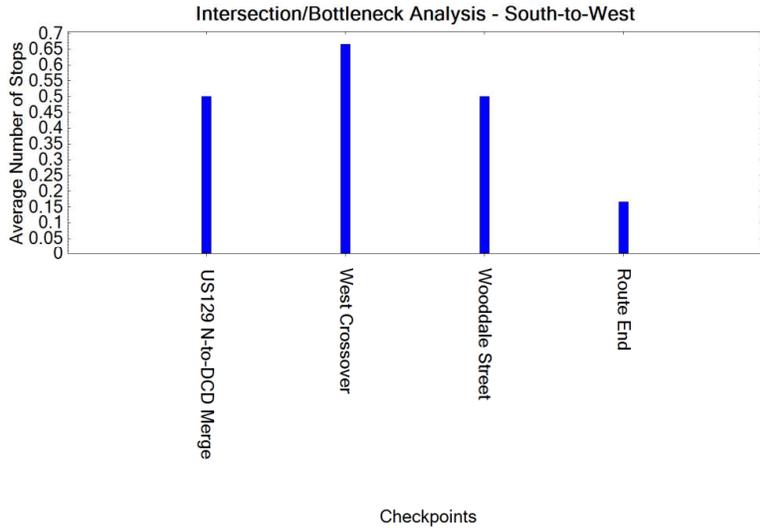


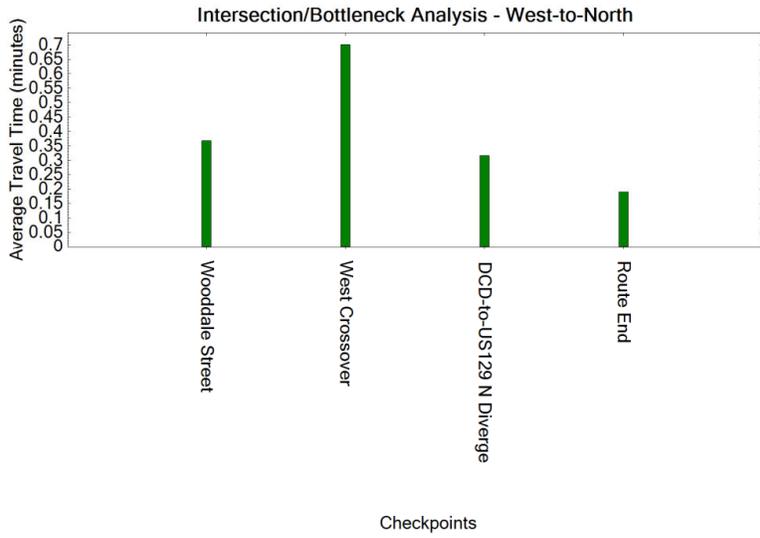
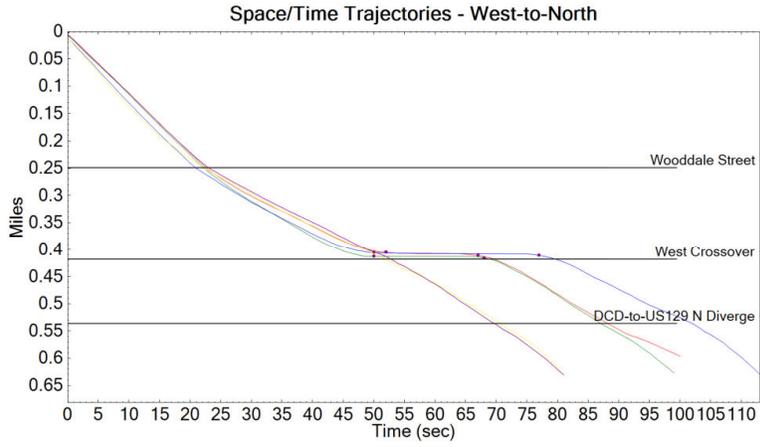


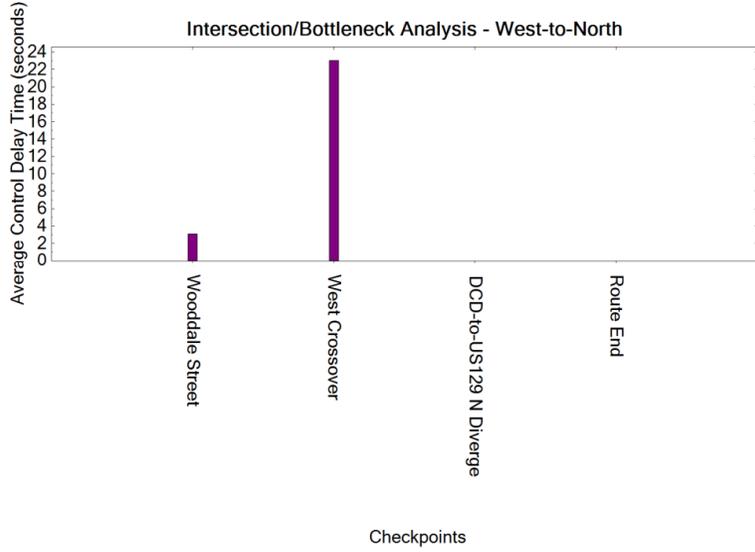
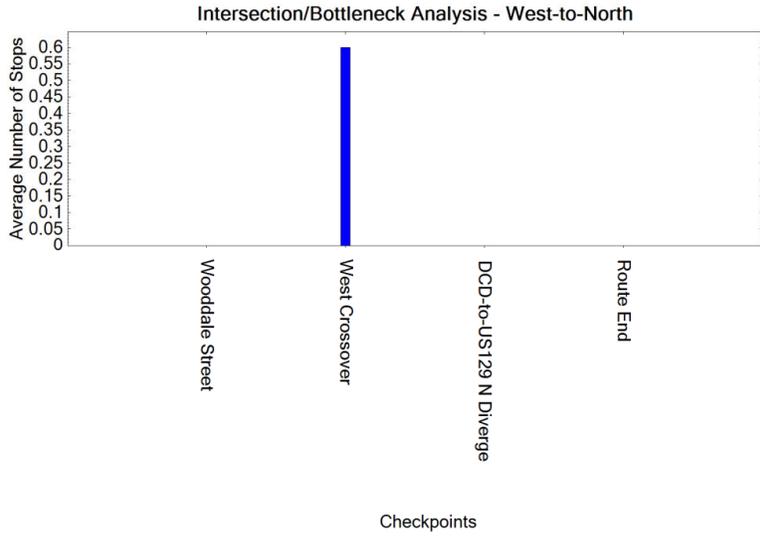


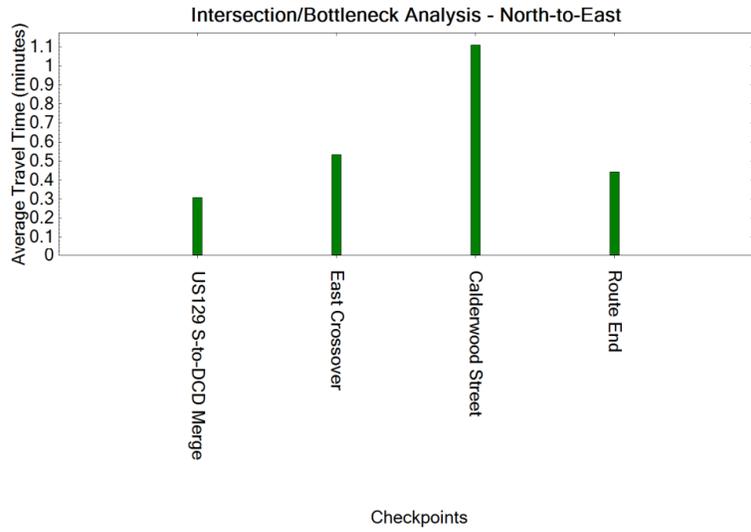
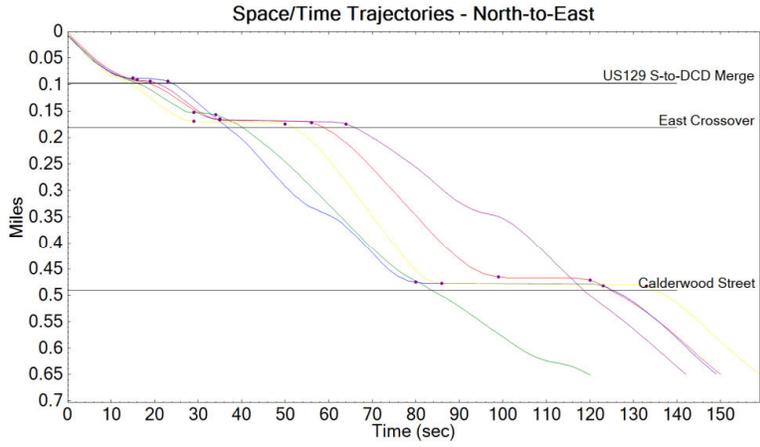


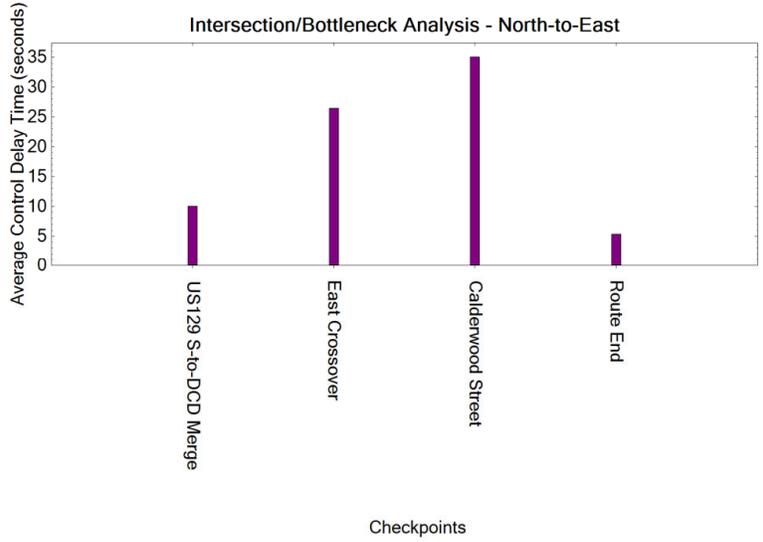
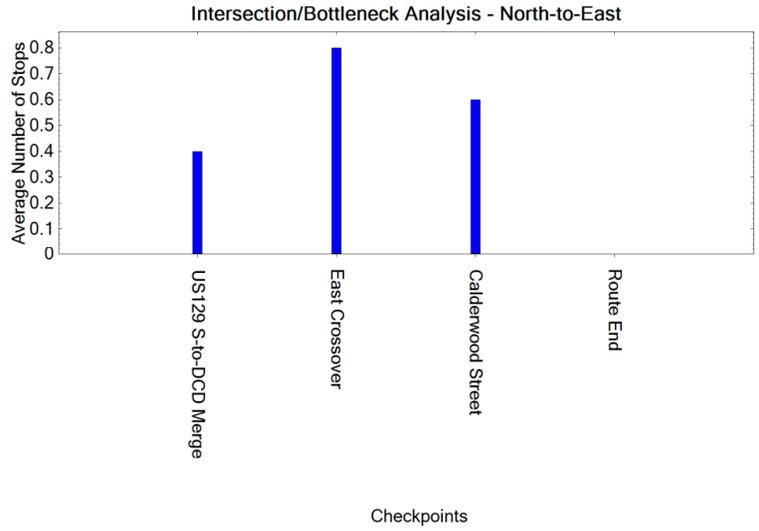




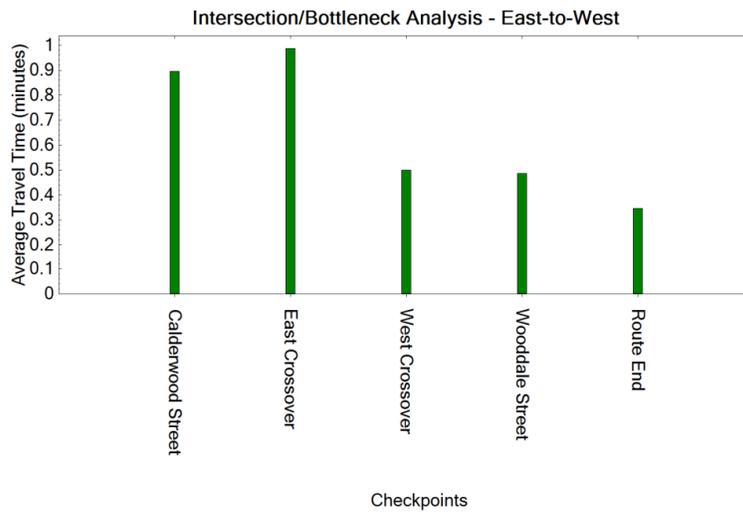
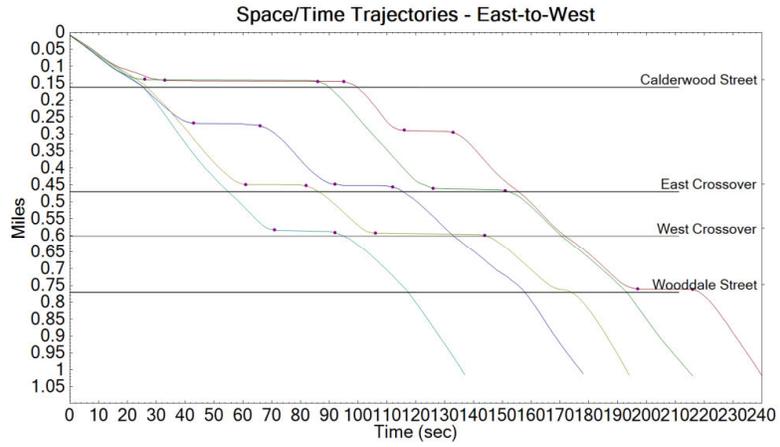


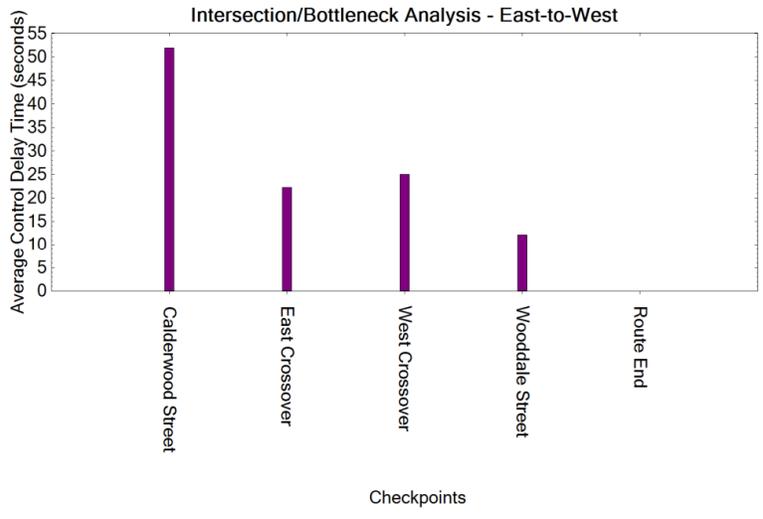
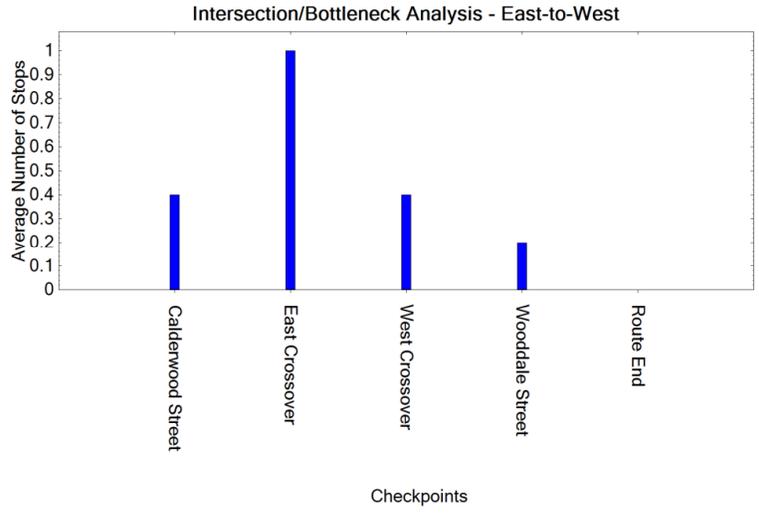


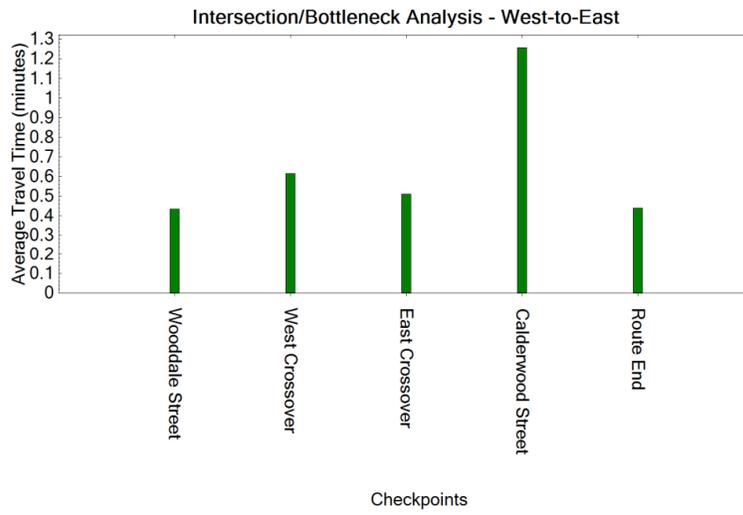
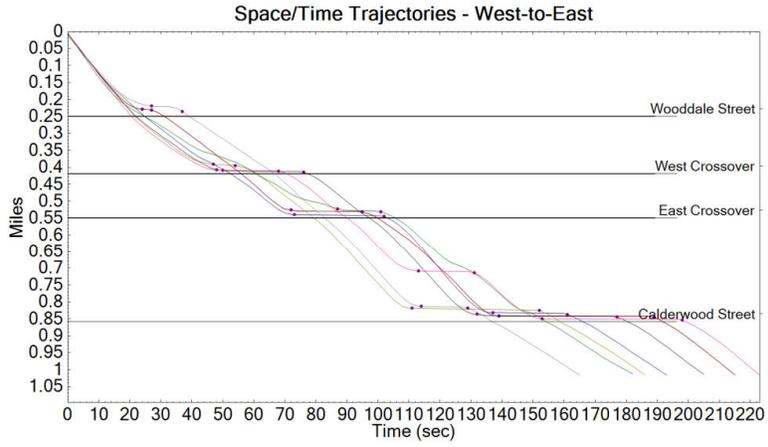


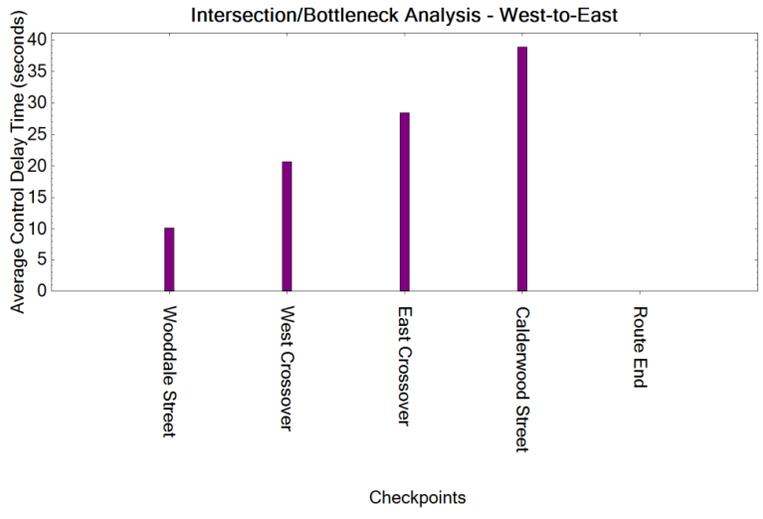
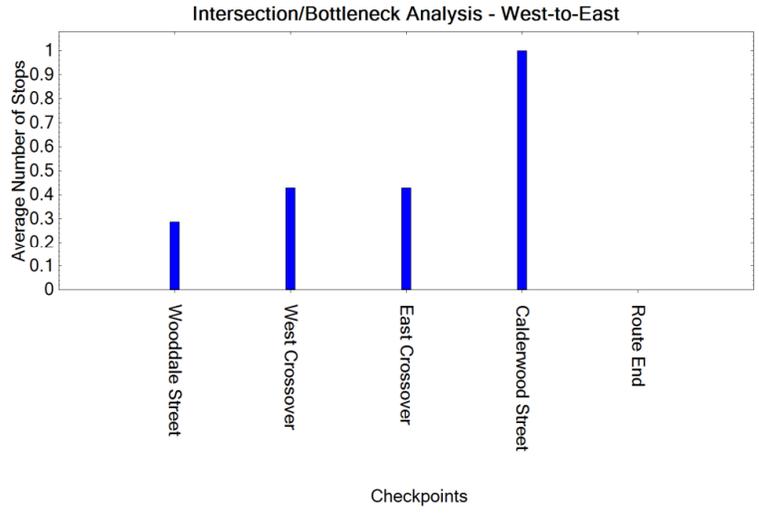


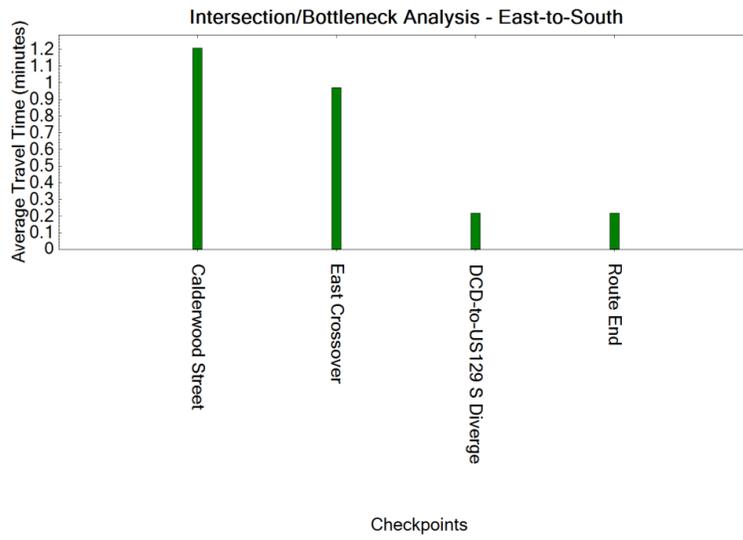
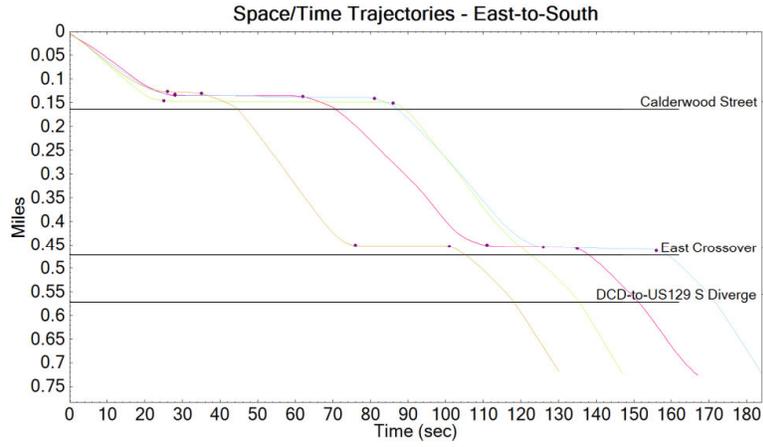
PM Peak Period After Data

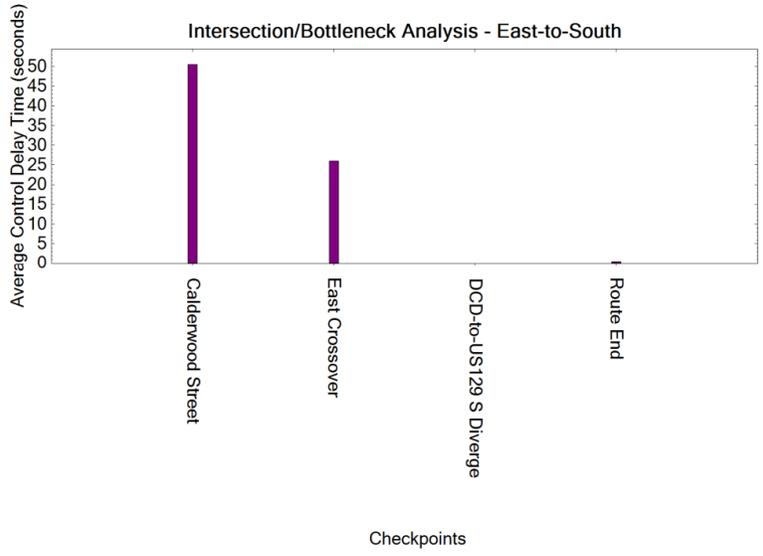
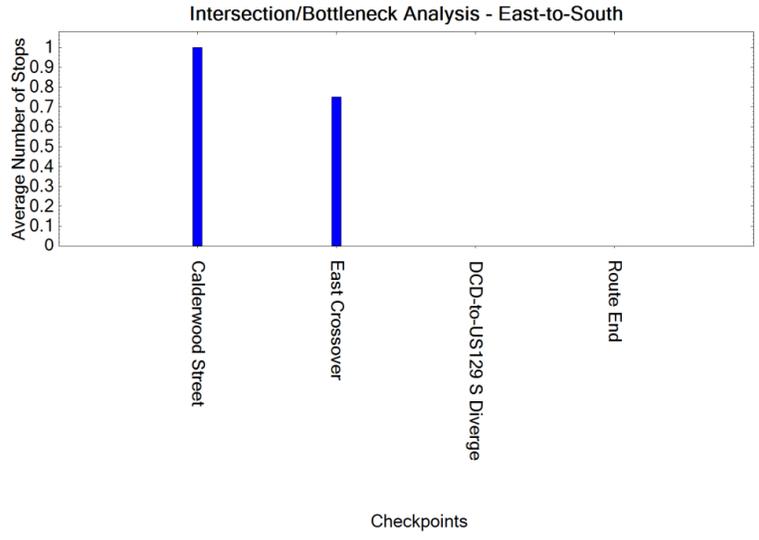


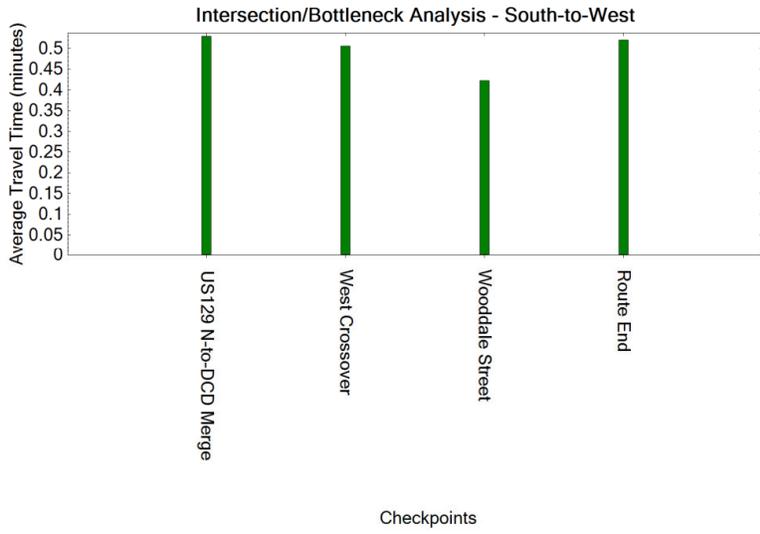
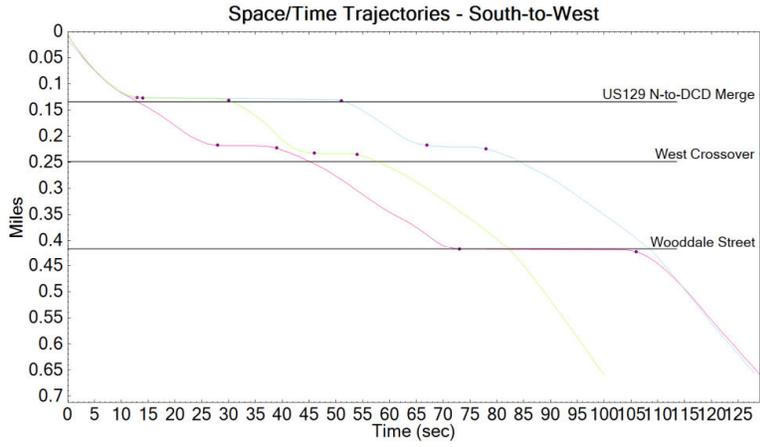


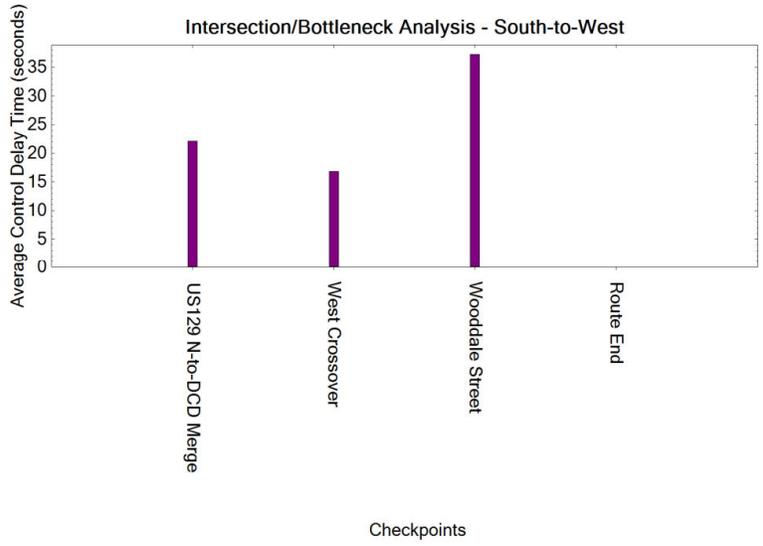
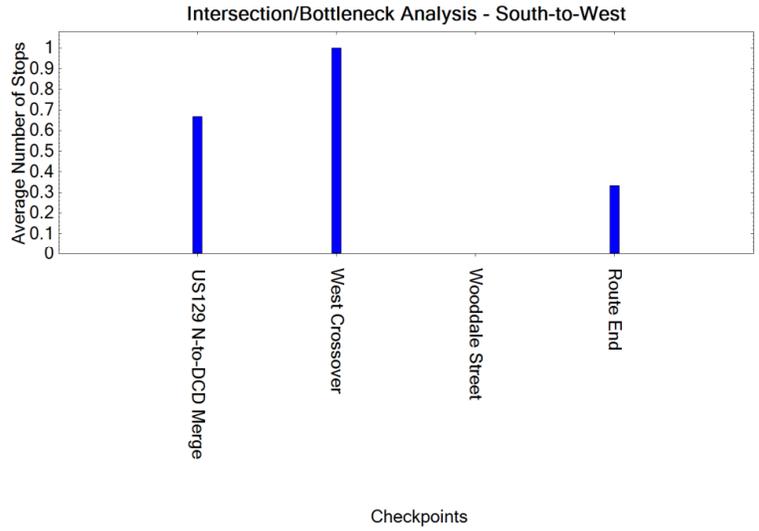


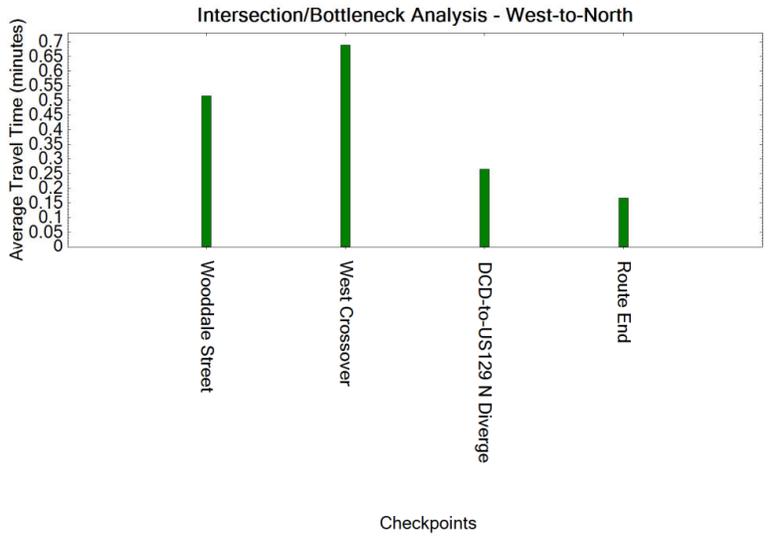
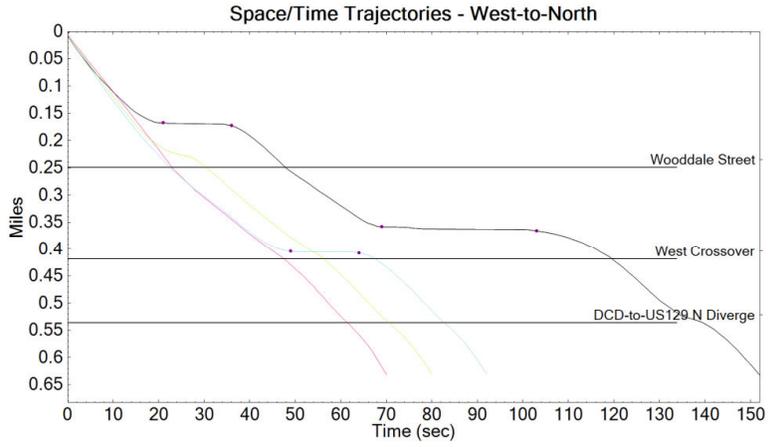


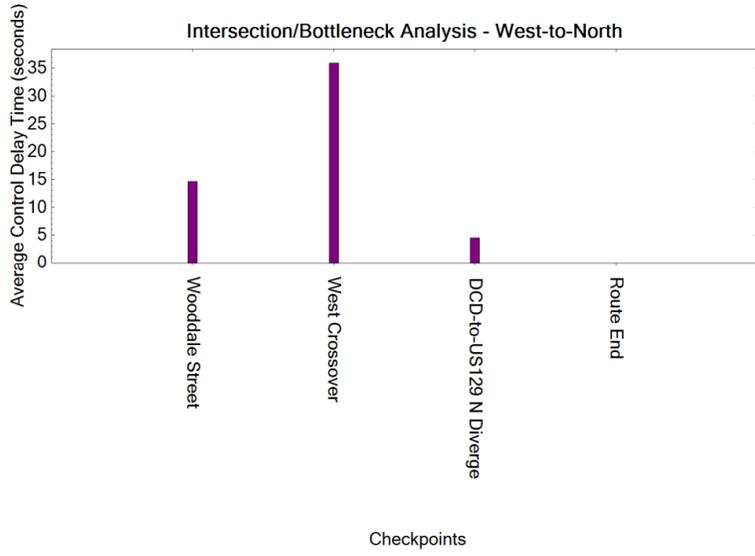
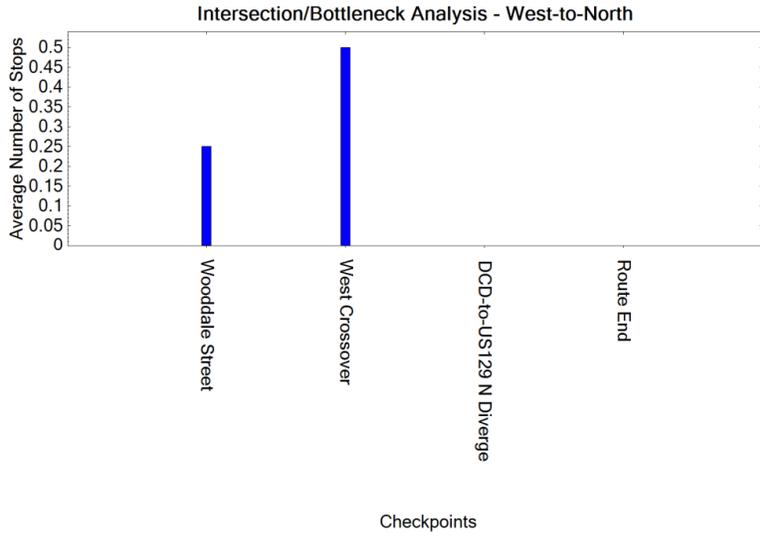


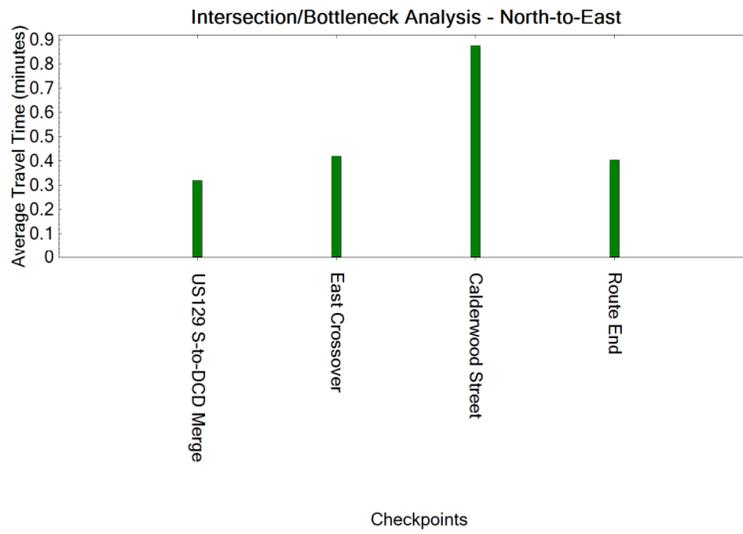
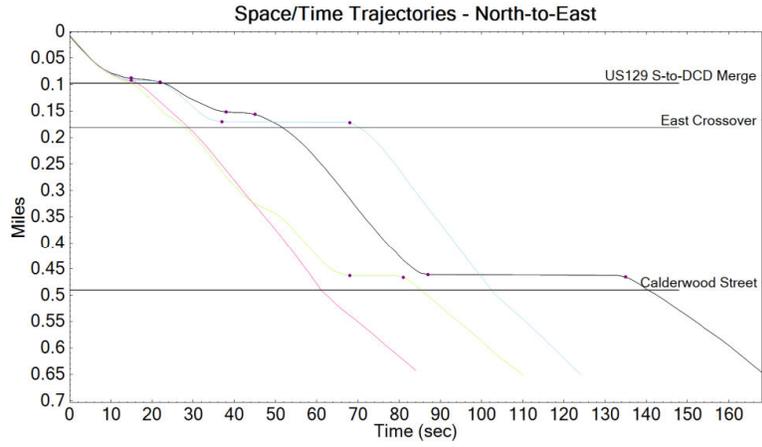


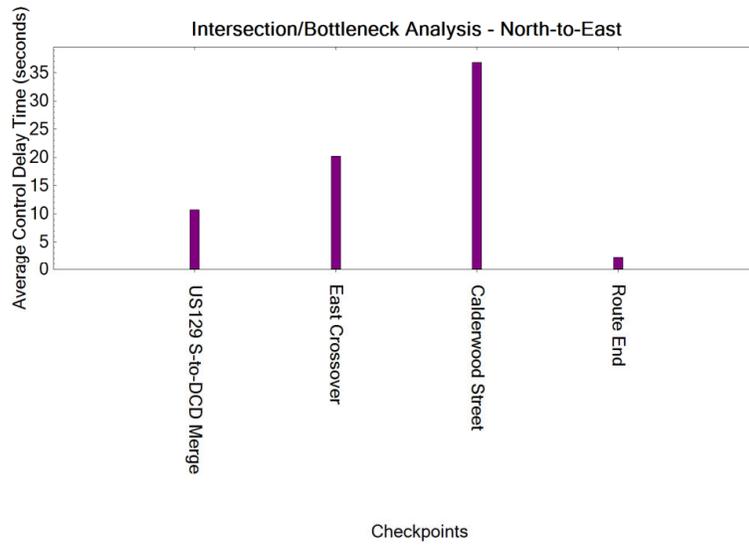
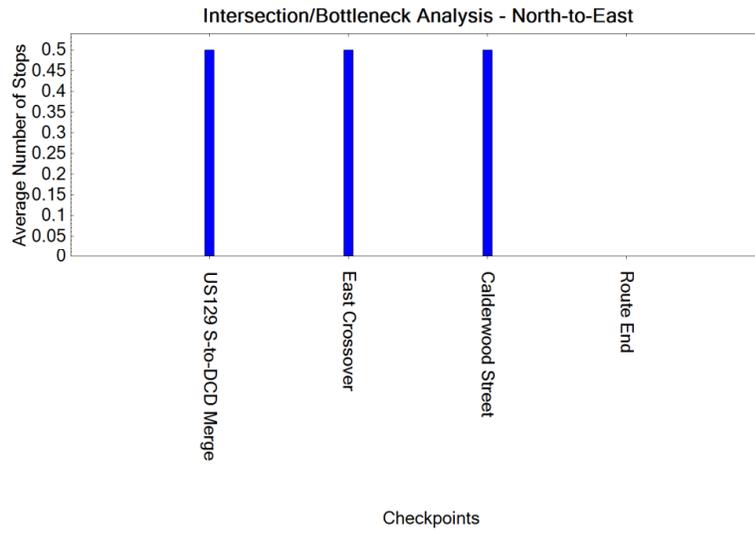










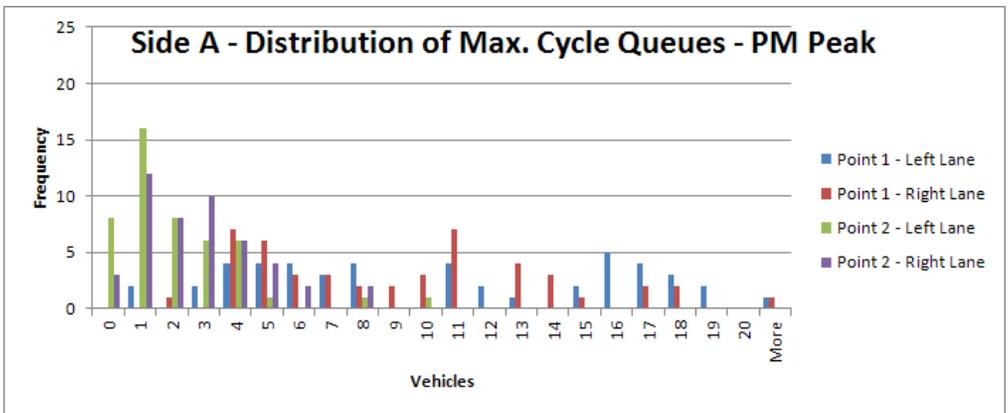
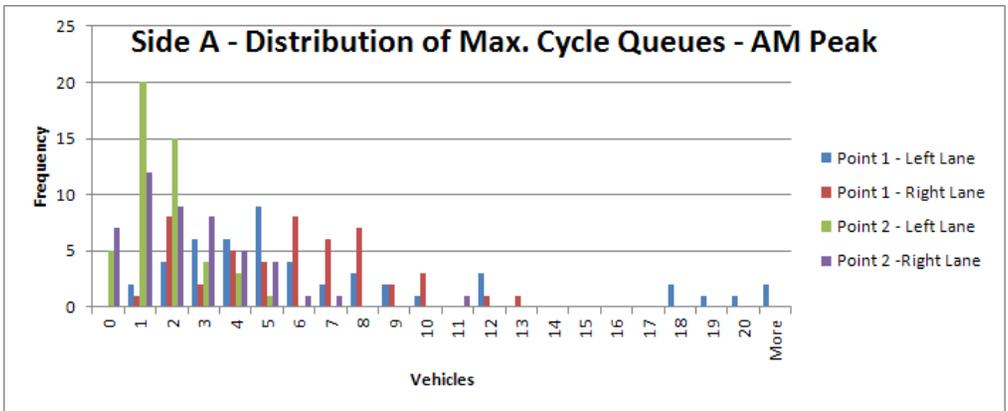


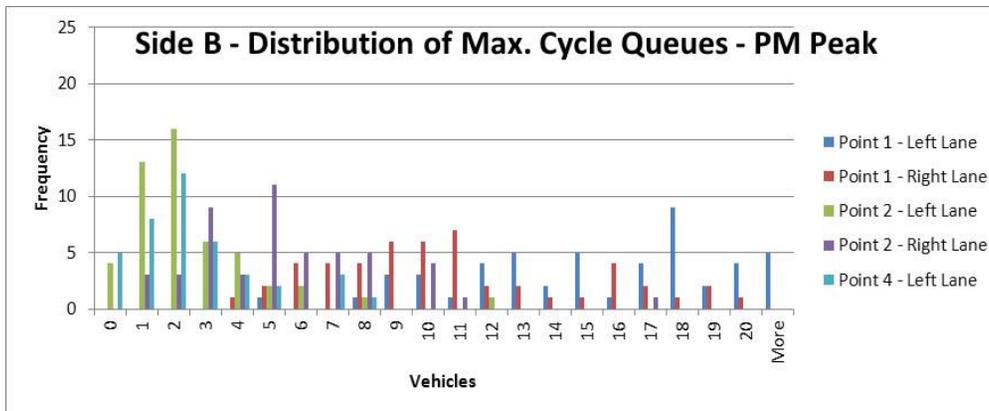
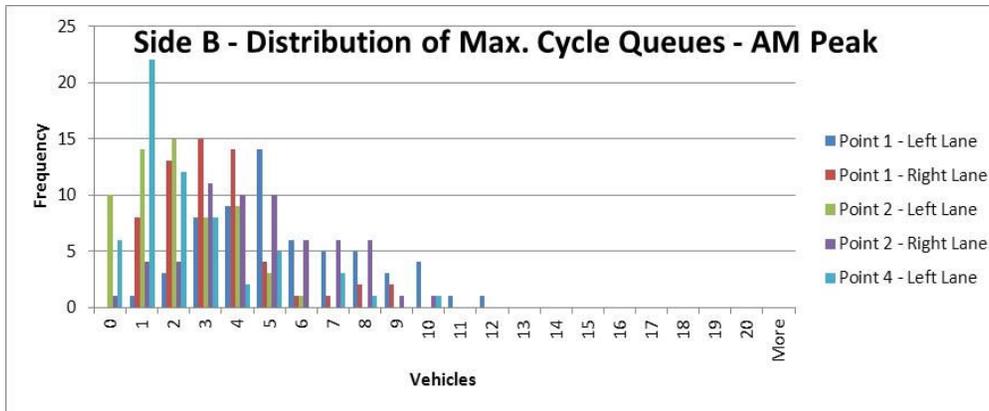
APPENDIX C:
Field Data from MO-13 DCD, Springfield, MO

Peak Period Queues – After

North Crossover (Side A)	AM Peak – 7:00am – 9:00am				PM Peak – 4:30am – 6:30am			
	Point 1 – SB Left Lane	Point 1 – SB Right Lane	Point 2 – NB Left Lane	Point 2 – NB Right Lane	Point 1 – SB Left Lane	Point 1 – SB Right Lane	Point 2 – NB Left Lane	Point 2 – NB Right Lane
Mean	7.5	5.9	1.6	2.5	10.6	9.4	2.1	2.8
Min	1	1	0	0	1	2	0	0
Max	32	13	5	11	24	22	10	8
Std.Dev	6.6	2.8	1.1	2.1	5.9	4.7	2.0	1.9
n	48	48	48	48	47	47	47	47

South Crossover (Side B)	AM Peak – 7:00am – 9:00am					PM Peak – 4:30am – 6:30am				
	Point 1 – NB Left Lane	Point 1 – NB Right Lane	Point 2 – SB Left Lane	Point 2 – SB Right Lane	Point 4 – EB Left Lane	Point 1 – NB Left Lane	Point 1 – NB Right Lane	Point 2 – SB Left Lane	Point 2 – SB Right Lane	Point 4 – EB Left Lane
Mean	5.6	3.4	2.1	4.7	2.4	15.5	10.8	2.5	5.5	2.5
Min	1	1	0	0	0	5	4	0	1	0
Max	12	9	6	10	10	24	20	12	17	8
Std.Dev	2.5	1.9	1.5	2.2	2.2	4.3	4.1	2.2	3.0	2.1
n	60	60	60	60	60	50	50	50	50	40

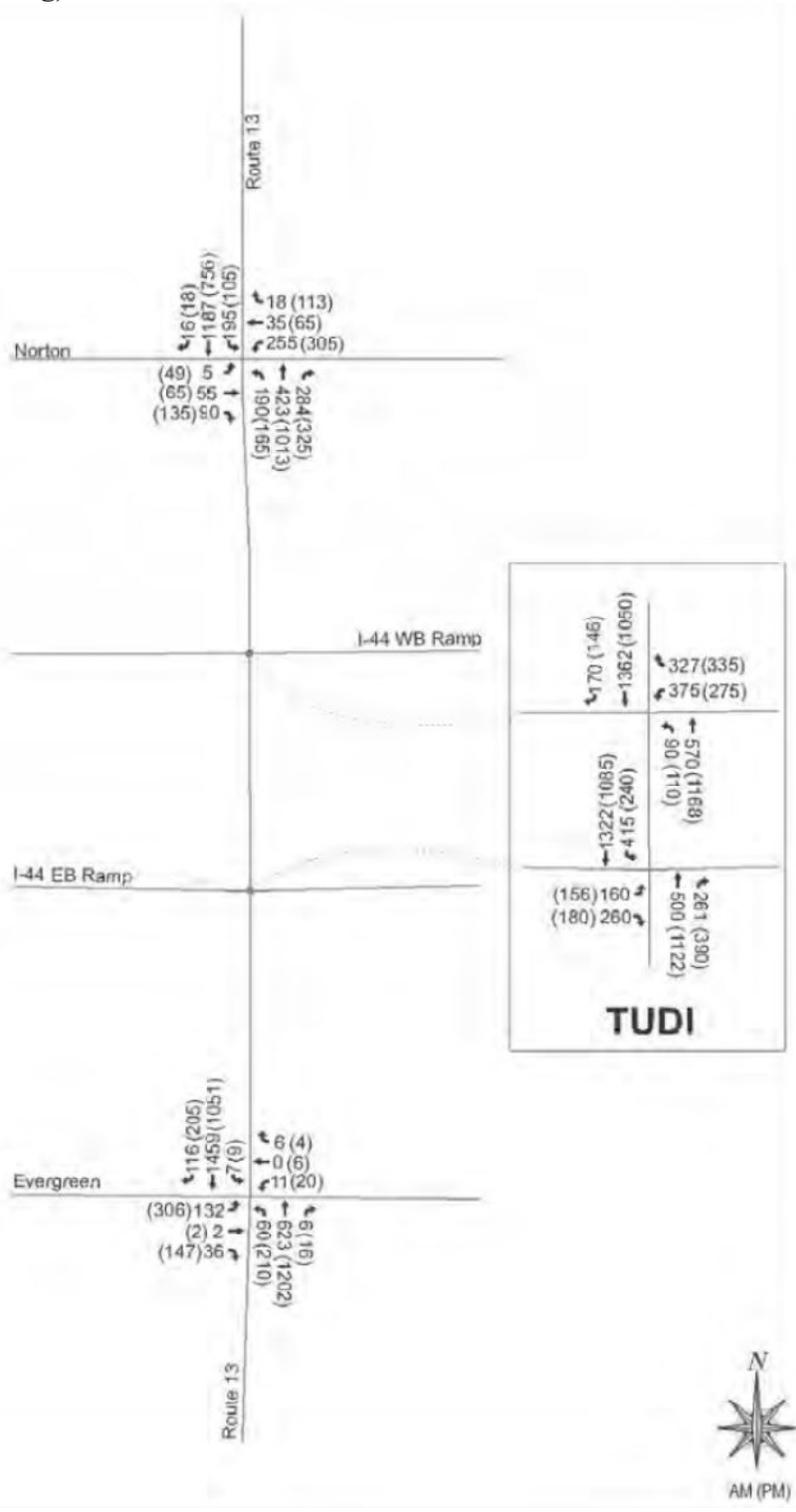




Saturation Flow Rate Calculation - After

DCD Site	Scenario	Crossover	Direction	Lane	Sat. Flow (vph)	Number of Cycles Observed	Number of Vehicles Observed
MO-13	After	South	NB	Left	1662	38	228
				Right	1668	35	194
		North	SB	Left	1400	20	103
				Right	1575	22	103

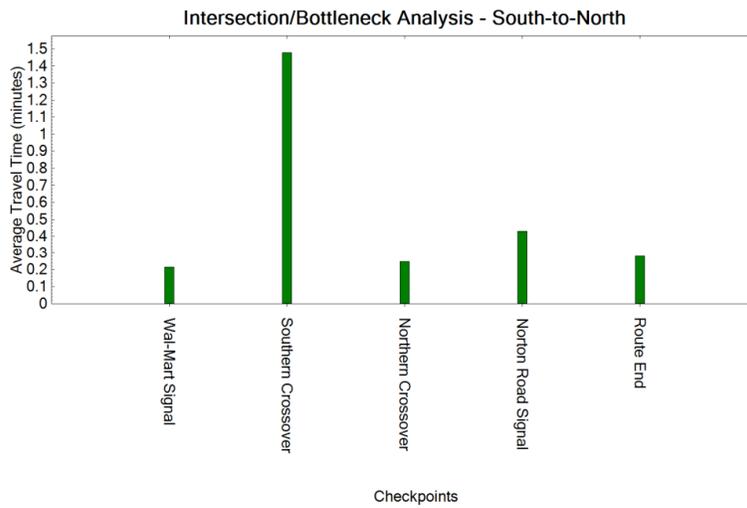
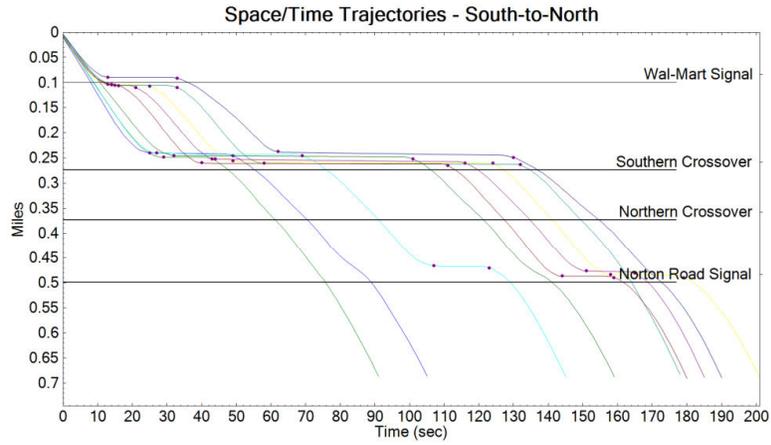
Peak Hour Turning Movement Counts – After (Source: MoDOT and HDR Engineering)

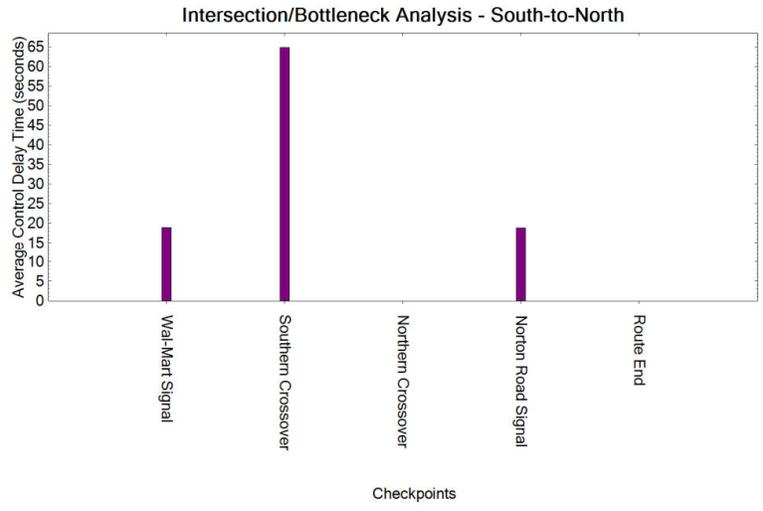
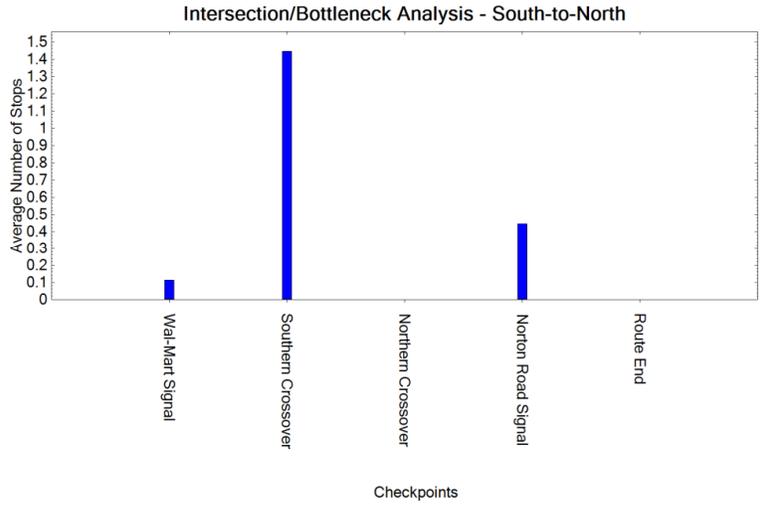


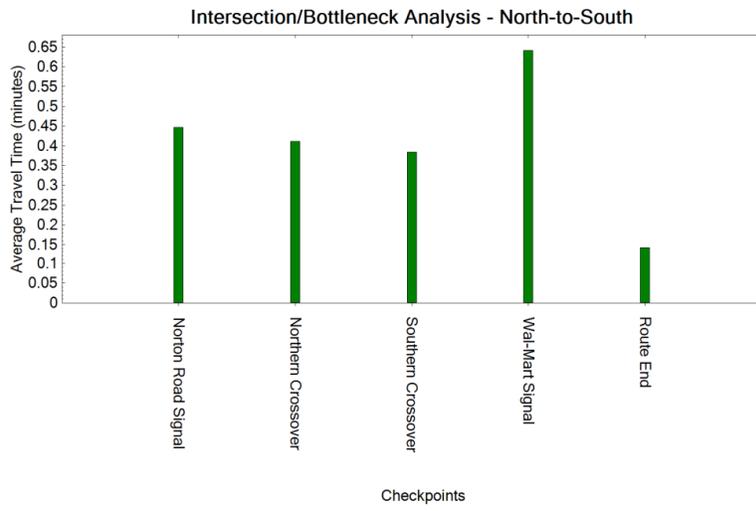
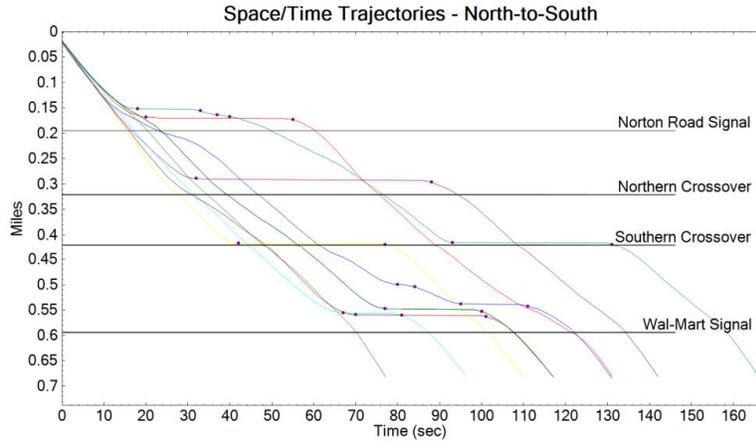
APPENDIX D:

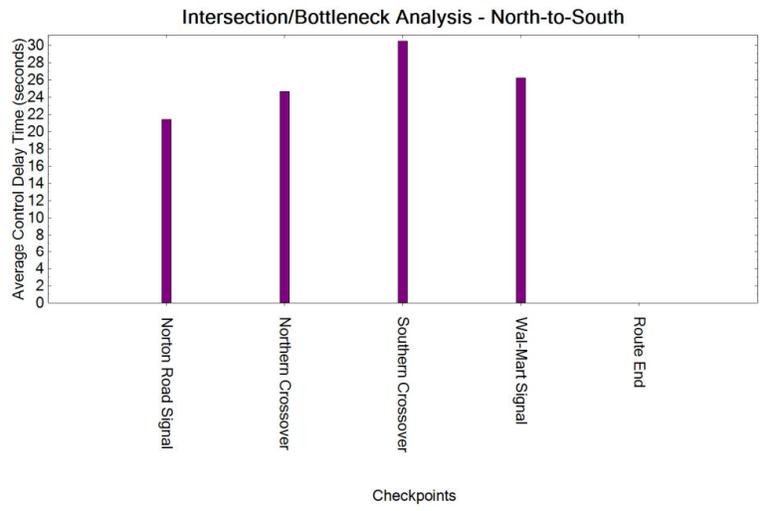
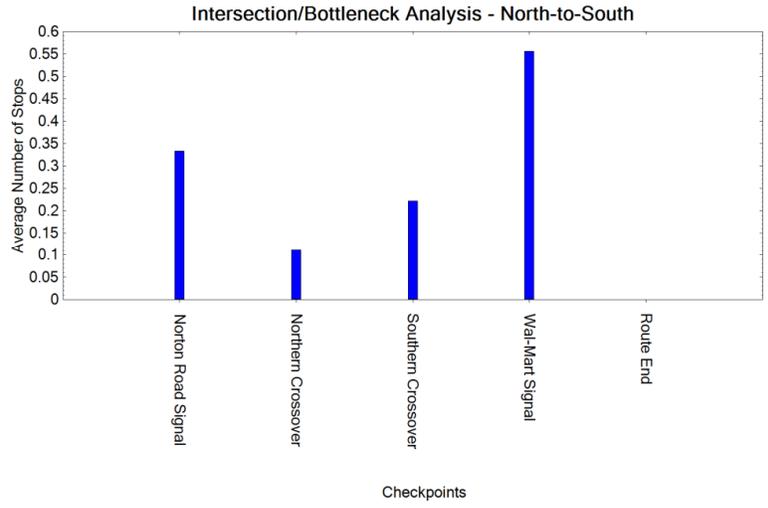
Travel Time Data from MO-13 DCD, Springfield, MO

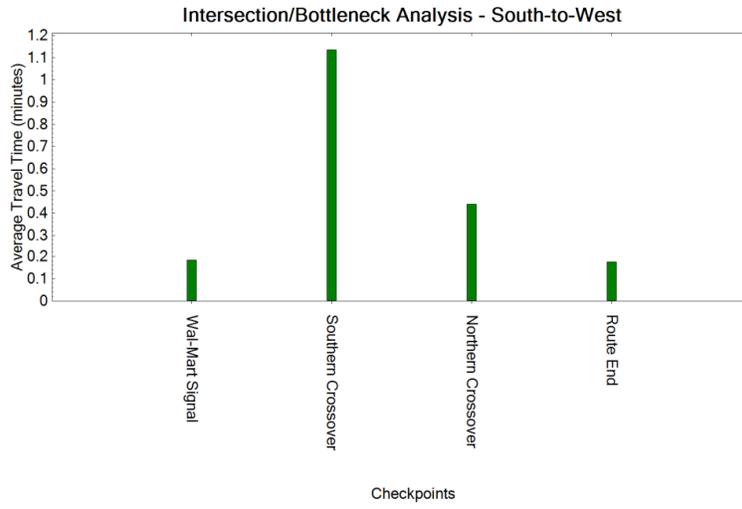
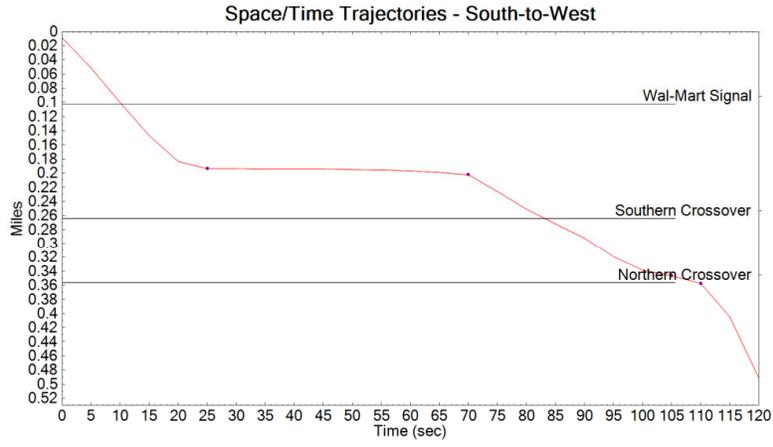
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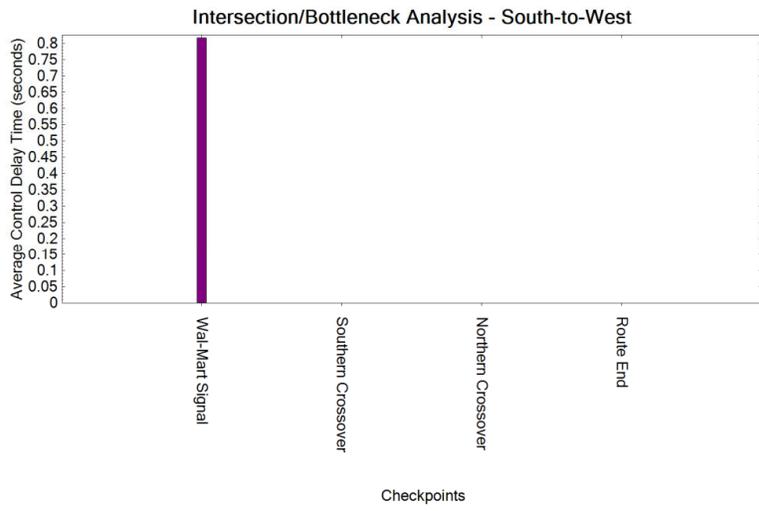
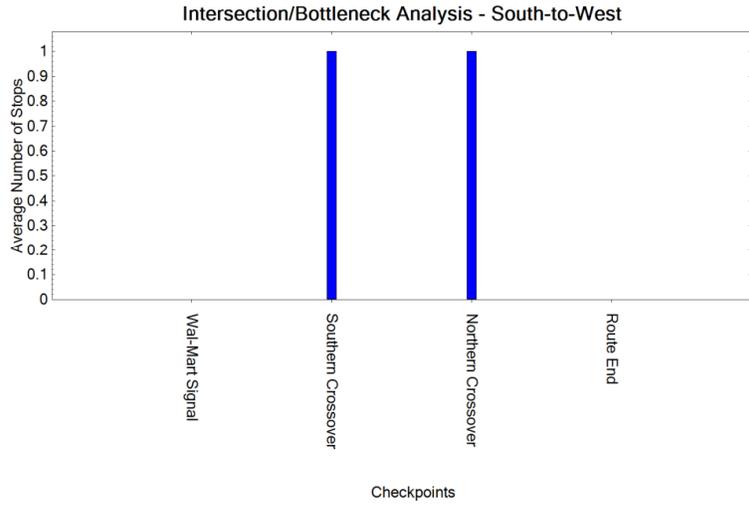


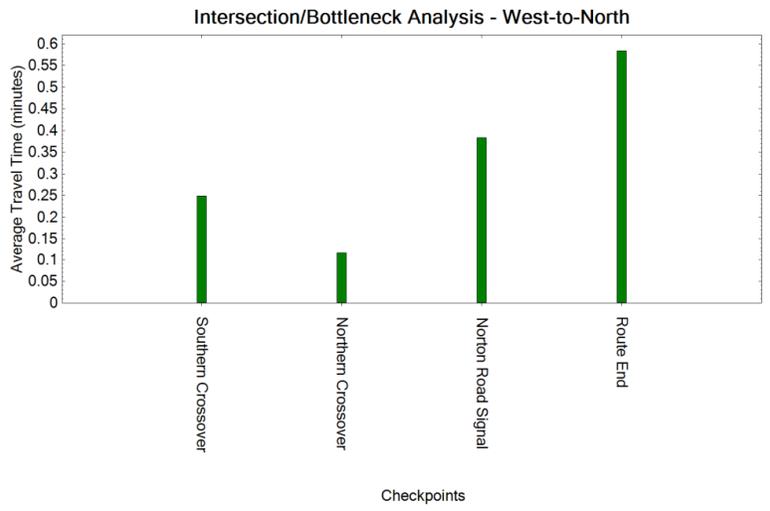
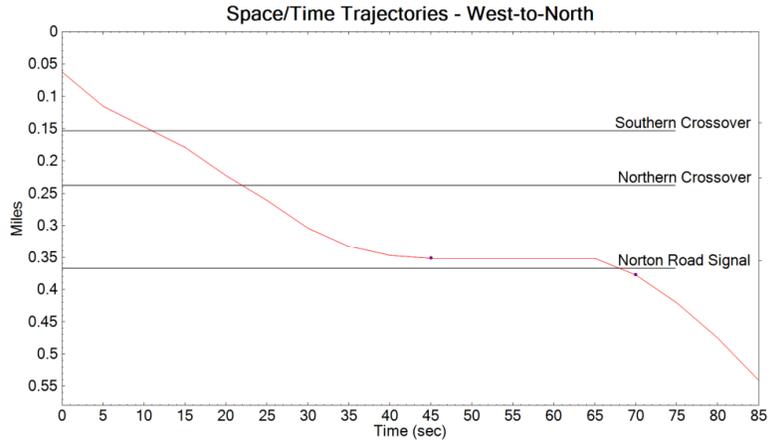


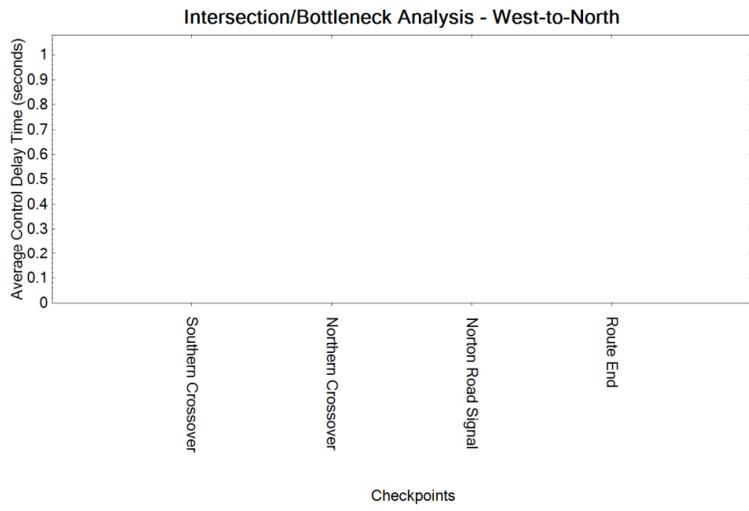
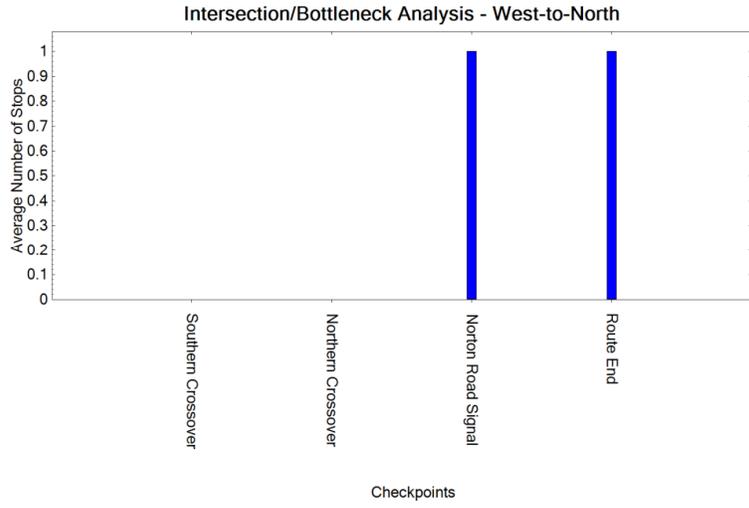


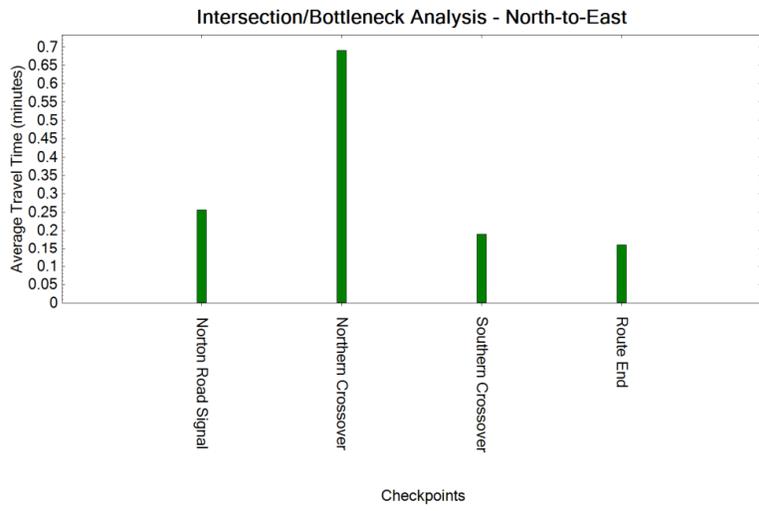
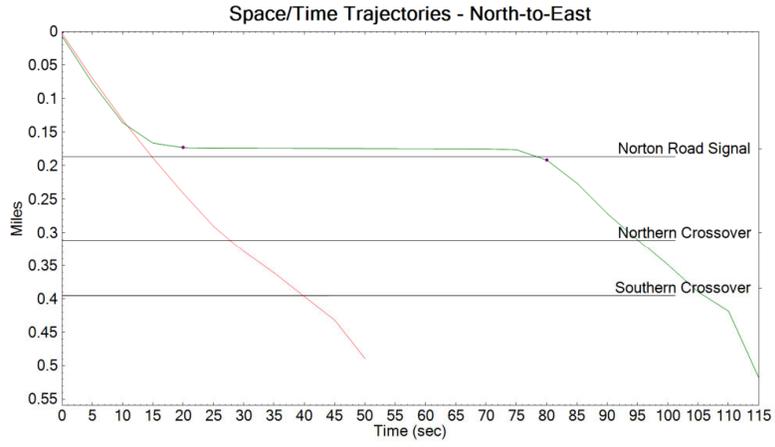


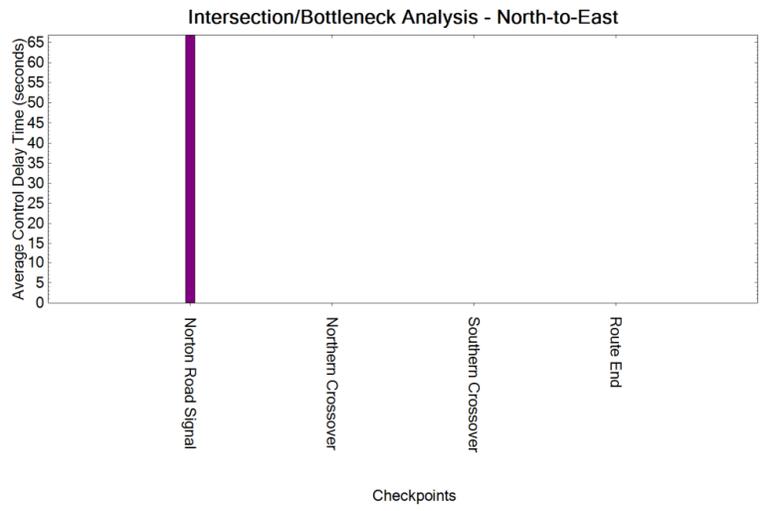
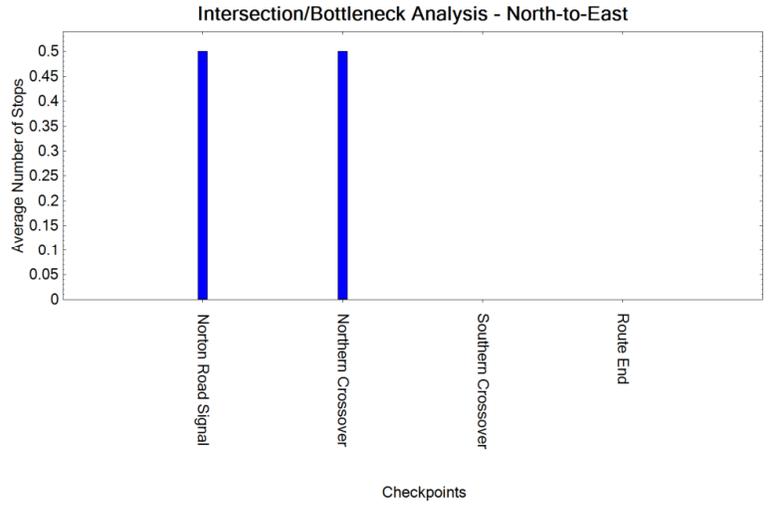


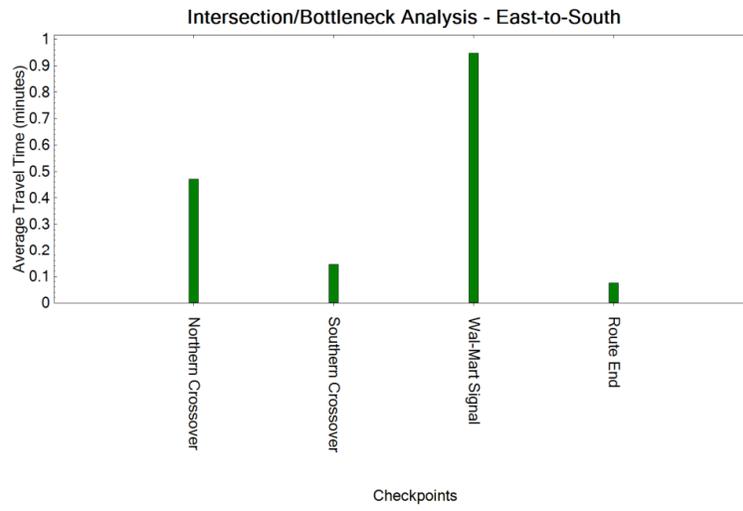
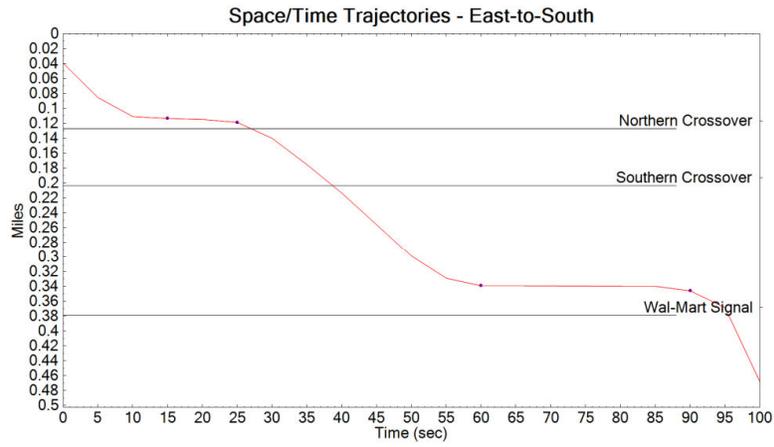


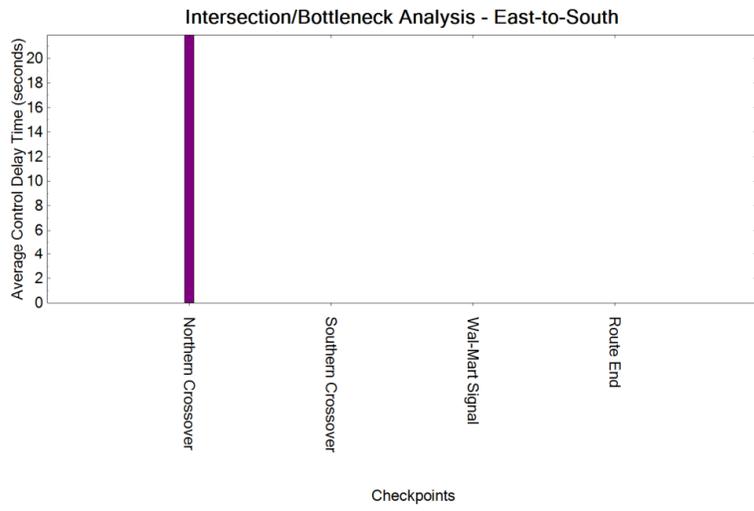
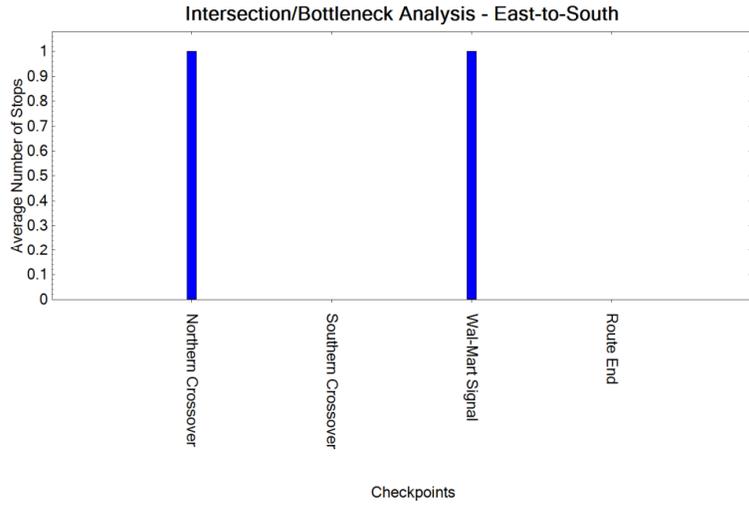




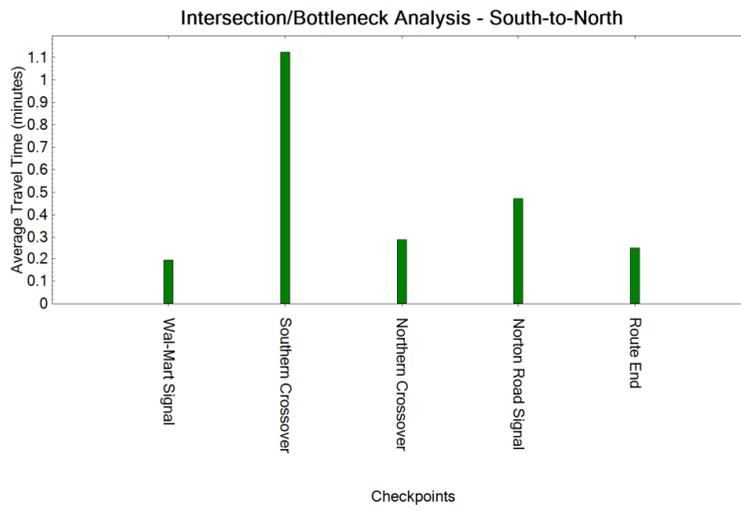
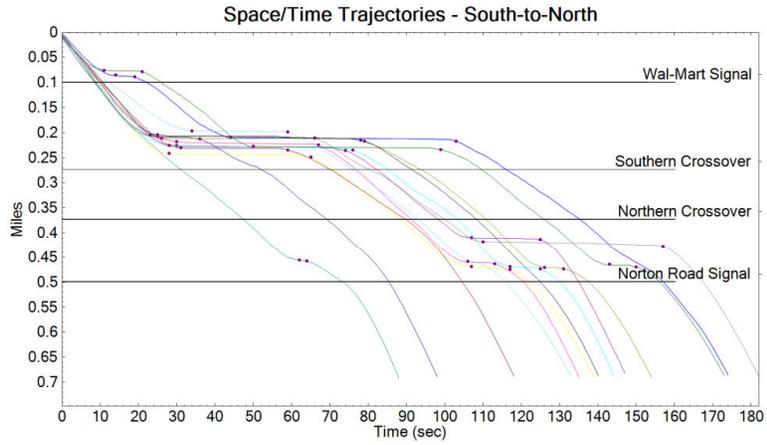


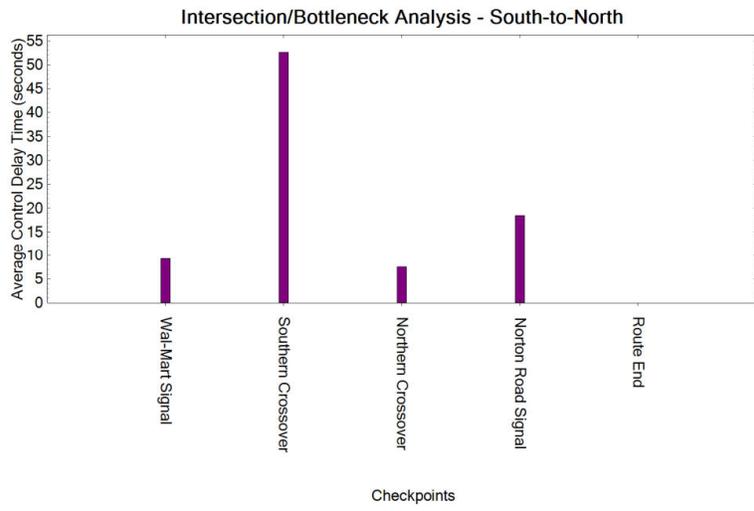
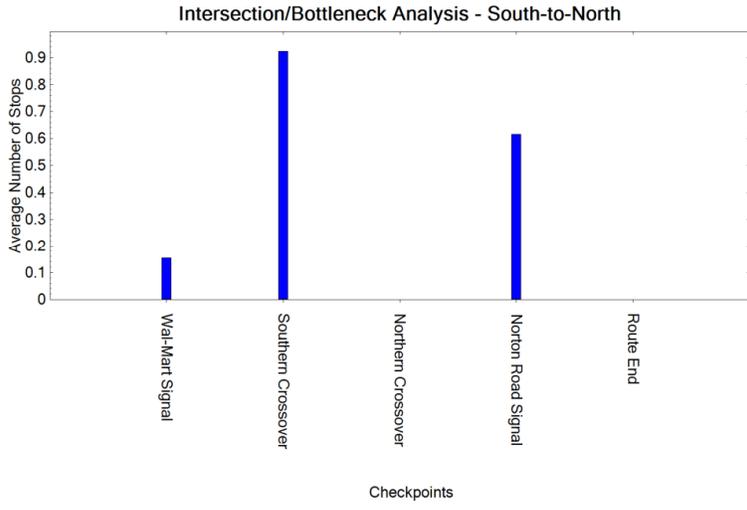


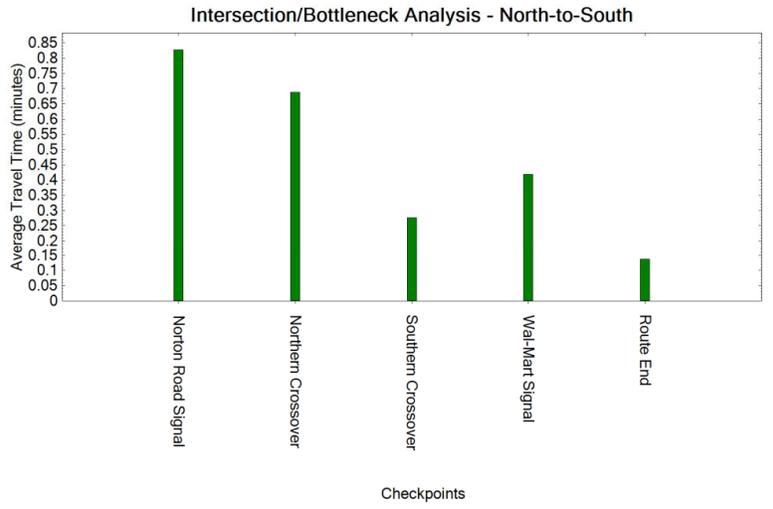
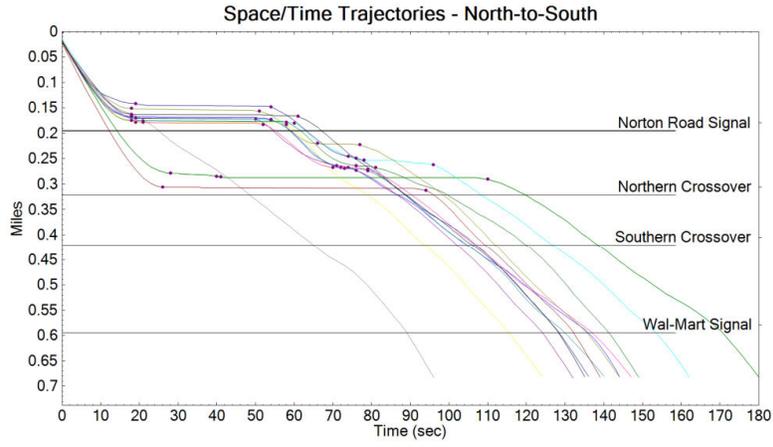


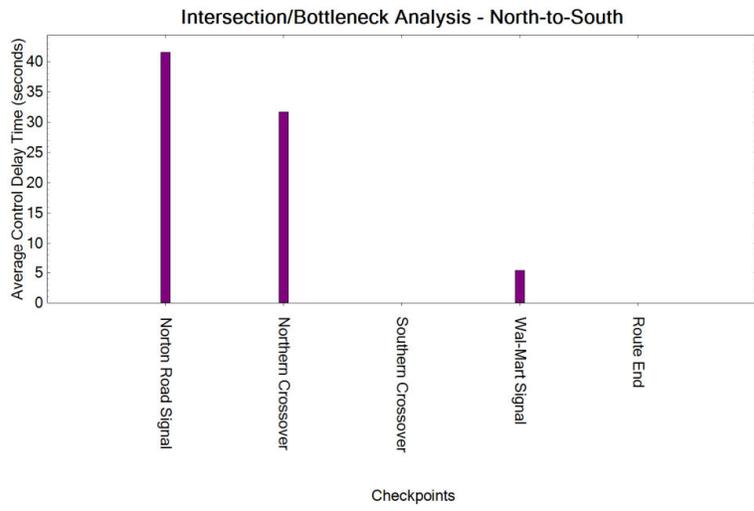
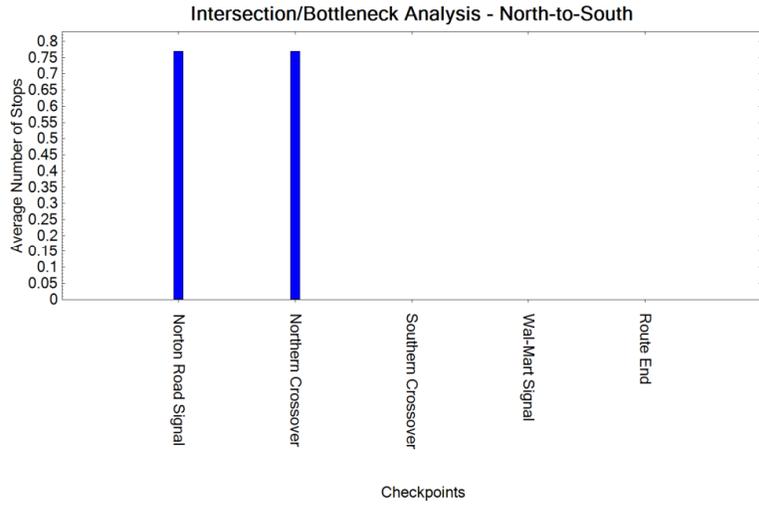


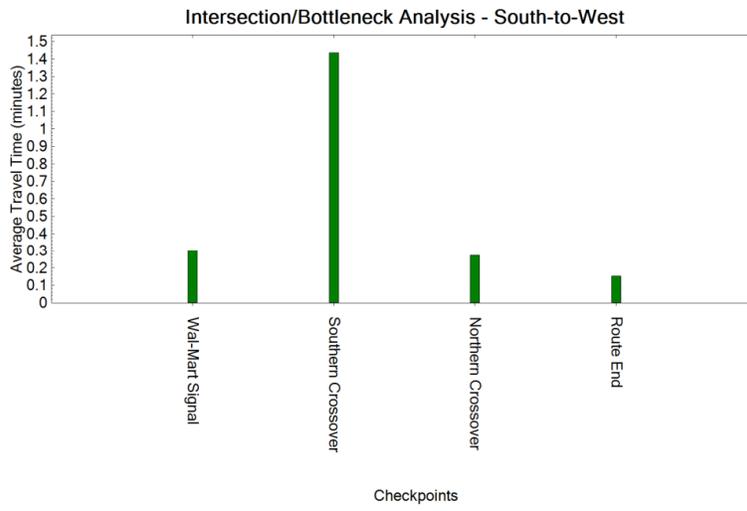
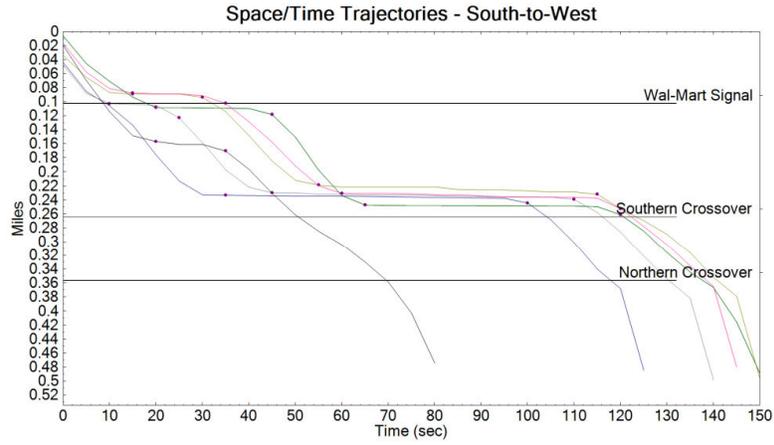
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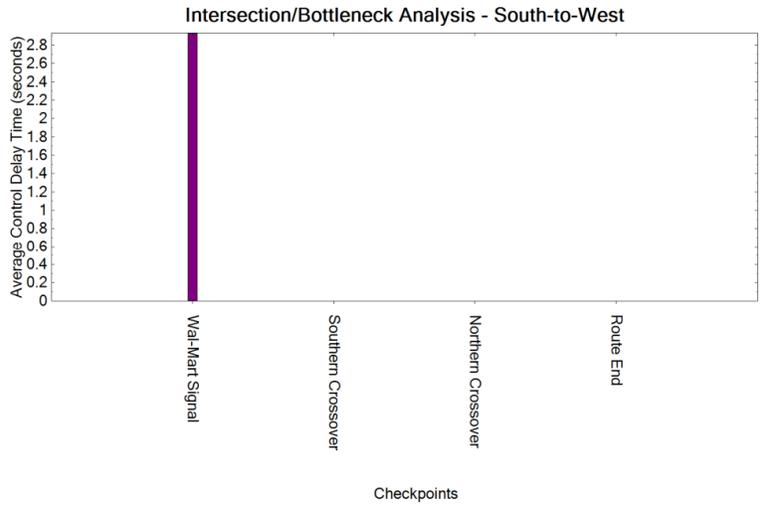
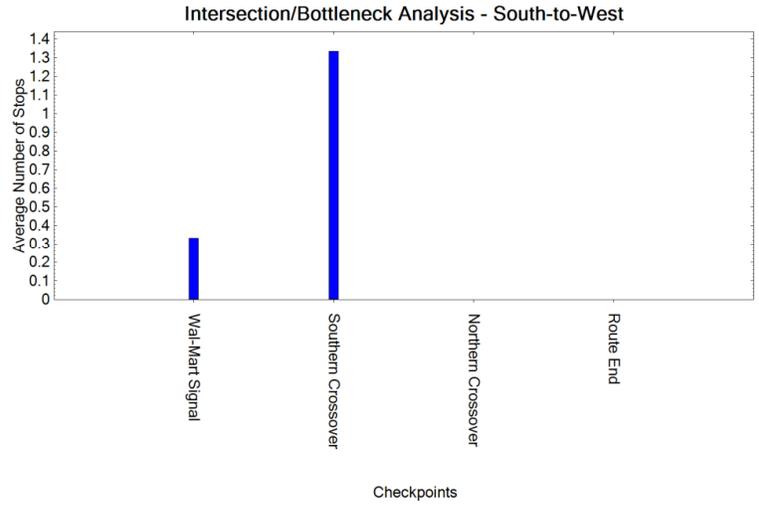


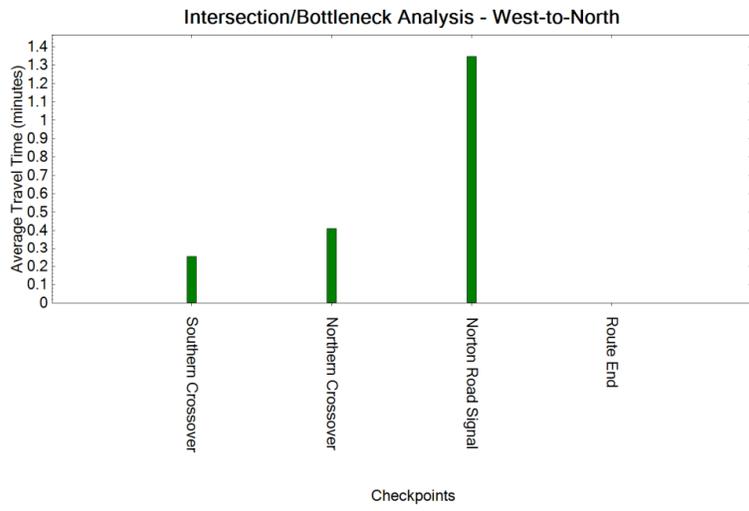
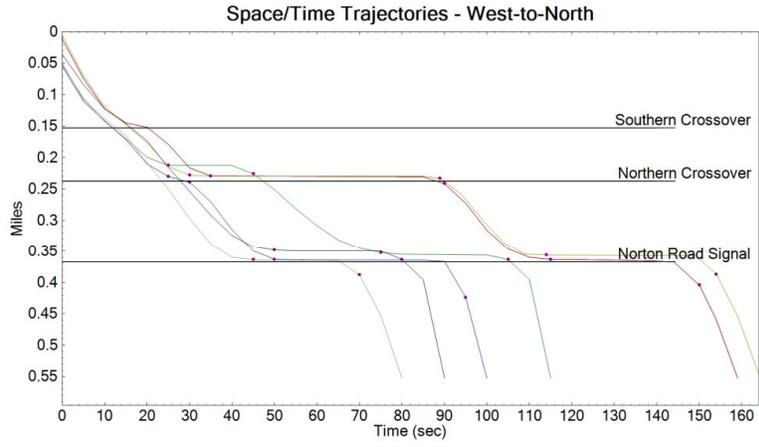


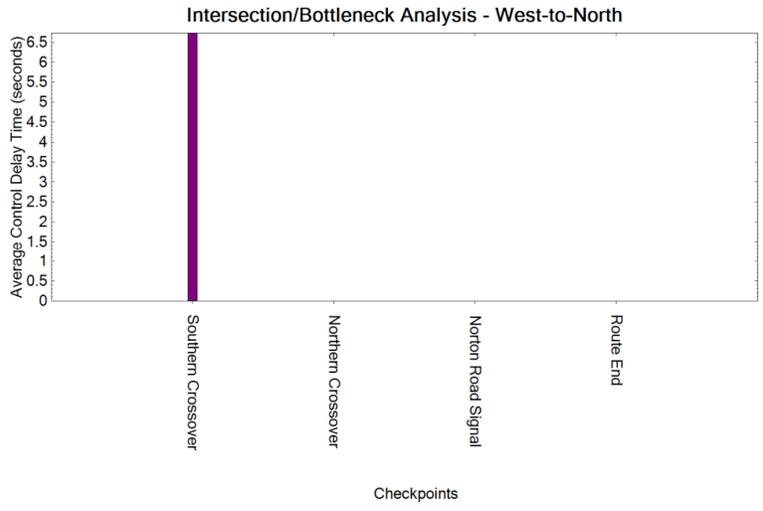
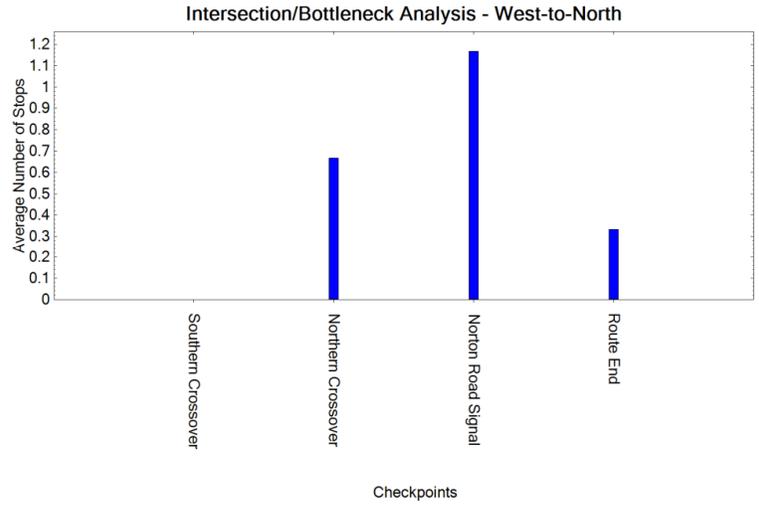


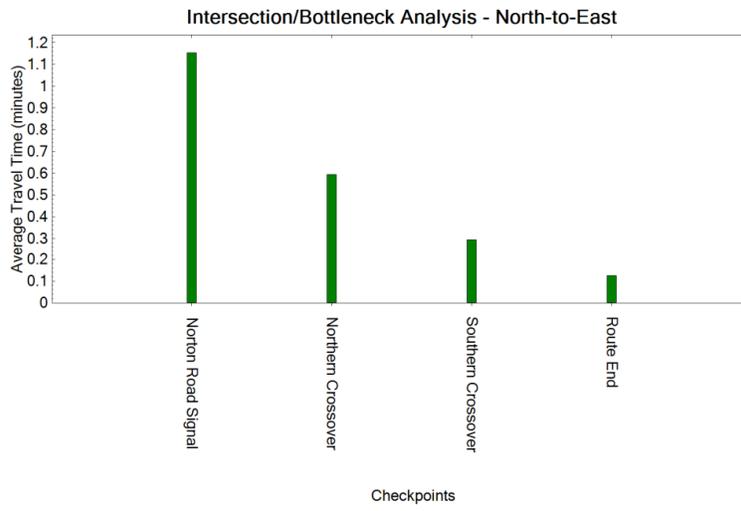
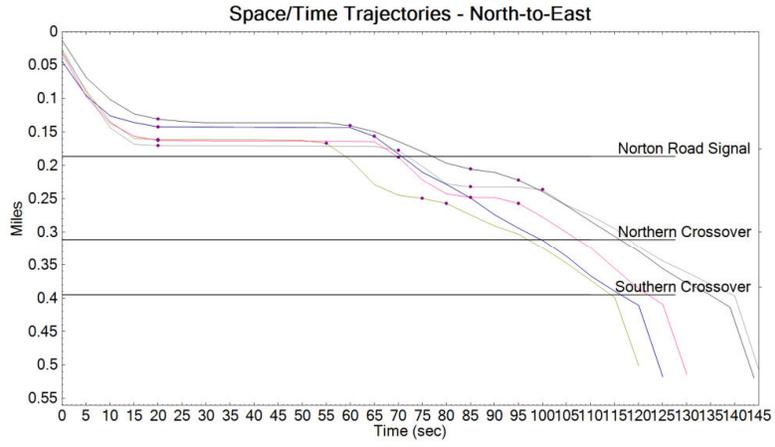


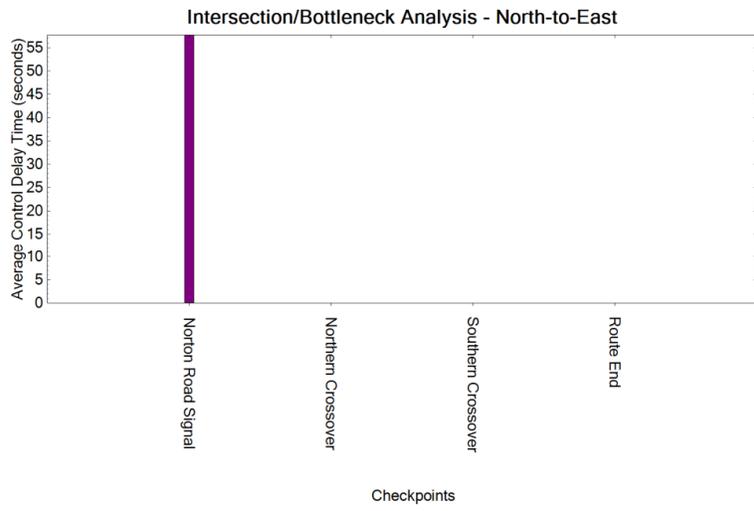
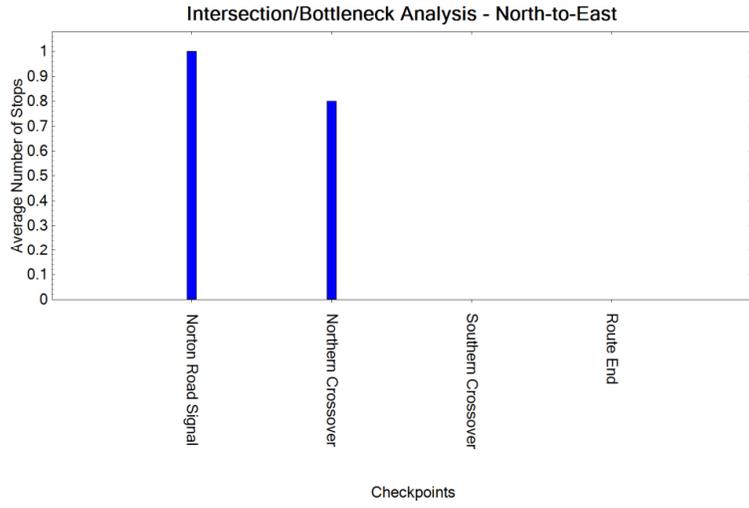


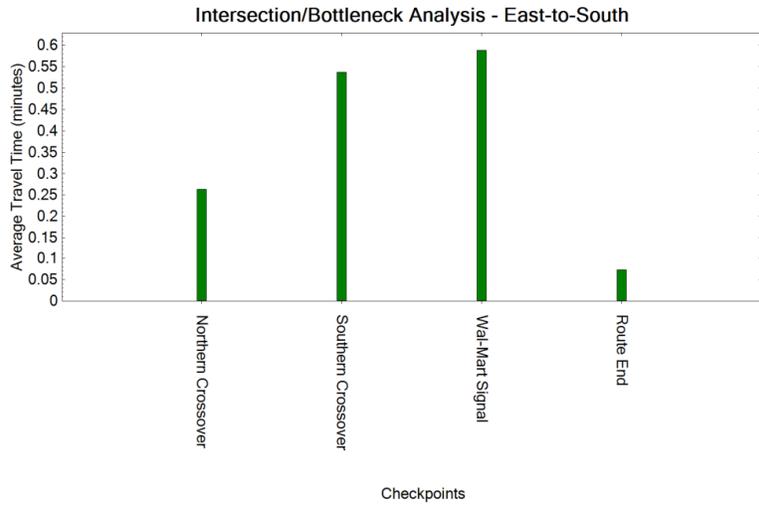
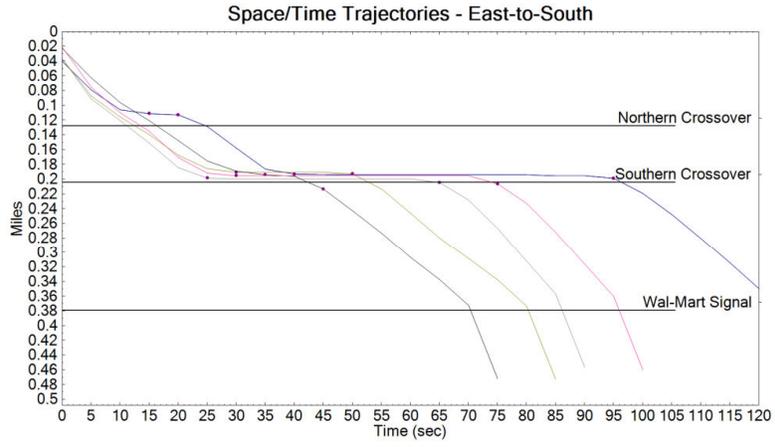


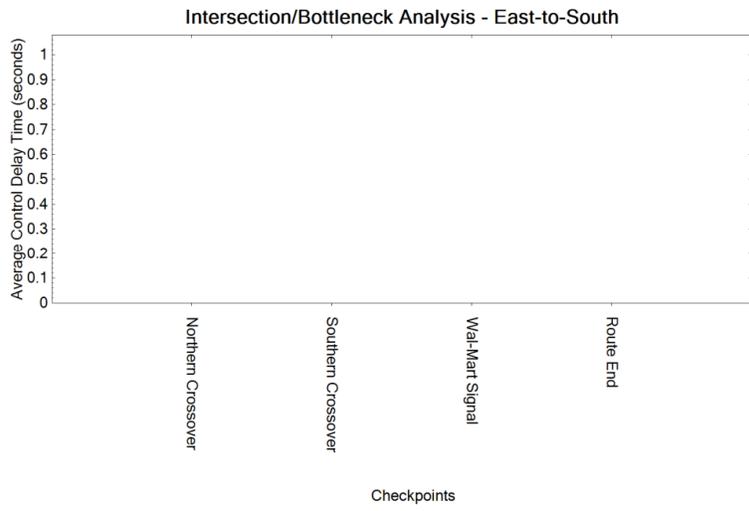
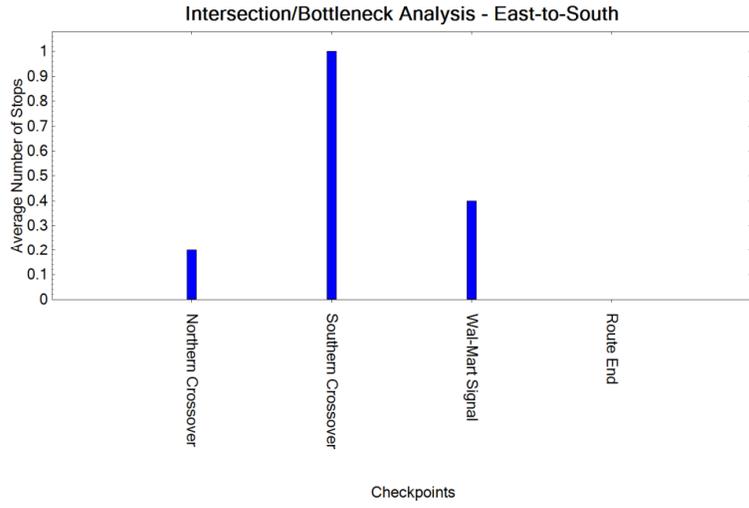










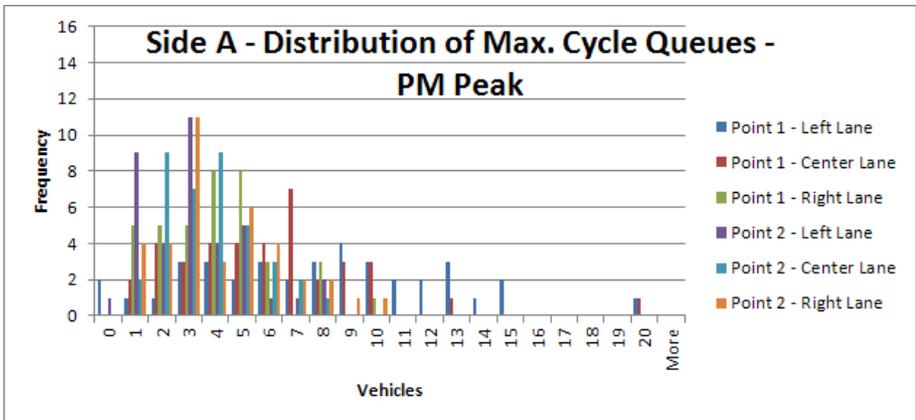
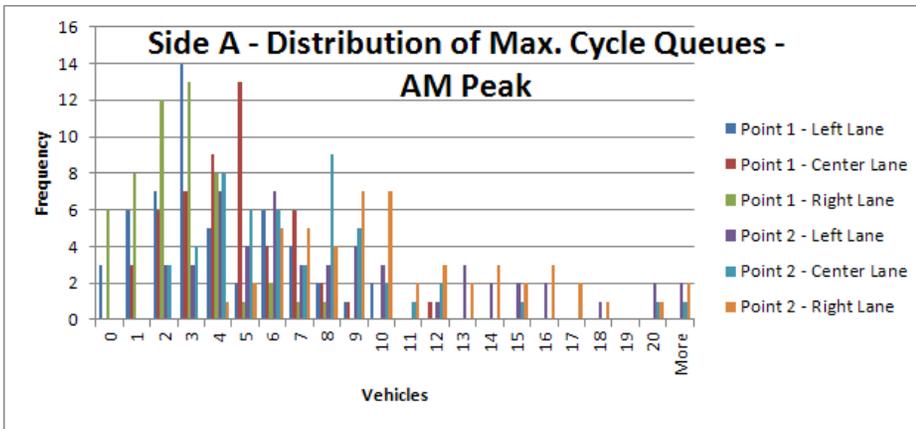


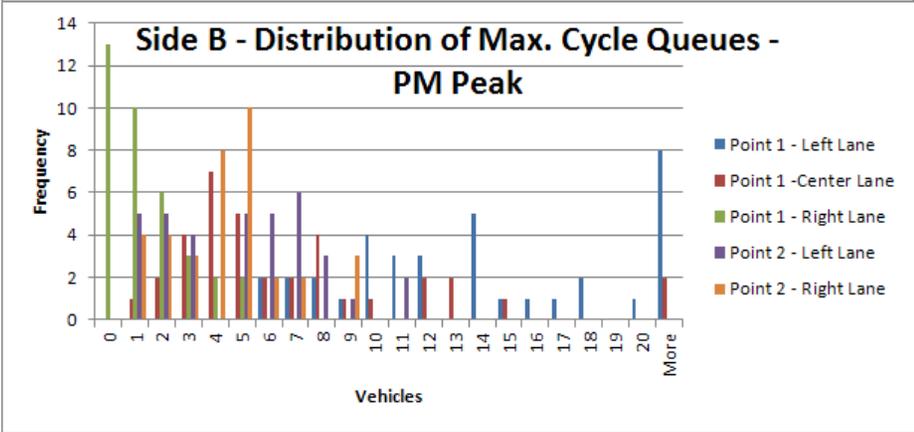
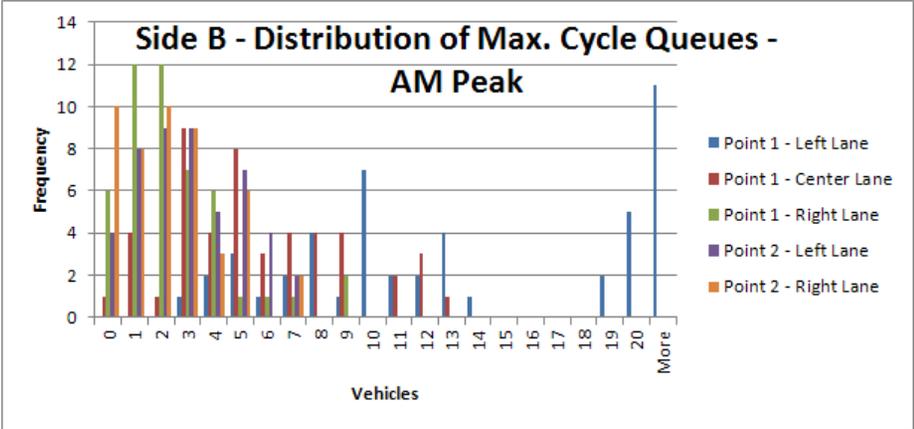
APPENDIX E:

Field Data from National Avenue, Springfield, MO

Peak Period Queues – After

North Crossover (Side A)	AM Peak – 7:00am – 9:00am						PM Peak – 4:30am – 6:30am					
	SB Left Lane	SB Center Lane	SB Right Lane	NB Left Lane	NB Center Lane	NB Right Lane	SB Left Lane	SB Center Lane	SB Right Lane	NB Left Lane	NB Center Lane	NB Right Lane
Mean	3.9	4.6	2.6	8.8	7.0	10.8	8.0	6.2	4.1	3.2	3.7	4.2
Min	0	1	0	2	2	4	0	1	1	0	1	1
Max	10	12	8	22	22	22	20	20	10	8	8	10
Std.Dev	2.6	2.2	1.8	5.4	4.0	4.4	4.6	3.7	2.2	2.0	1.7	2.3
n	52	52	52	52	52	52	38	38	38	38	38	38
South Crossover (Side B)	AM Peak – 7:00am – 9:00am					PM Peak – 4:30am – 6:30am						
	NB Left Lane	NB Center Lane	NB Right Lane	SB Left Lane	SB Right Lane	NB Left Lane	NB Center Lane	NB Right Lane	SB Left Lane	SB Right Lane		
Mean	14.3	5.7	2.4	3.0	2.3	16.8	7.2	1.4	5.0	4.3		
Min	3	0	0	0	0	6	1	0	1	1		
Max	34	13	9	7	7	40	25	5	11	9		
Std.Dev	7.9	3.3	2.1	1.9	1.9	10.1	5.3	1.5	2.9	2.2		
n	48	48	48	48	48	36	36	36	36	36		





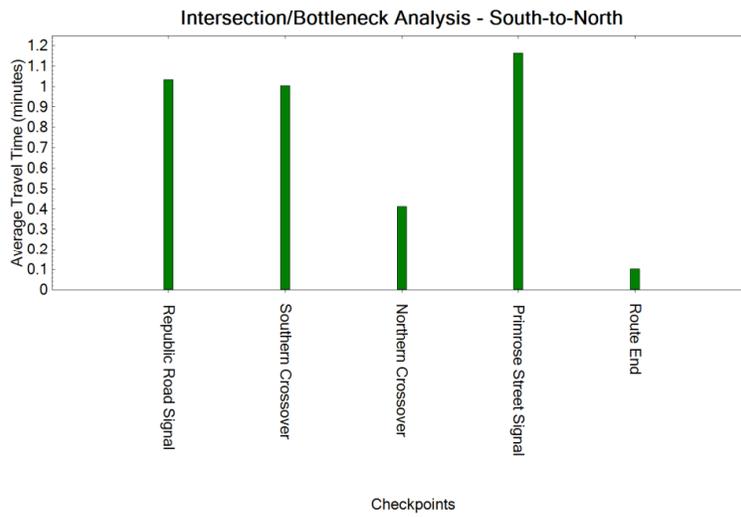
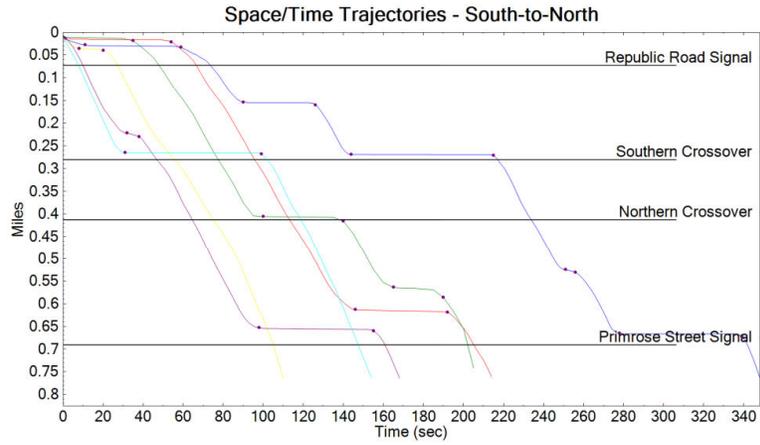
Saturation Flow Rate Calculation - After

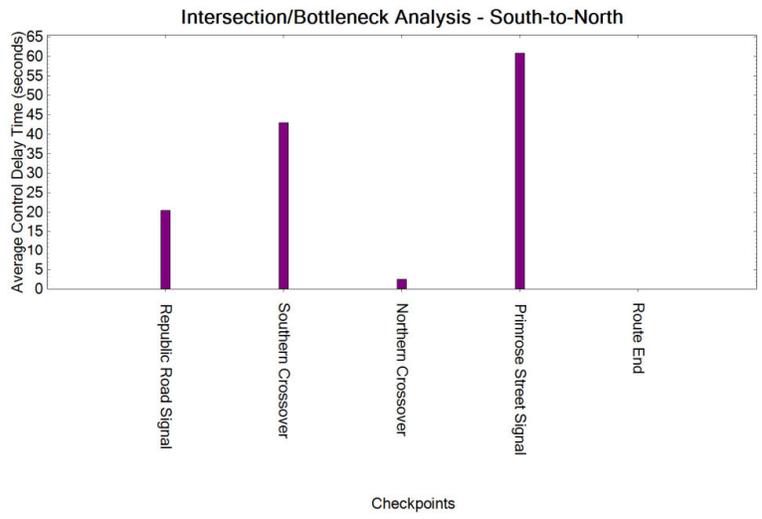
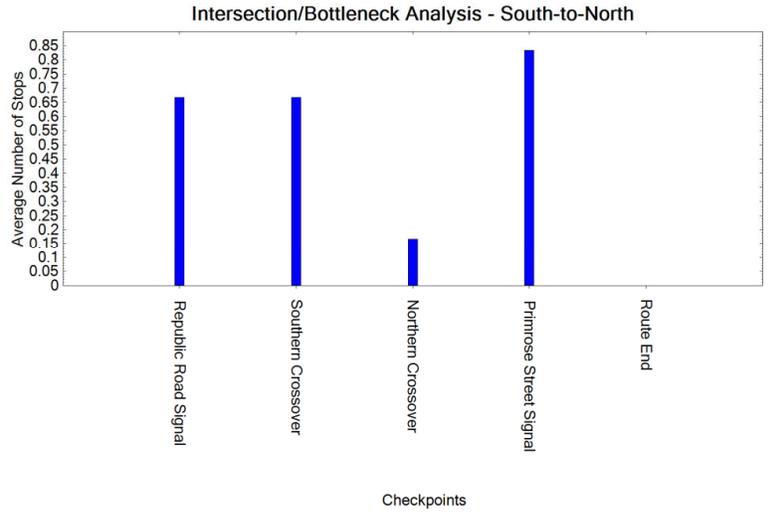
DCD Site	Scenario	Crossover	Direction	Lane	Sat. Flow (vph)	Number of Cycles Observed	Number of Vehicles Observed
National Avenue	After	South	NB	Left	1813	36	208
				Center	1833	19	83
				Right	1874	5	26
		North	SB	Left	1817	25	140
				Center	1762	22	111
				Right	1625	8	35

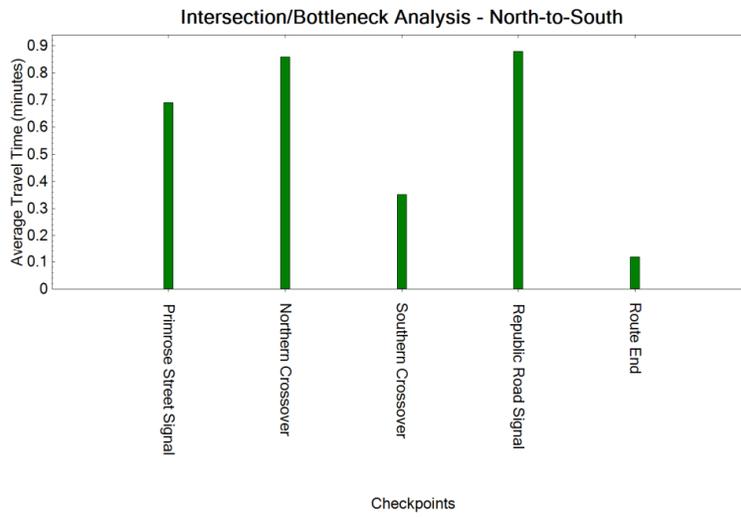
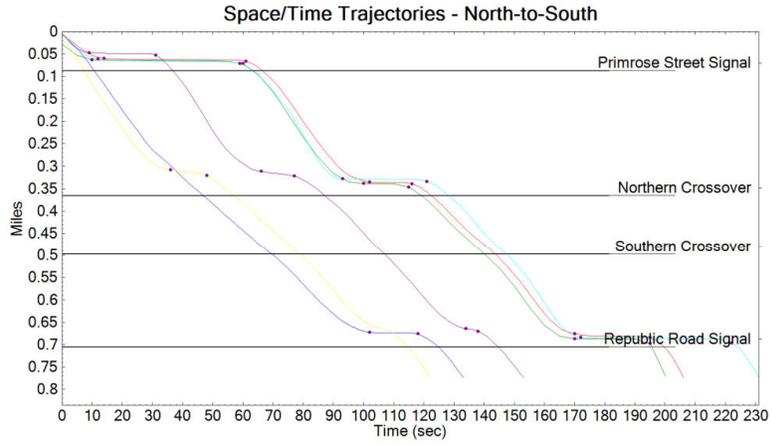
APPENDIX F:

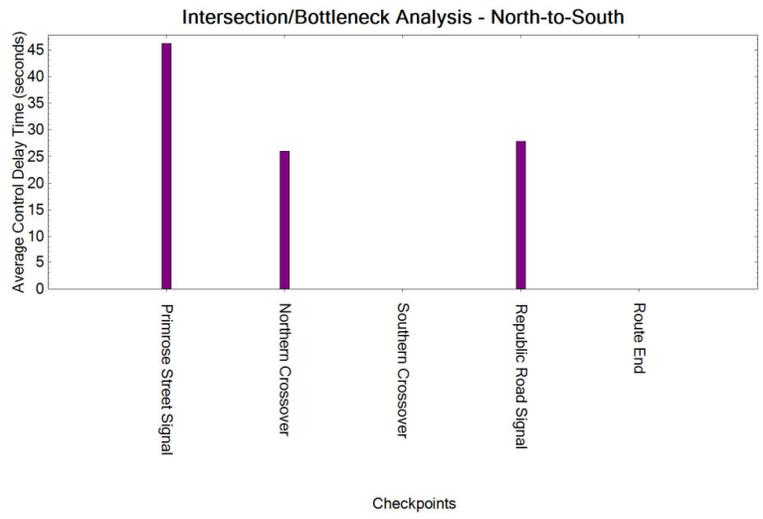
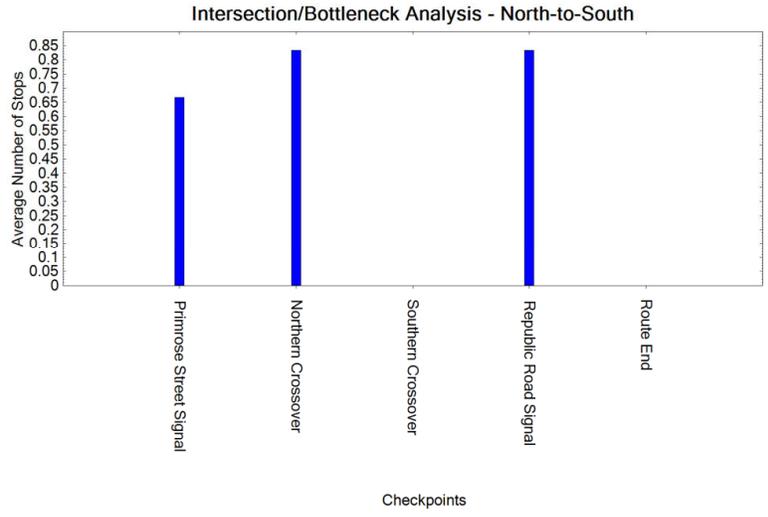
Travel Time Data from National Avenue, Springfield, MO

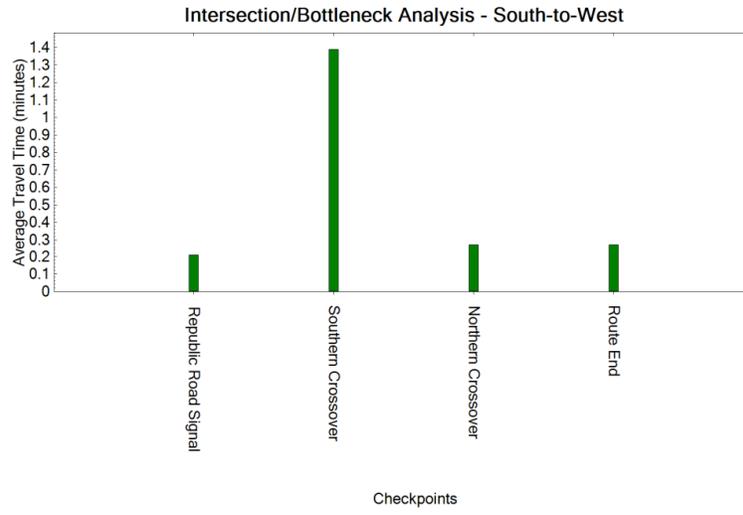
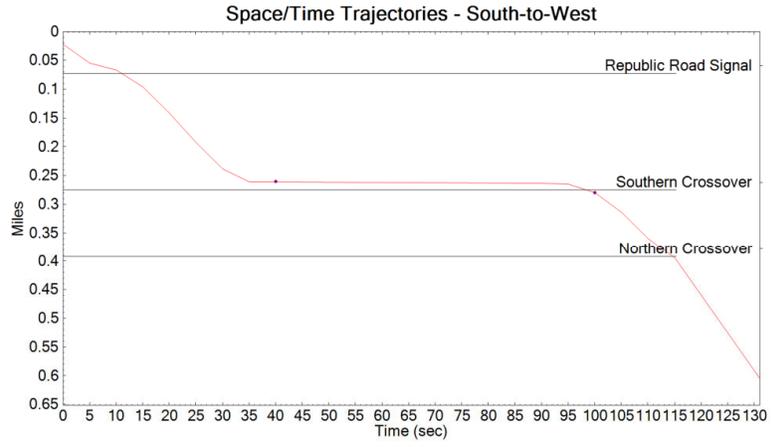
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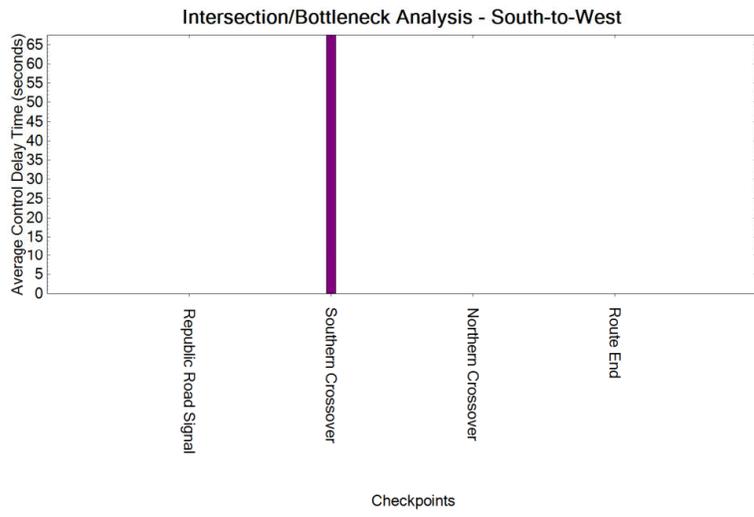
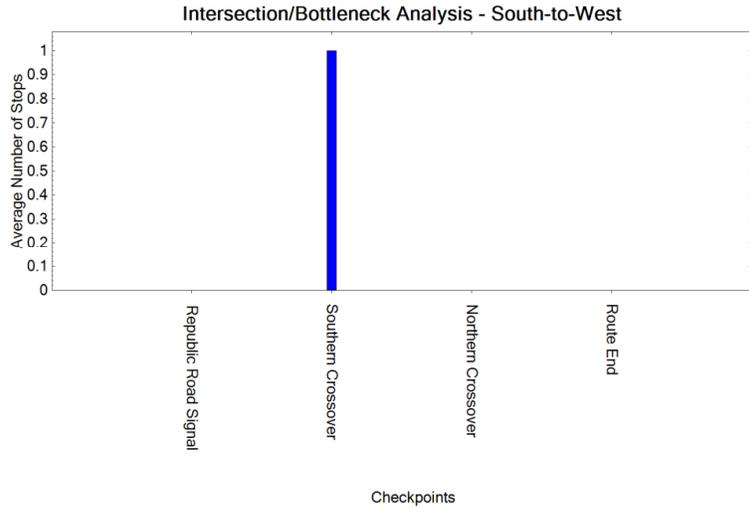


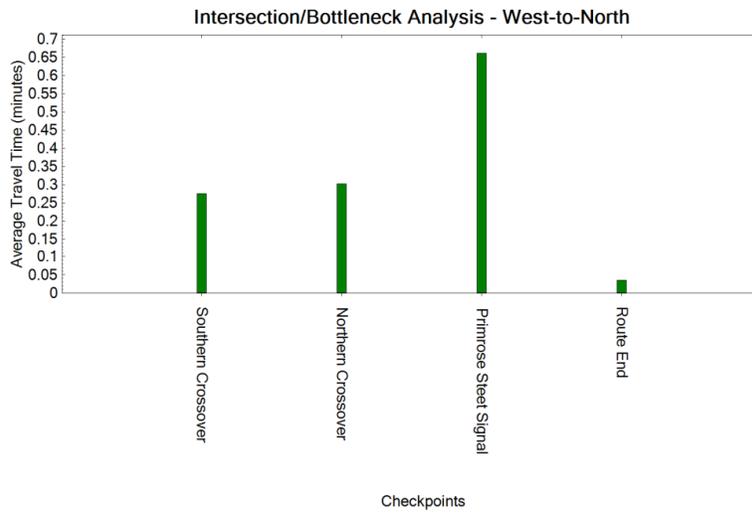
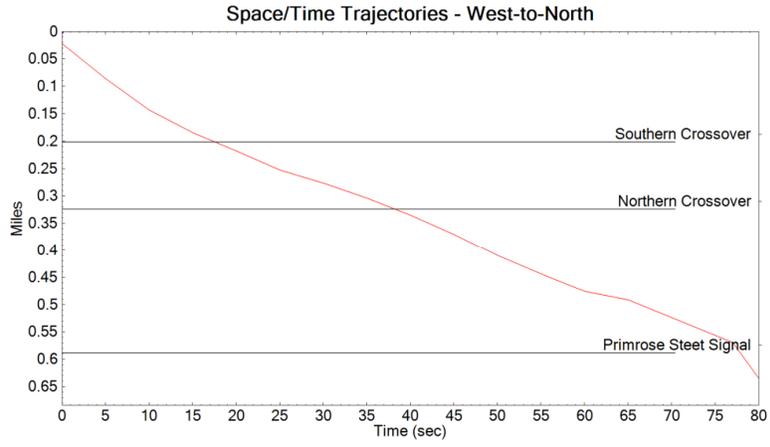


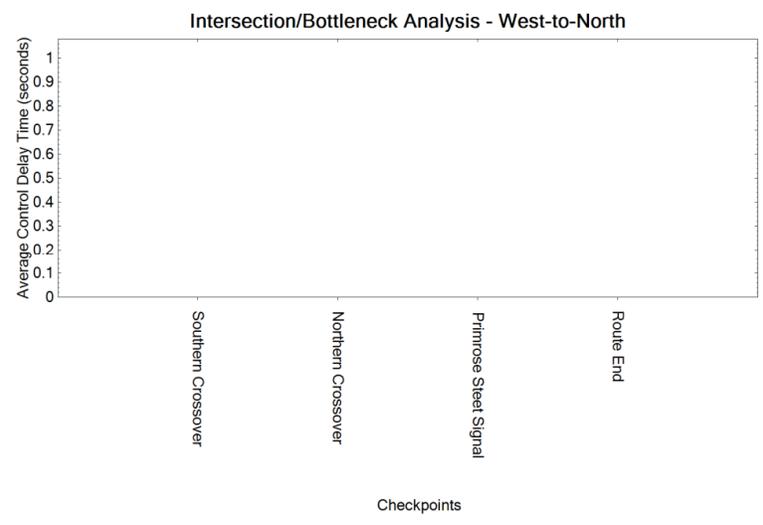
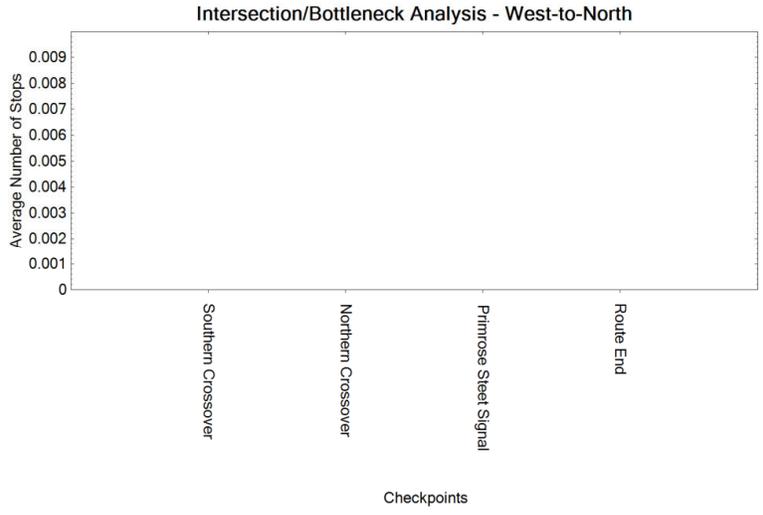


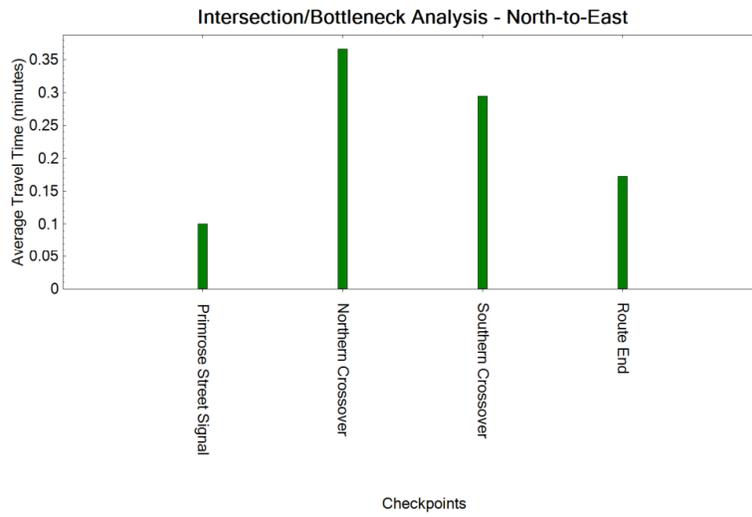
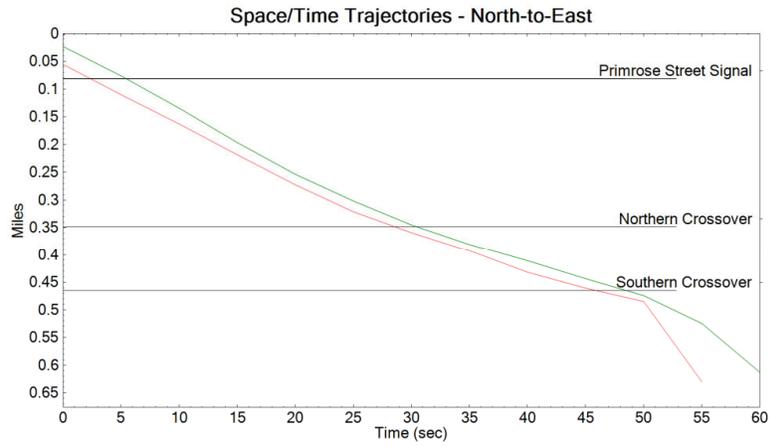


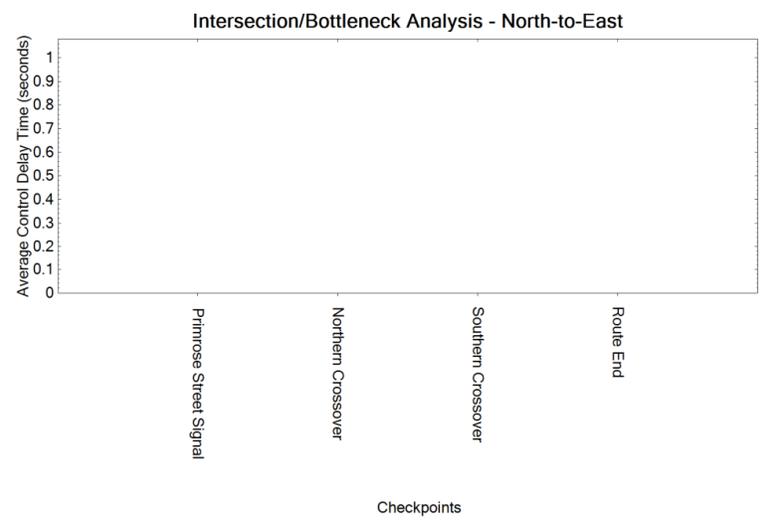
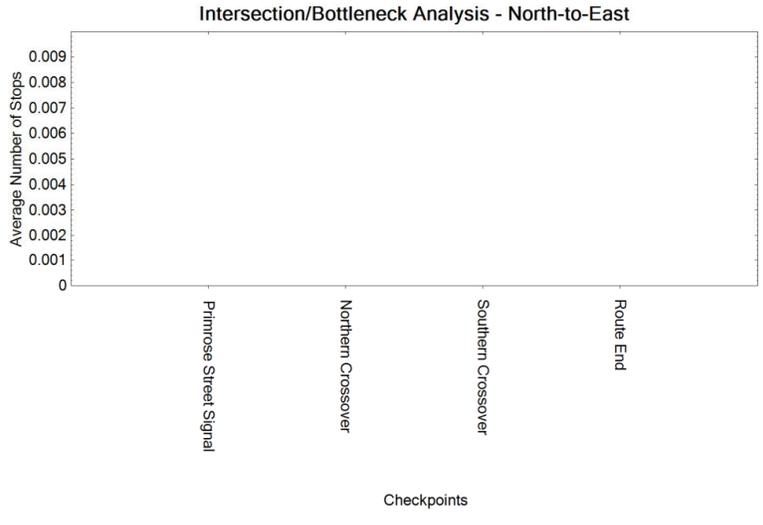


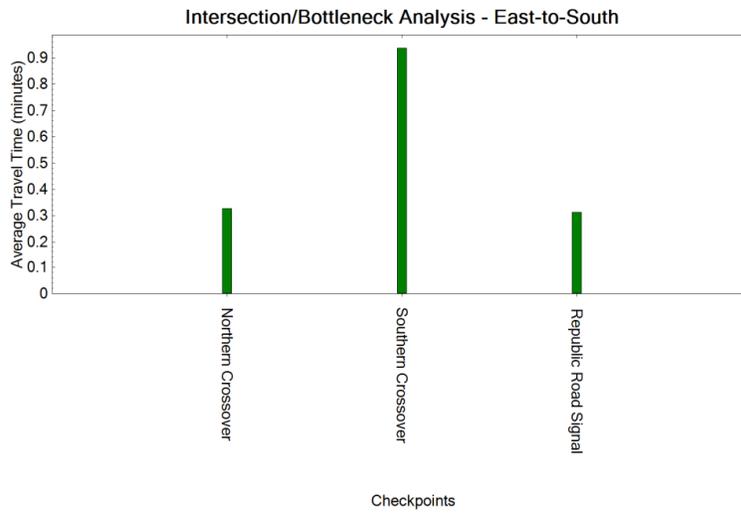
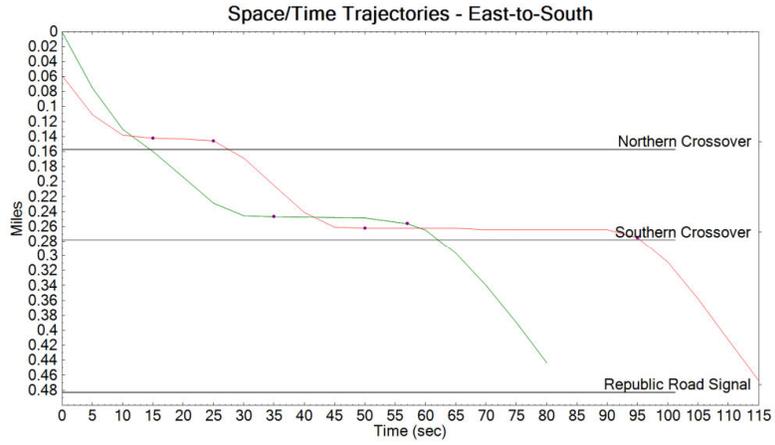


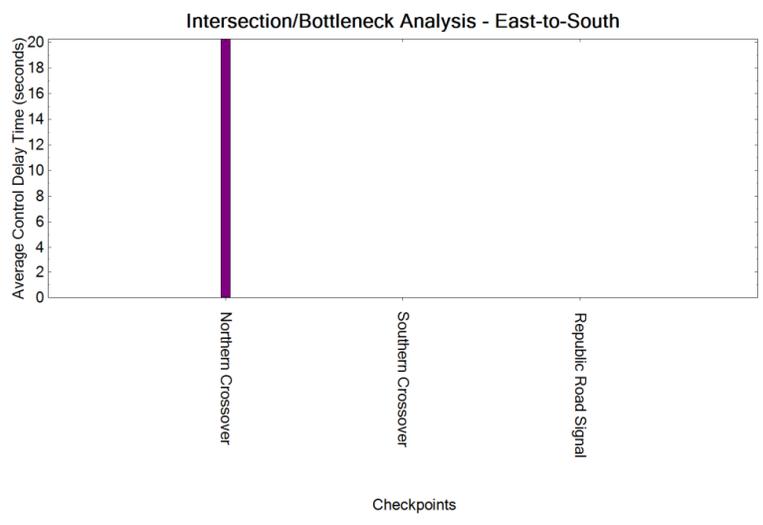
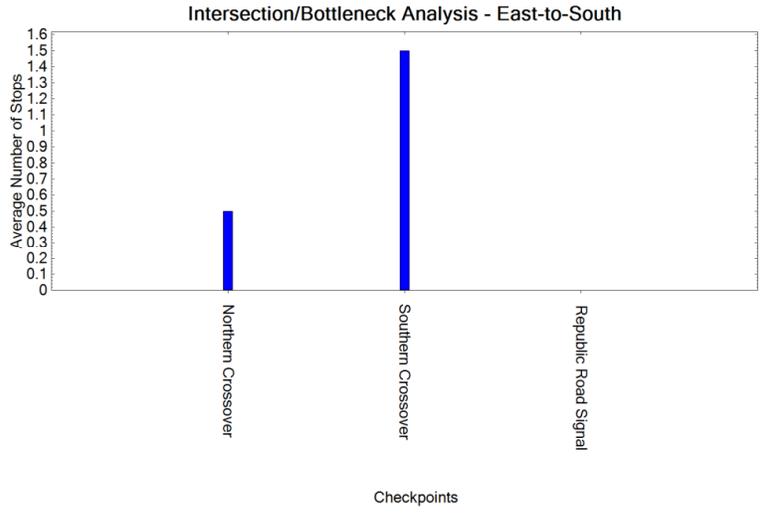




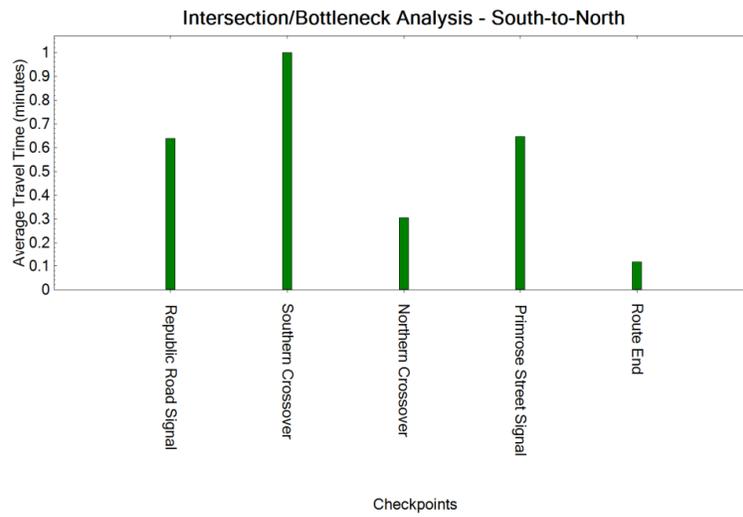
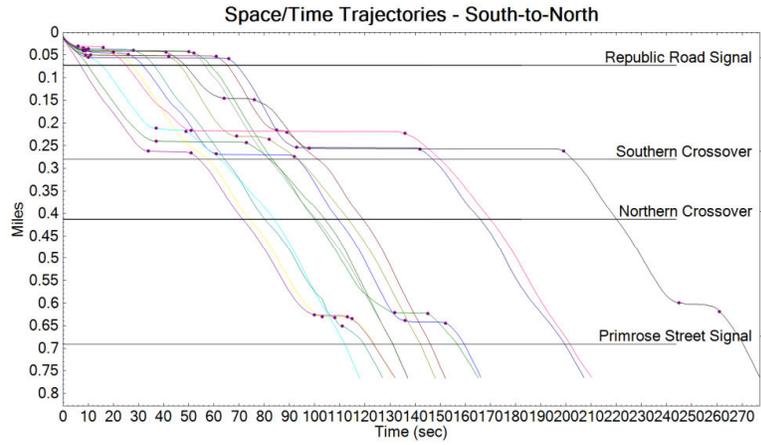


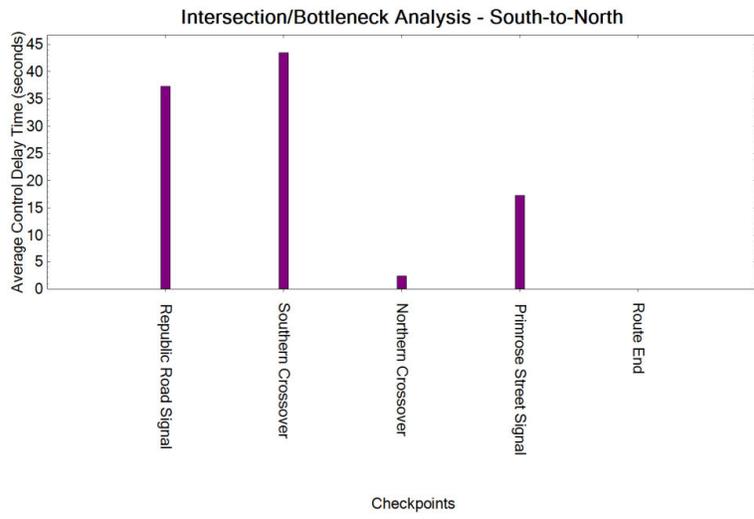
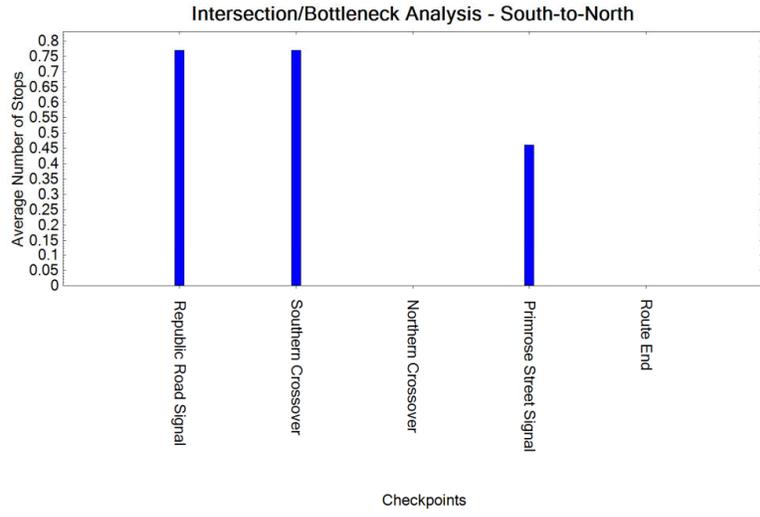


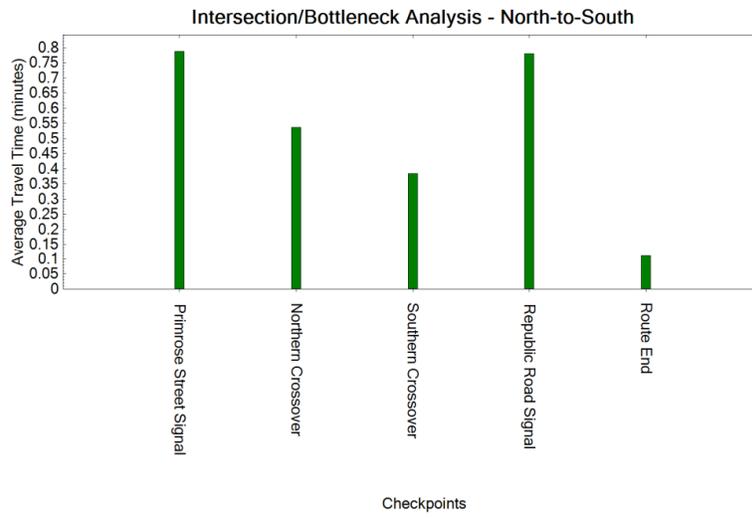
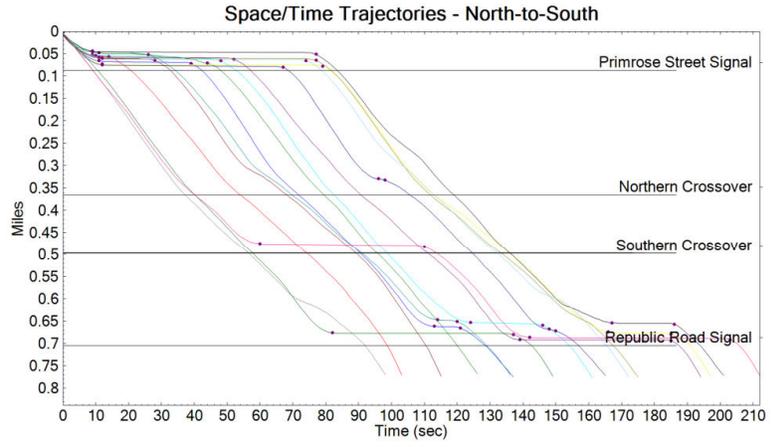


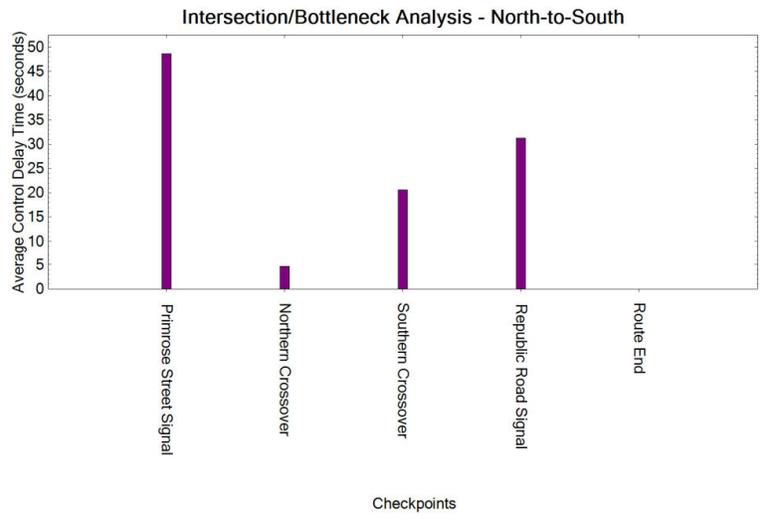
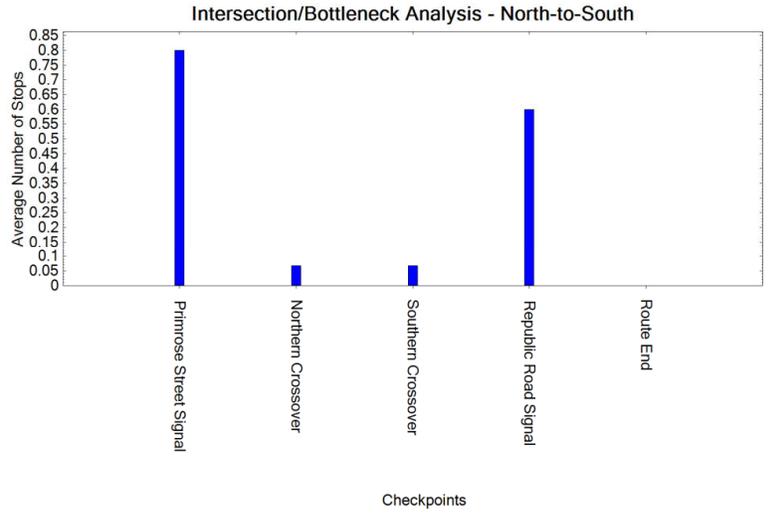


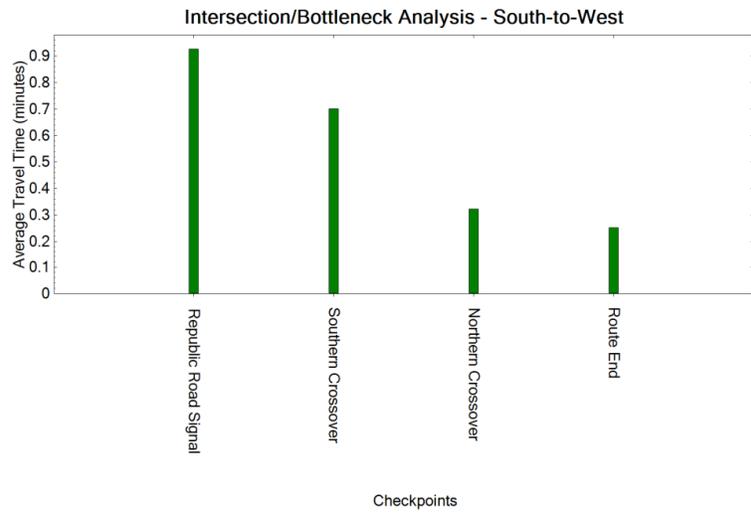
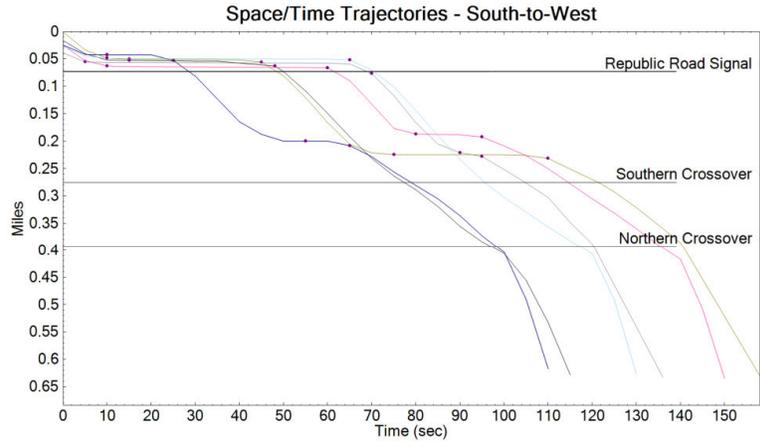
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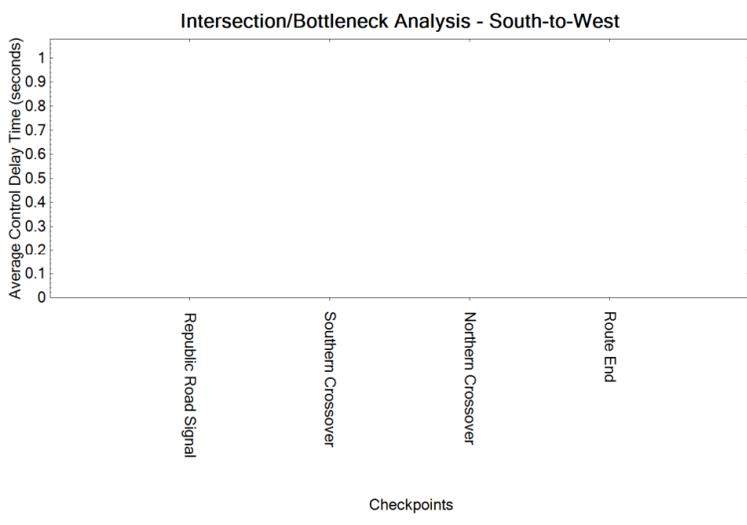
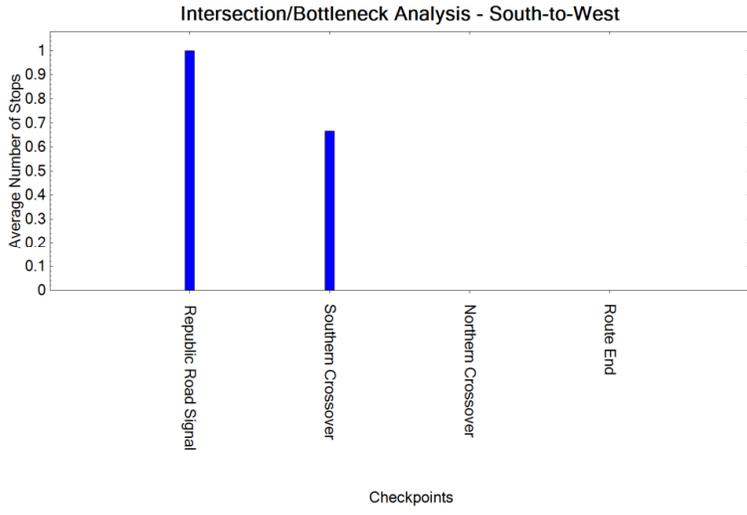


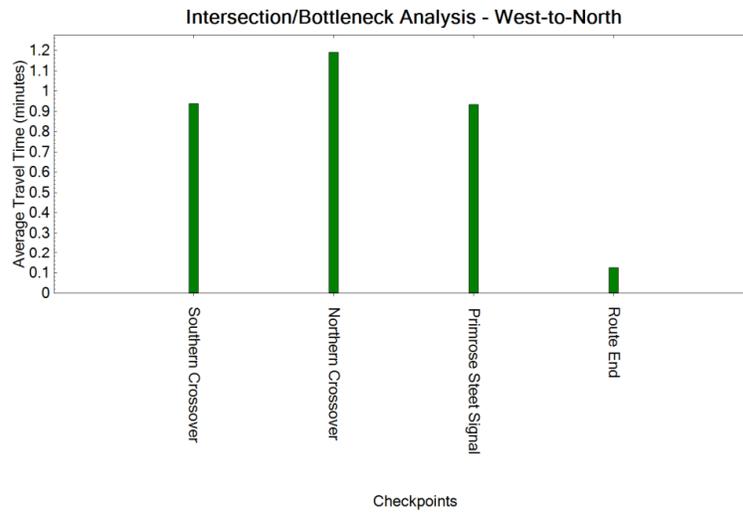
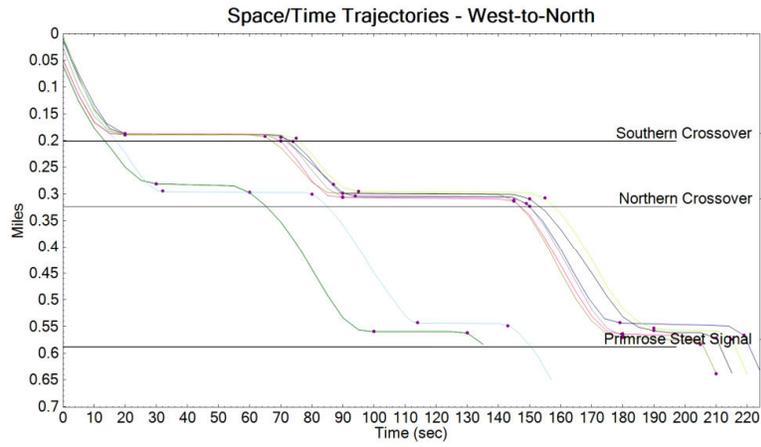


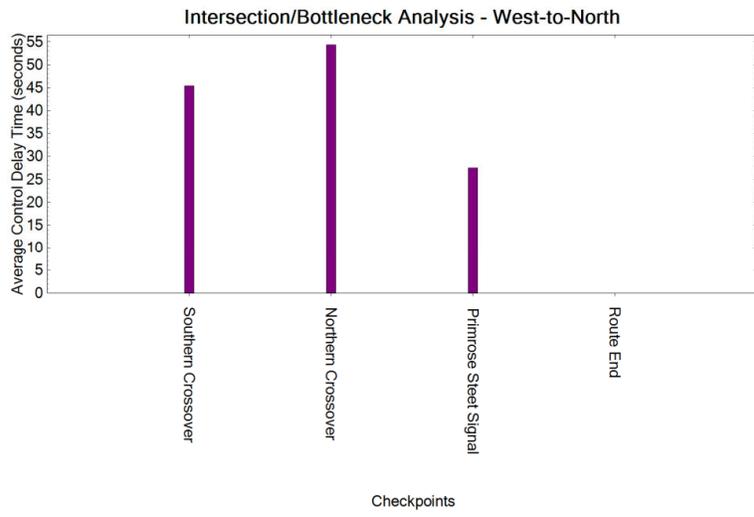
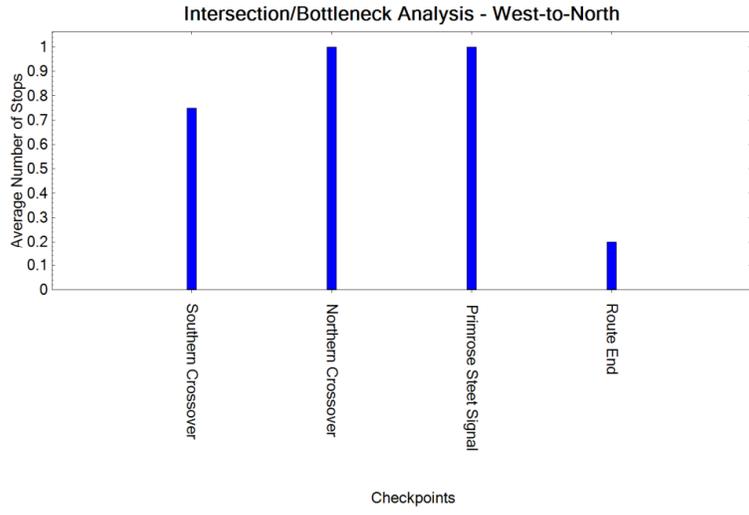


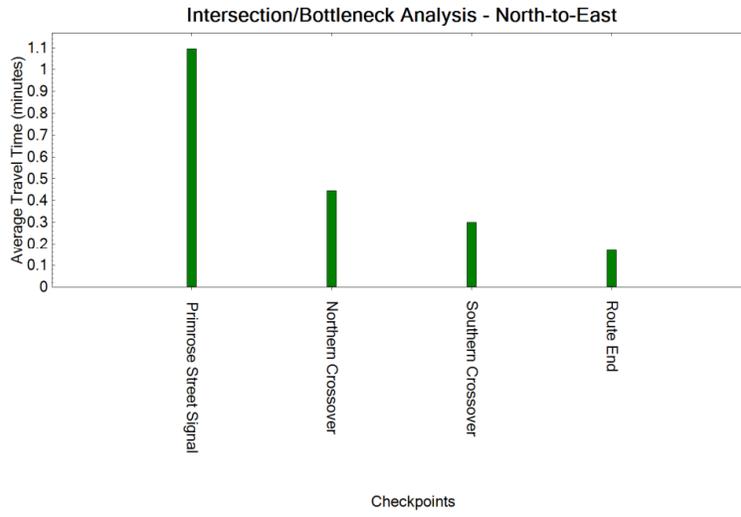
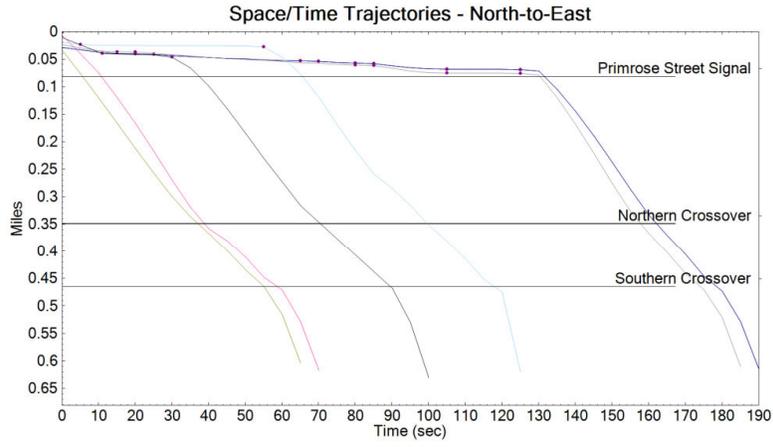


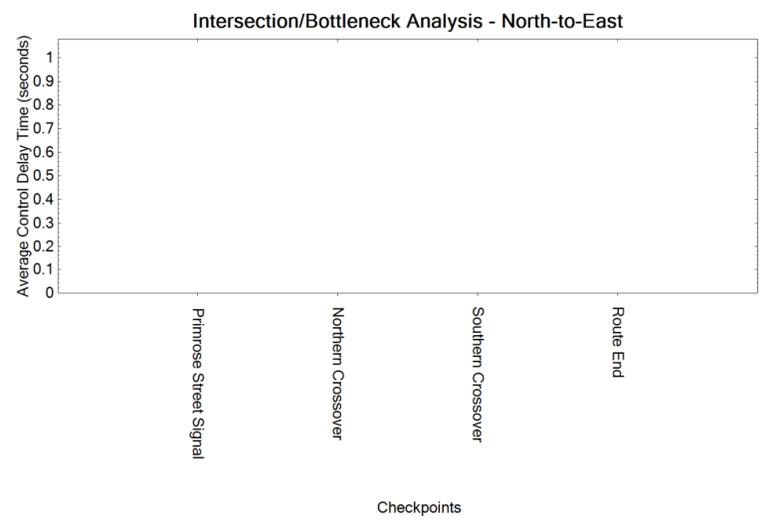
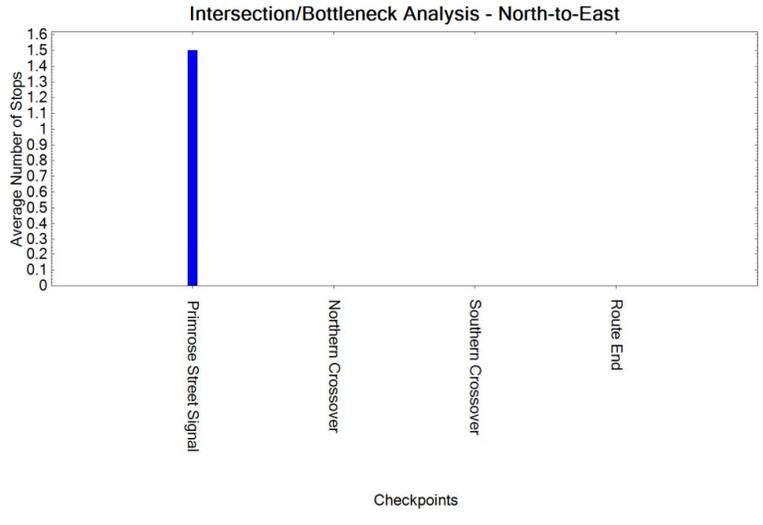


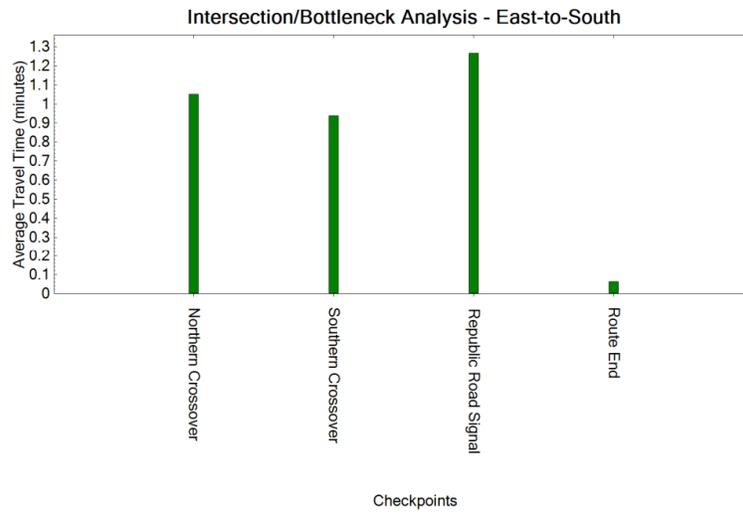
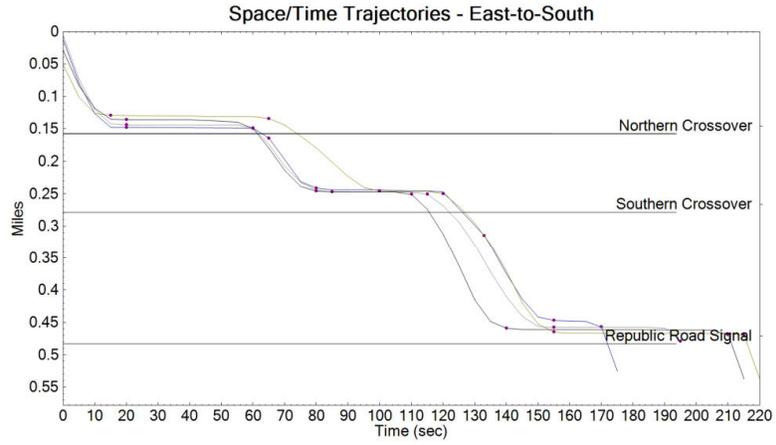


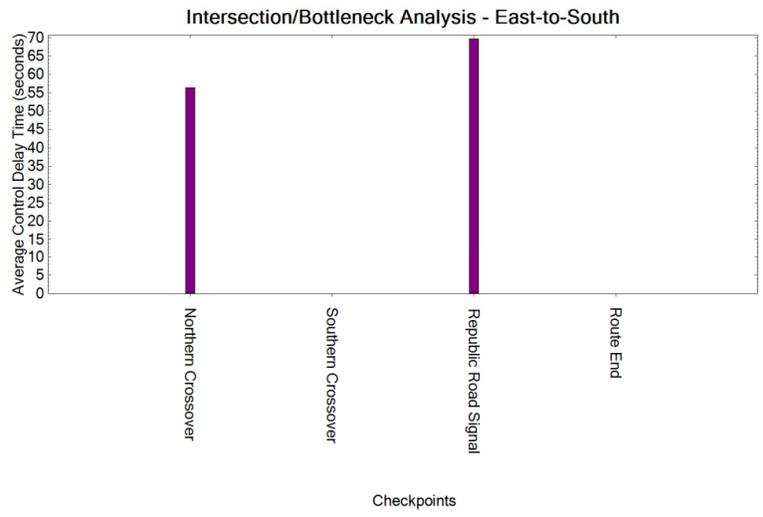
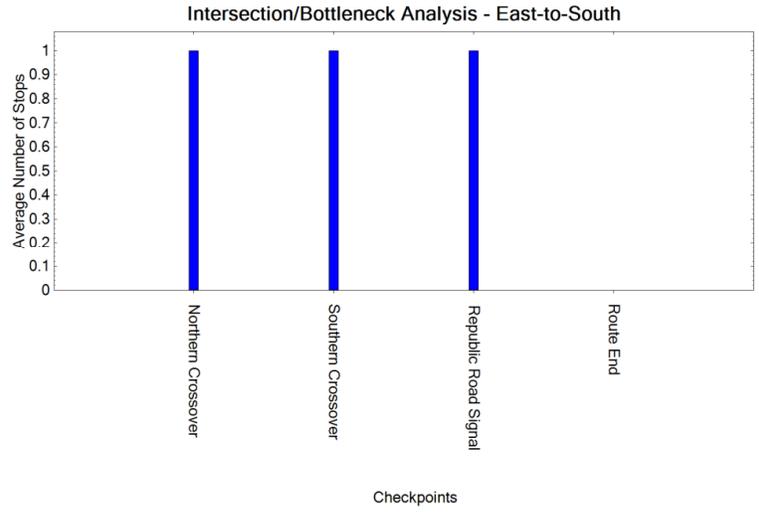












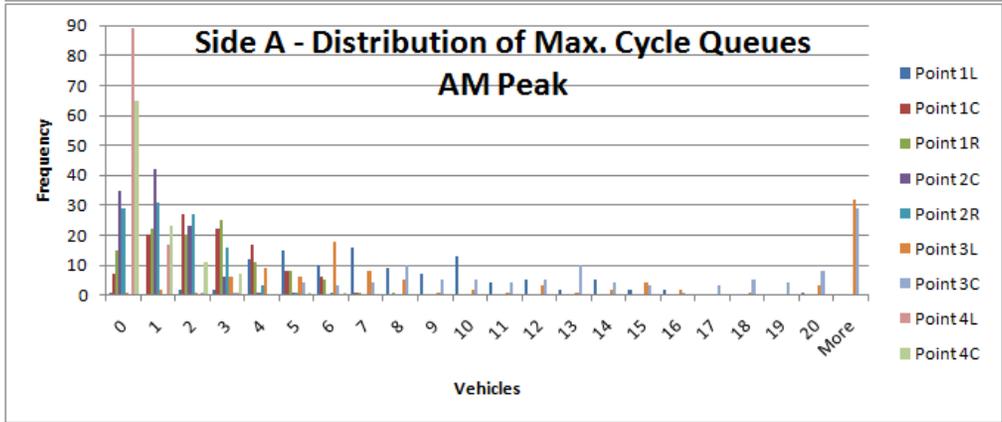
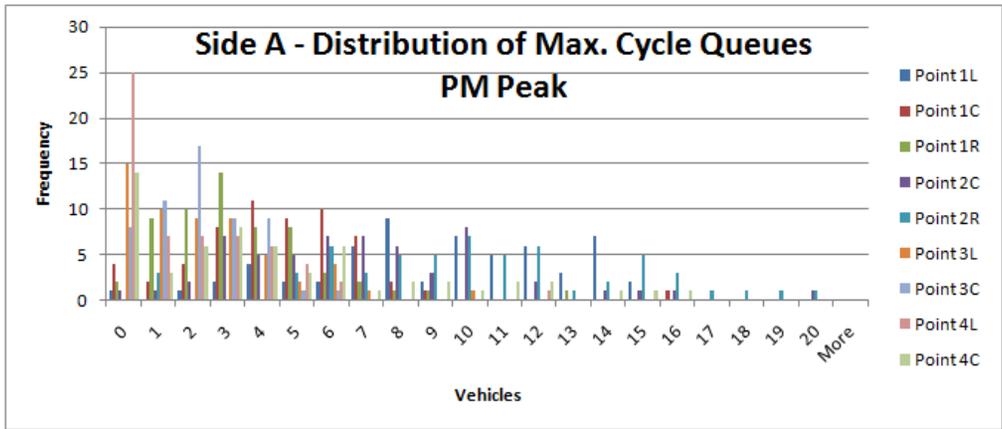
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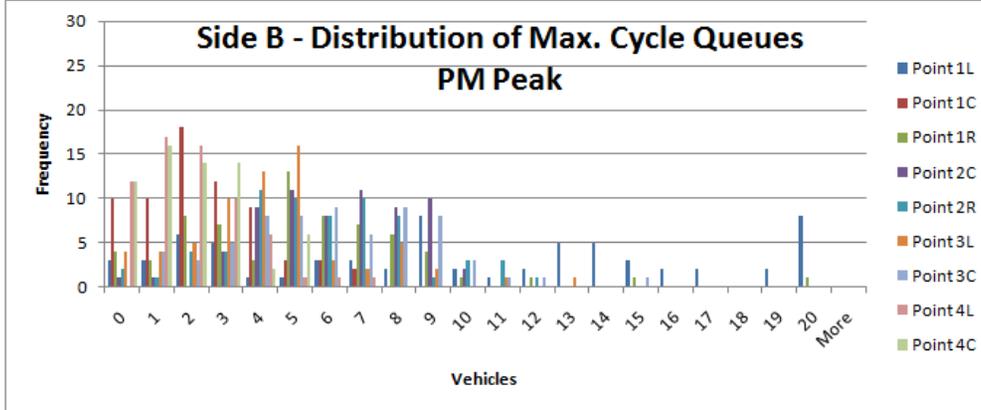
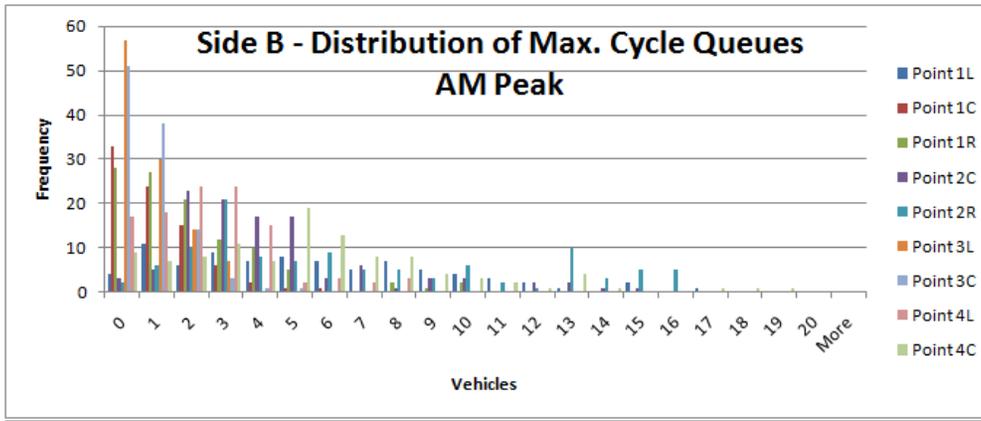
Field Data from Dorsett Road DCD, Maryland Heights, MO

Peak Period Queues – After

West Crossover (Side A) AM Peak	Point 1			Point 2		Point 3		Point 4	
	Left Lane	Center Lane	Right Lane	Center Lane	Right Lane	Left Lane	Center Lane	Left Lane	Center Lane
Mean	7.9	2.7	2.5	1.1	1.4	12.7	15.7	0.2	0.7
Min	0	0	0	0	0	0	3	0	0
Max	20	7	8	5	6	25	25	3	6
Std. Dev.	3.5	1.6	1.8	1.0	1.2	8.8	6.9	0.5	1.1
n	108	108	108	108	108	108	108	108	108
West Crossover (Side A) PM Peak	Point 1			Point 2		Point 3		Point 4	
	Left Lane	Center Lane	Right Lane	Center Lane	Right Lane	Left Lane	Center Lane	Left Lane	Center Lane
Mean	9.2	4.6	3.5	7.0	10.3	2.3	2.1	1.9	4.3
Min	0	0	0	0	1	0	0	0	0
Max	15	16	13	20	20	10	6	12	16
Std. Dev.	3.6	2.6	2.3	3.8	4.3	2.2	1.4	2.3	4.2
n	59	59	59	58	58	56	56	59	59

East Crossover (Side B) PM Peak	Point 1			Point 2		Point 3		Point 4	
	Left Lane	Center Lane	Right Lane	Center Lane	Right Lane	Left Lane	Center Lane	Left Lane	Center Lane
Mean	5.5	1.1	2.0	4.4	6.8	0.7	0.8	2.4	5.5
Min	0	0	0	0	0	0	0	0	0
Max	17	6	10	15	16	3	5	8	19
Std. Dev.	3.9	1.3	2.1	3.0	4.7	0.9	1.0	1.9	3.9
n	82	82	108	108	108	108	108	108	108
East Crossover (Side B) PM Peak	Point 1			Point 2		Point 3		Point 4	
	Left Lane	Center Lane	Right Lane	Center Lane	Right Lane	Left Lane	Center Lane	Left Lane	Center Lane
Mean	9.9	2.5	5.3	6.2	5.8	4.4	6.1	1.9	1.9
Min	0	0	0	0	0	0	1	0	0
Max	20	7	20	10	12	13	15	7	5
Std. Dev.	6.4	1.8	3.5	2.2	2.7	2.6	2.9	1.5	1.5
n	67	67	67	66	66	66	66	64	64





Saturation Flow Rate Calculation – After

Saturation Flow Rate Calculation for West Crossover EB Through Movement

Obs No.	Time (seconds) between 4th vehicle and...			
	7th veh.	8th veh.	9th veh.	10th veh.
1				12.47
2			10.72	
3	5.81			
4	8.34			
5				9.78
6			9.28	
7				10.47
8				9.28
9	5.34			
10		6.25		
11	5.97			
12				11.43
13		7.66		
14			11.03	
15		6.31		
16			8.97	
17		7.6		
18	5.91			
19	6.65			
20	5.84			
21				9.97
22		7.96		
23				11.16
24	6.94			
25			9.34	
26		8.84		
27		10.32		
28	7.81			
29				10.28
30	7.33			
31		8.78		
32				11.43
33			9.43	
34		7		
35	6.56			
36		8.22		
37			7.37	
38		7.09		
39				11.8
40	6.23			
41				10.4
42		8.81		
43				9.69
44	7.38			
45	6.43			
46				10.28
Sum	92.54	94.84	66.14	138.44
Total Observations = 46		Mean Sat Flow (vph) = 1823		

Saturation Flow Rate Calculation for West Crossover WB Through Movement

Obs No.	Time (seconds) between 4th vehicle and...			
	7th veh.	8th veh.	9th veh.	10th veh.
1				11.25
2	6.69			
3		6.34		
4		7.91		
5	6.69			
6				13.1
7		7.75		
8	5.69			
9	5.25			
10		8.72		
11	7.87			
12				11.19
13	6.72			
14				11.19
15		7.53		
16	6			
17	6.28			
18				10.66
19		8.19		
20		7.15		
21		8.47		
22		8.9		
23	5.34			
24	5.75			
25		7.75		
26		6.25		
27				10.94
28				9.68
Sum	62.28	84.96	0	78.01
Total Observations = 28		Mean Sat Flow (vph) = 1833		

Saturation Flow Rate Calculation for West Crossover SB Left Movement

Obs No.	Time (seconds) between 4th vehicle and...			
	7th veh.	8th veh.	9th veh.	10th veh.
1			9.25	
2	6			
3	5.23			
4	6.91			
5	5.42			
6	6.73			
7			9.1	
8	5.95			
9				12.97
10	6.76			
11		7.75		
12	5.85			
13	6.04			
14	6.54			
15	5.79			
16		7.78		
17		6.62		
18				11.81
19	5.53			
20			9.09	
21		8		
22	6.59			
23	5.81			
24	6.38			
25			9.44	
26	6.65			
27	7.12			
Sum	105.3	30.15	36.88	24.78
Total Observations = 27		Mean Sat Flow (vph) = 1795		

Saturation Flow Rate Calculation for East Crossover WB Through Movement

Obs No.	Time (seconds) between 4th vehicle and...			
	7th veh.	8th veh.	9th veh.	10th veh.
1				11.97
2	4.15			
3				8.9
4				9.16
5				11.12
6				10.97
7				10.59
8				10.97
9				14.96
10		7.41		
11		6.5		
12	5.07			
13				11.37
14	4.81			
15				12.1
16				11.47
17				11.79
18		8.97		
19				11.69
20				14.34
21				11.09
22				14.81
23				10.84
24				12.22
25				13.03
26				11.84
27		10.22		
28				10.63
29				10.72
30				14.47
31				12.91
Sum	14.03	33.1	0	283.96
Total Observations = 31		Mean Sat Flow (vph) = 1851		

Saturation Flow Rate Calculation for East Crossover EB Through Movement

Obs No.	Time (seconds) between 4th vehicle and...			
	7th veh.	8th veh.	9th veh.	10th veh.
1			9.97	
2	7.78			
3	10			
4	6.93			
5			9.97	
6				10.31
7				12.97
8	7.47			
9	6.44			
10		7.37		
11				10.09
12		8.38		
13		13		
14	5.25			
15	7.09			
16	6.03			
17			9.75	
18	5.37			
19				12.03
20	5.54			
21		7.09		
Sum	67.9	35.84	29.69	45.4
Total Observations = 21		Mean Sat Flow (vph) = 1676		

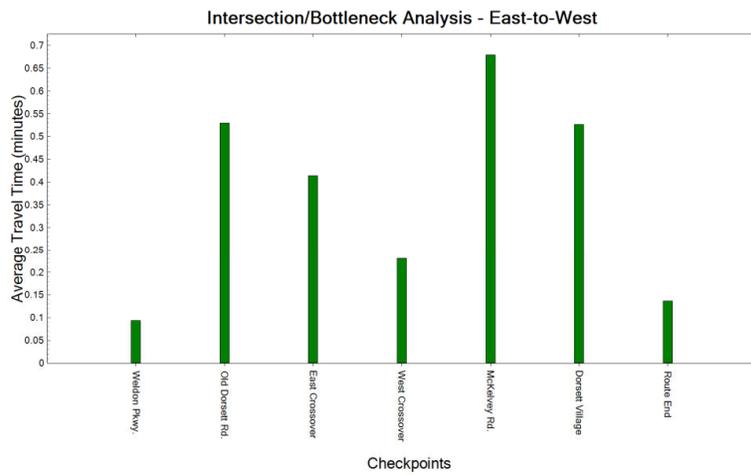
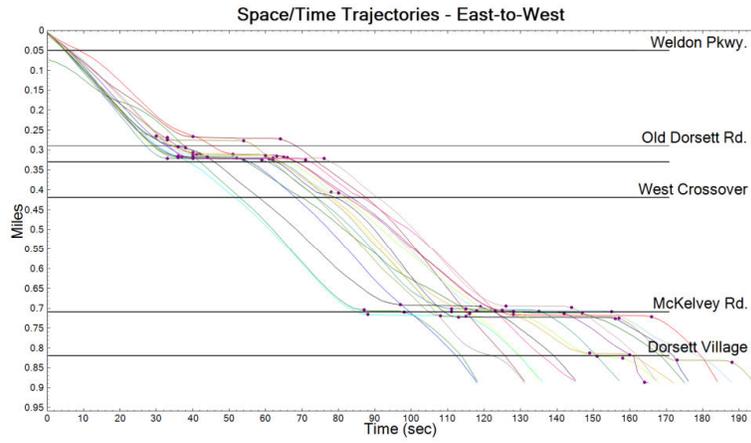
Saturation Flow Rate Calculation for East Crossover NB Left Movement

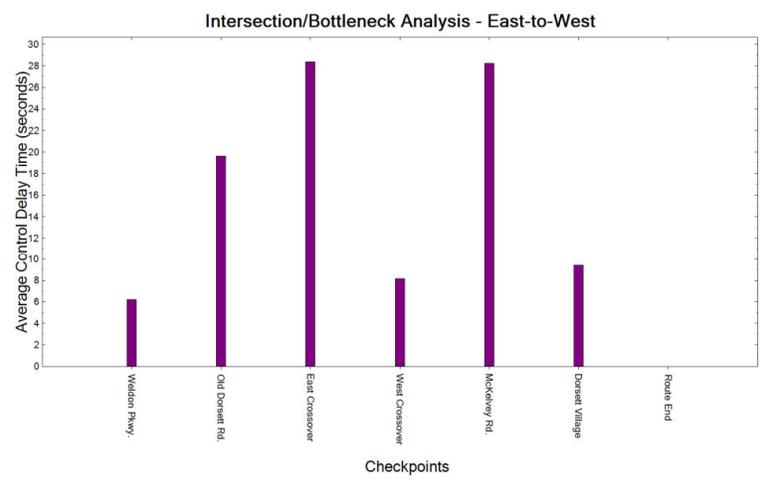
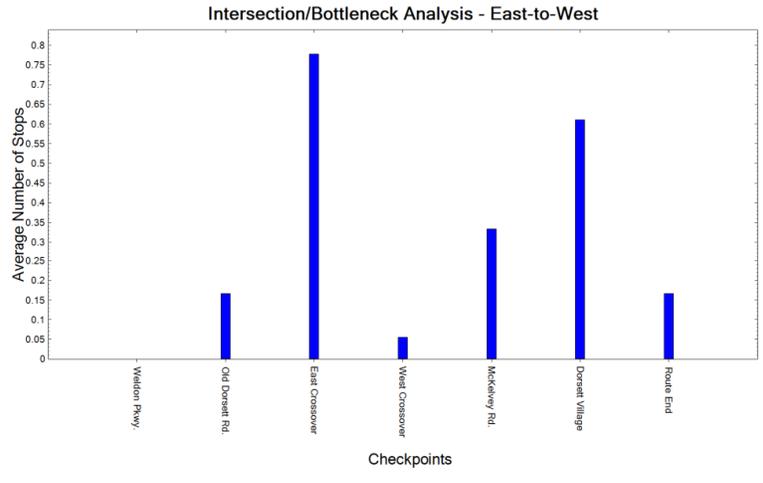
Obs No.	Time (seconds) between 4th vehicle and...			
	7th veh.	8th veh.	9th veh.	10th veh.
1			10.04	
2			9.22	
3				12.09
4	6.62			
5	6.03			
6		7.91		
7				11.34
8	7			
9			8.5	
10	4.72			
11	5.09			
12	5.4			
13	7.59			
14		6.59		
15			8.69	
16		8.25		
17				12.85
18	5.22			
19		7.5		
20			7.28	
Sum	47.67	30.25	43.73	36.28
Total Observations = 20		Mean Sat Flow (vph) = 1883		

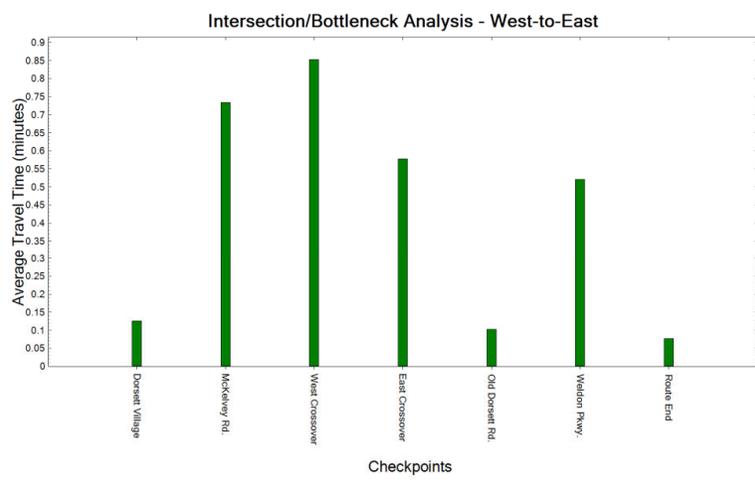
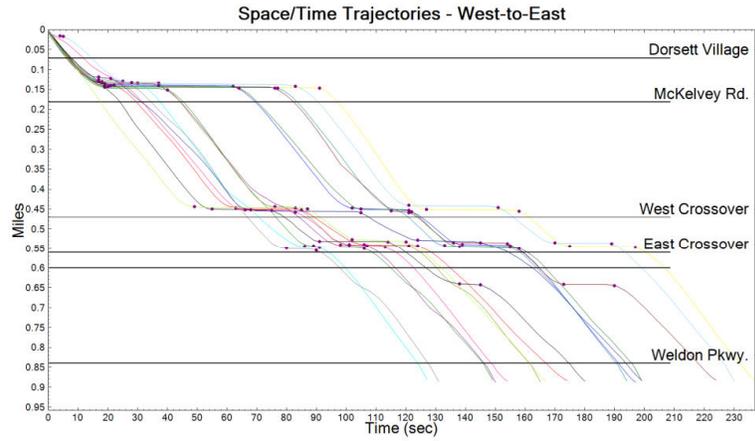
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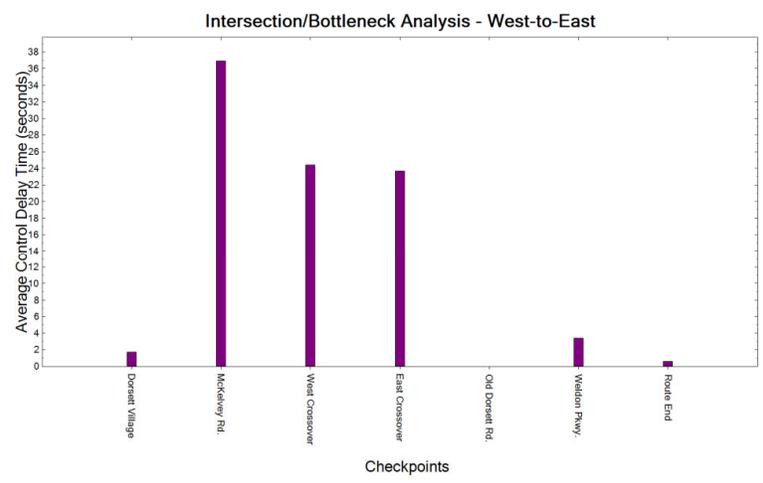
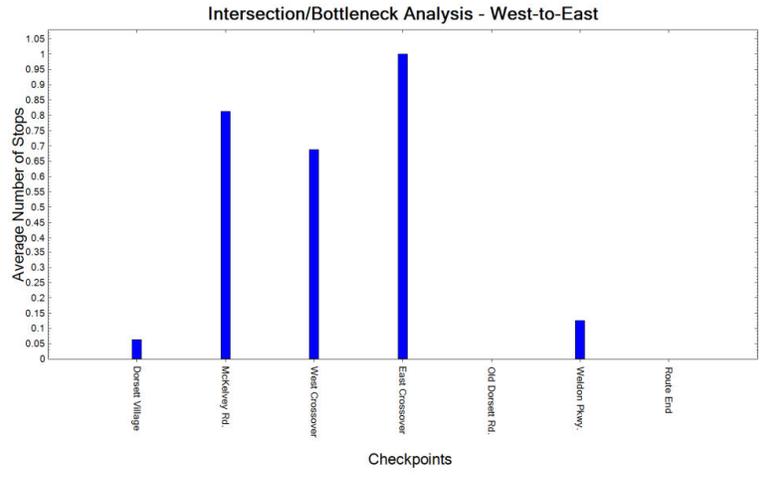
Travel Time Data from Dorsett Road DCD, Maryland Heights, MO

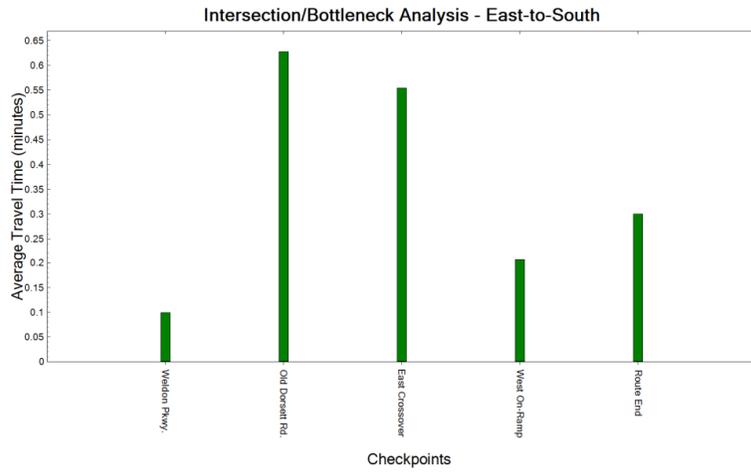
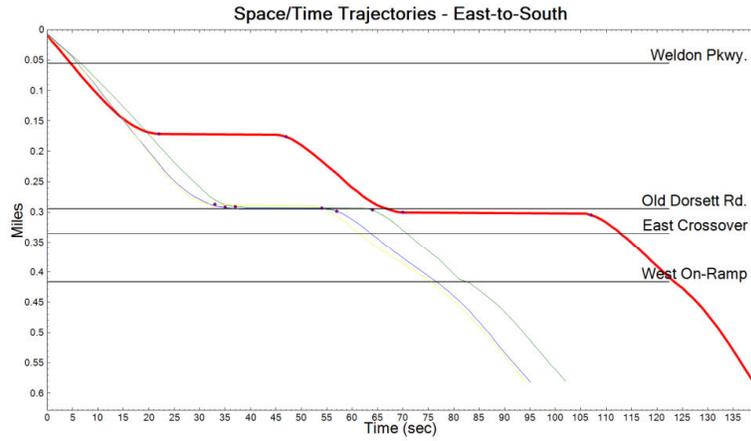
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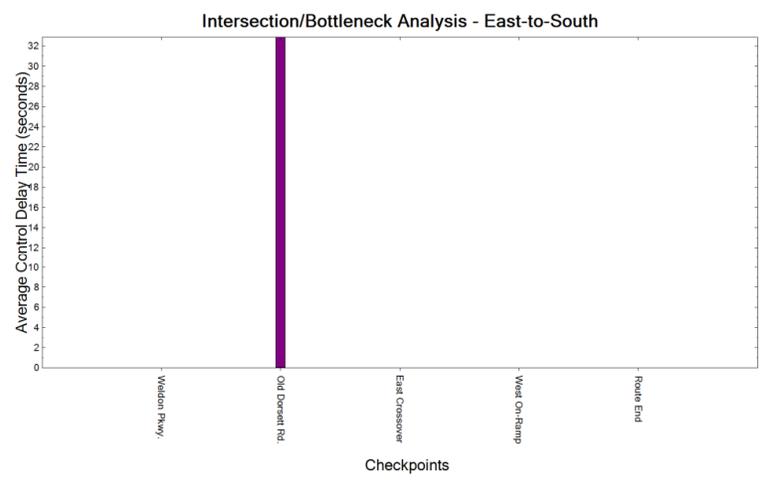
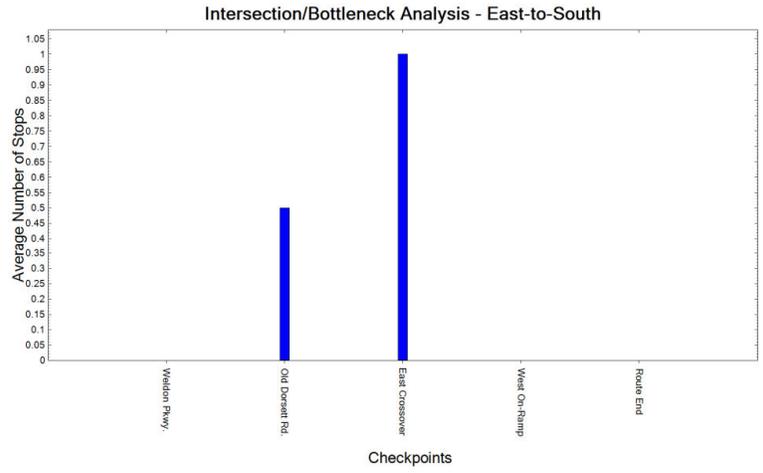


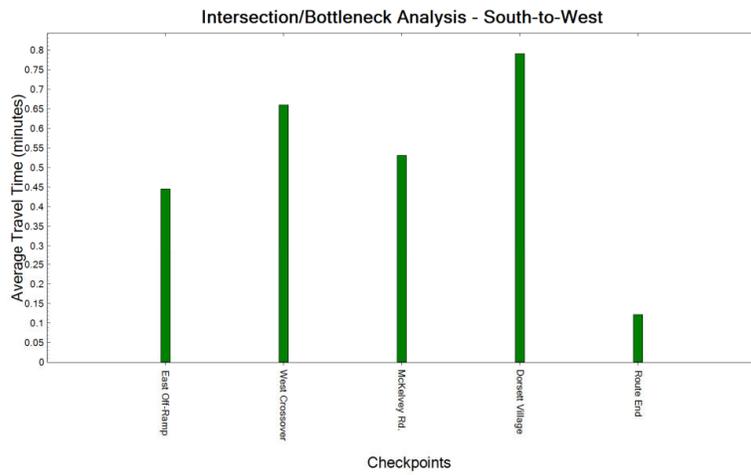
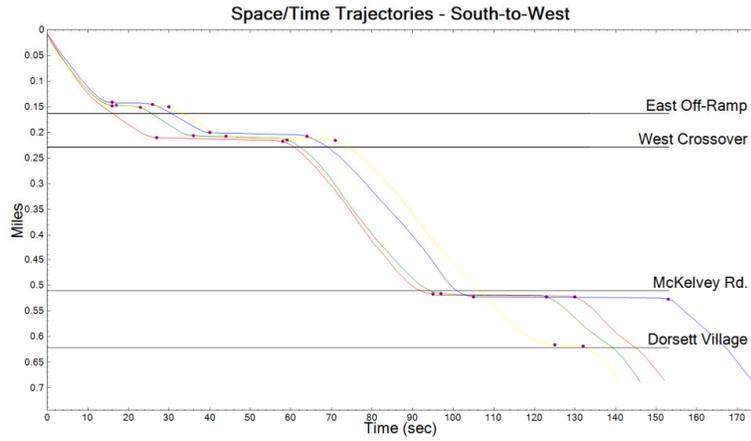


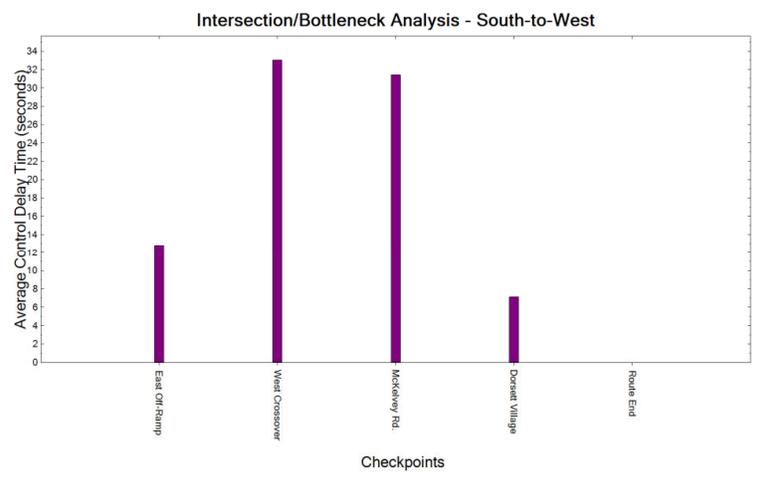
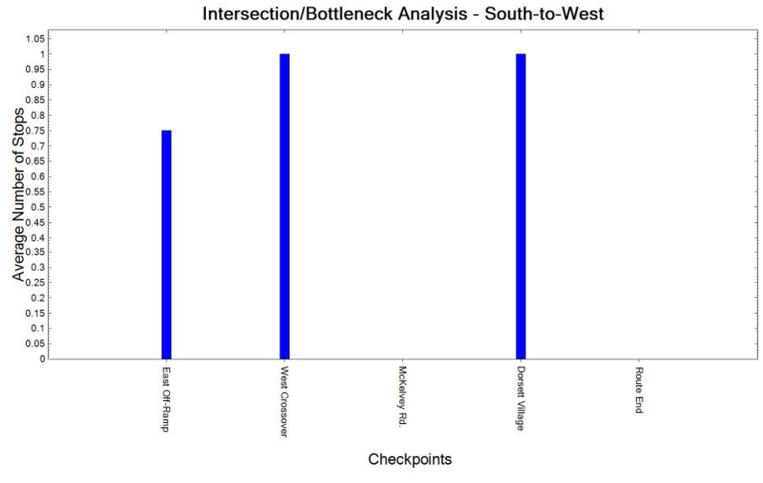


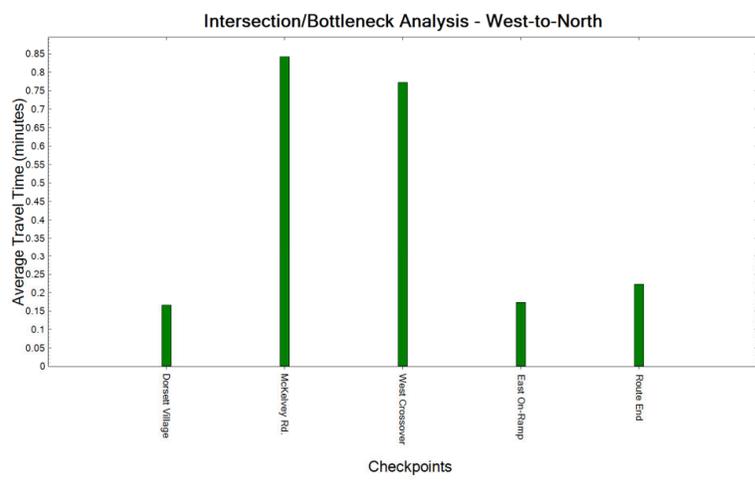
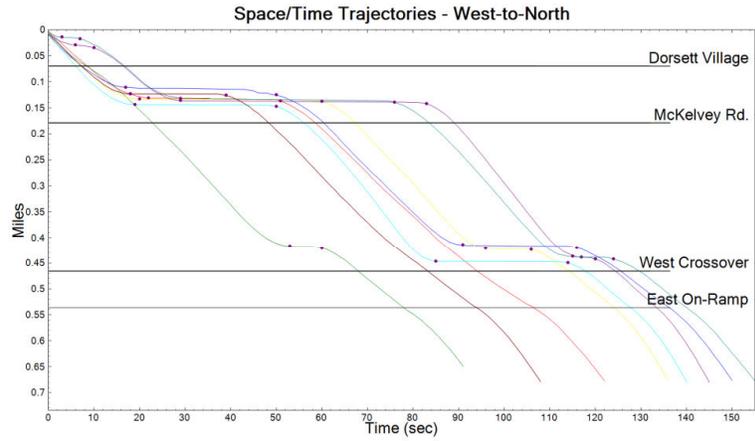


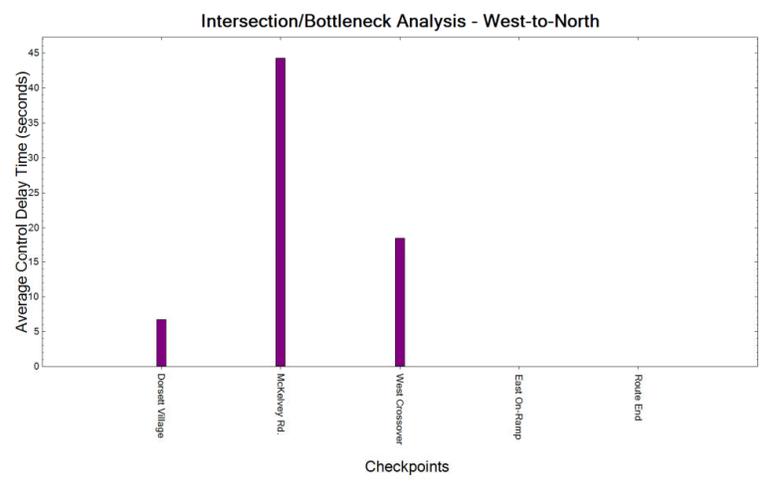
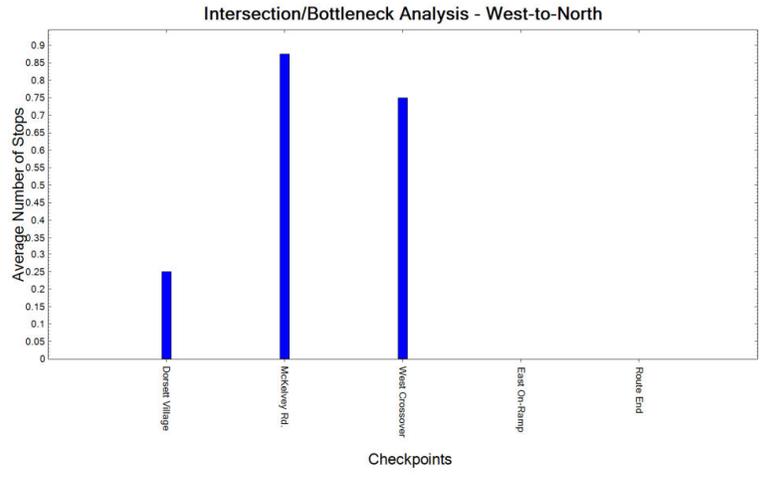


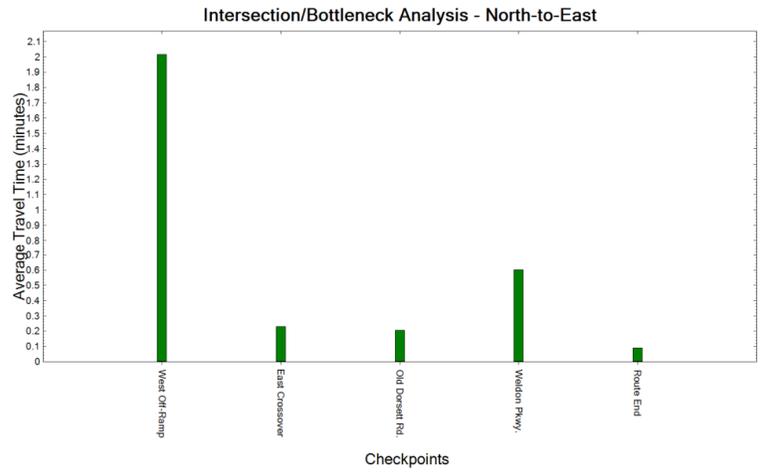
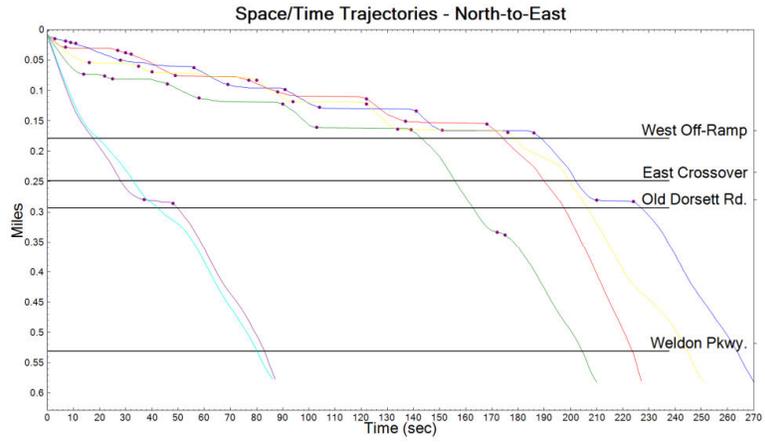


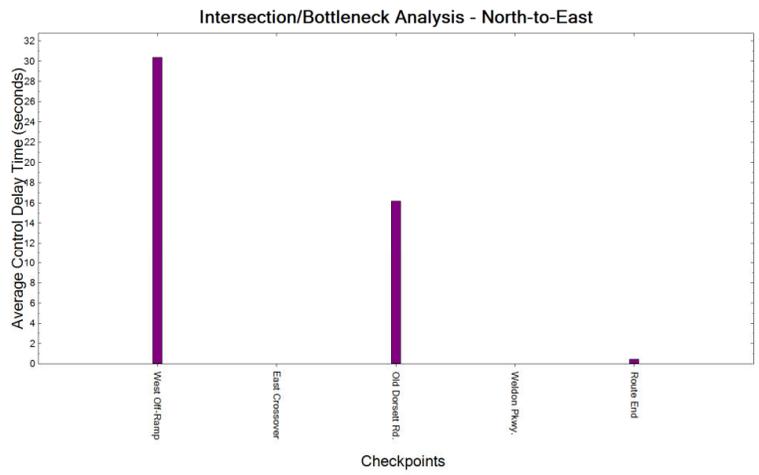
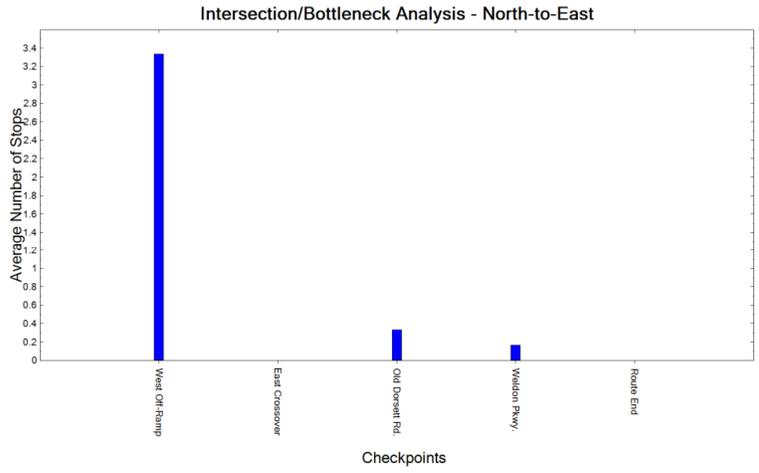




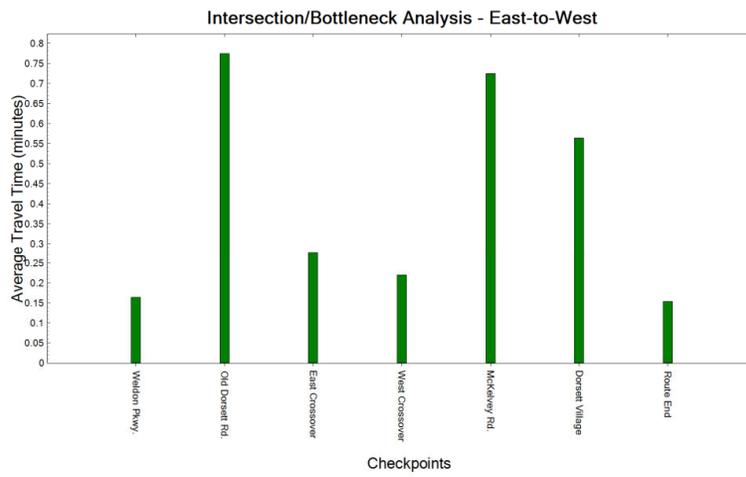
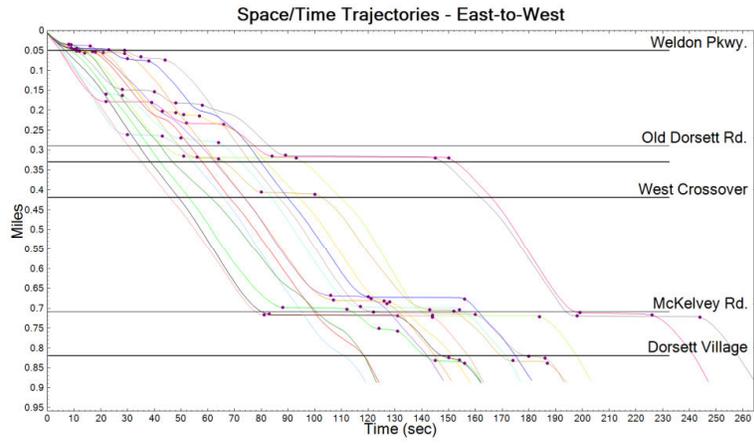


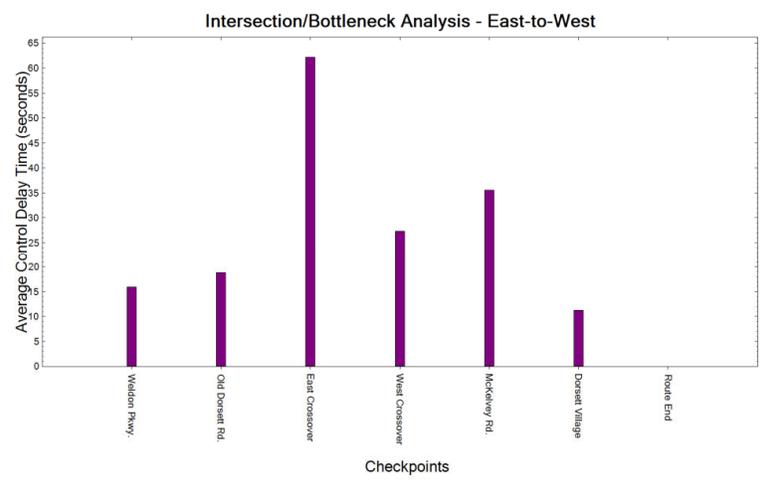
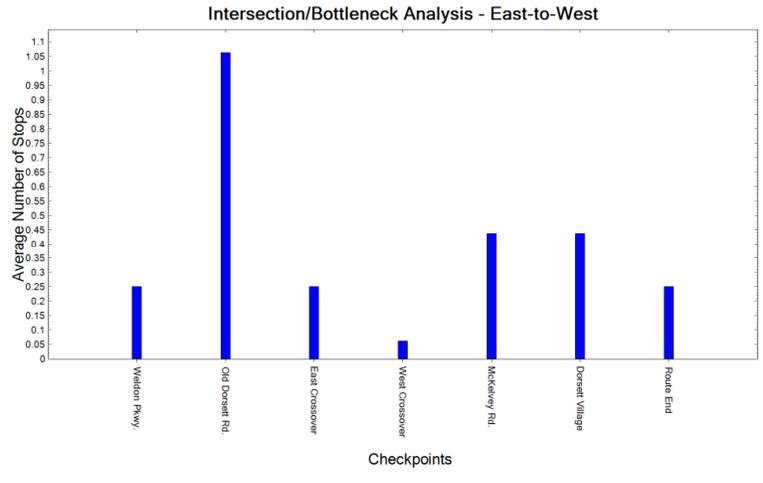


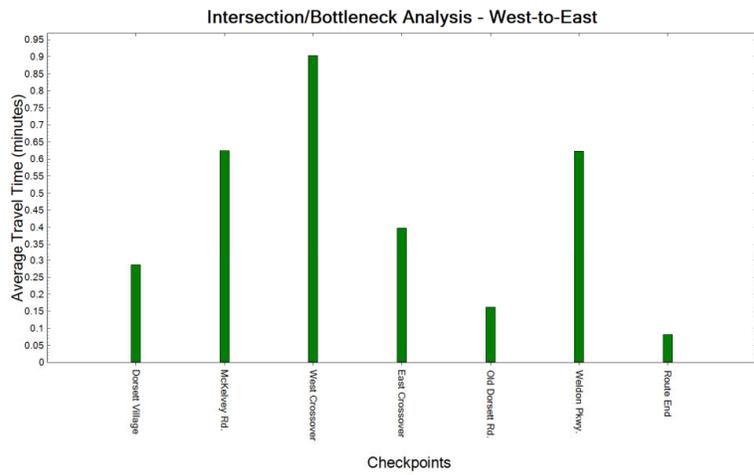
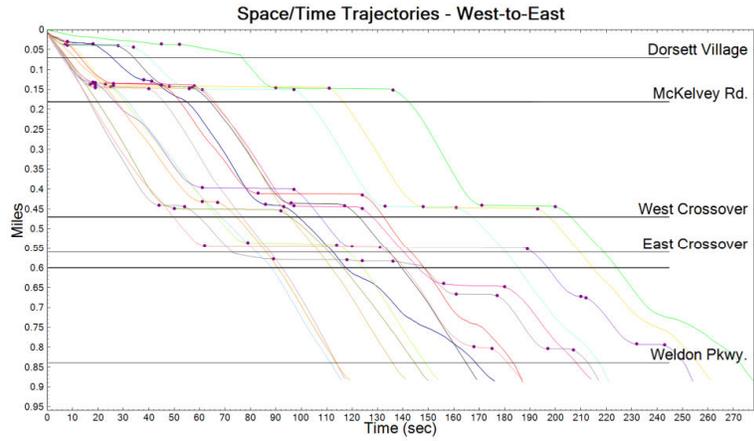


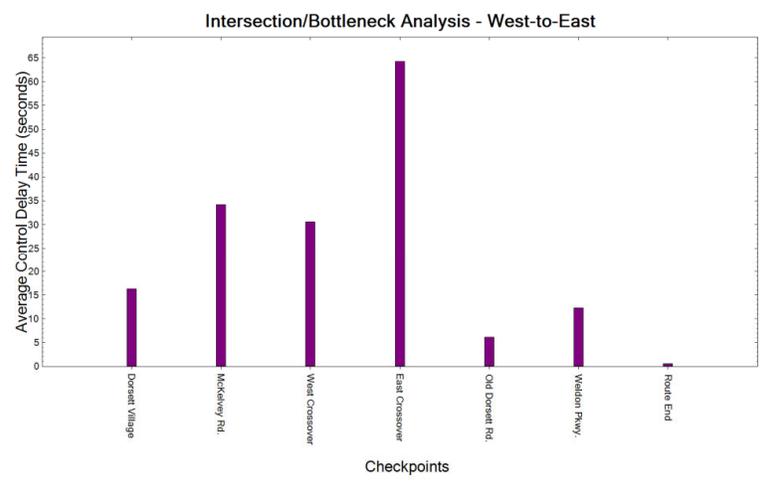
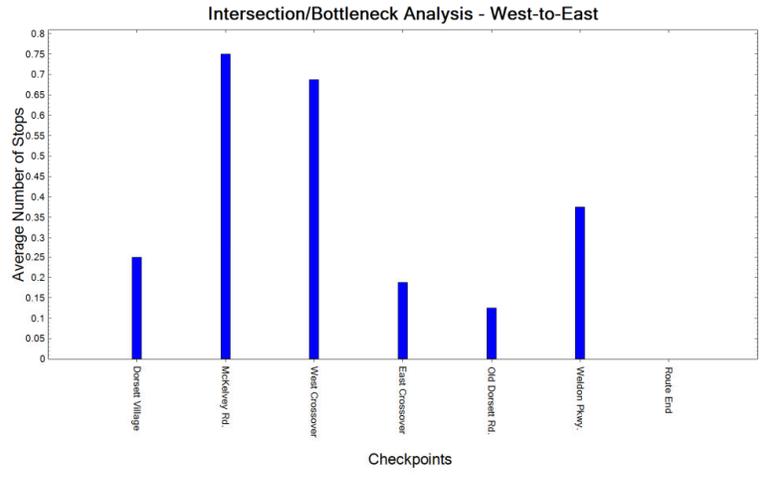


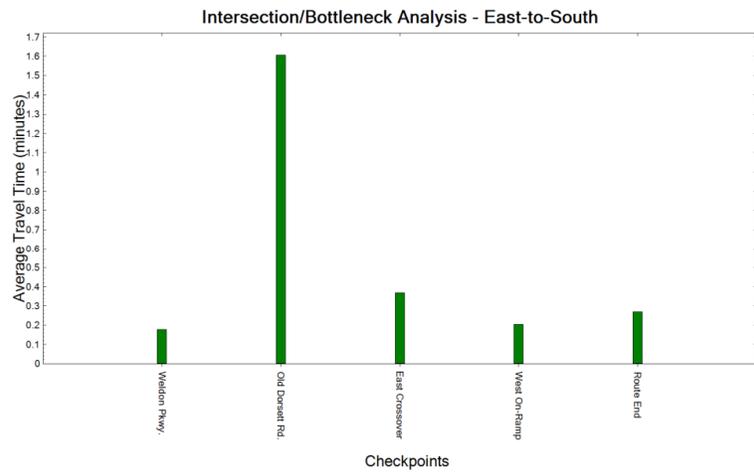
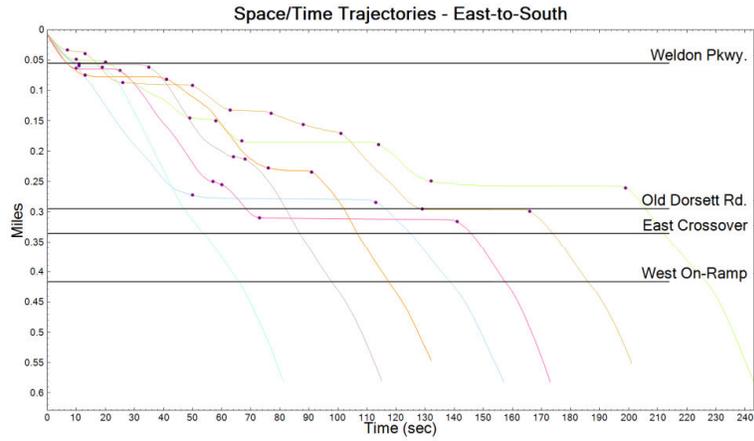
PM Peak Period After Data

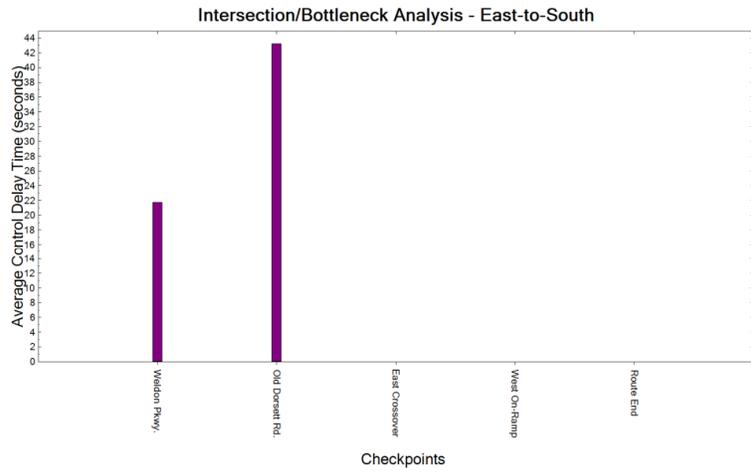
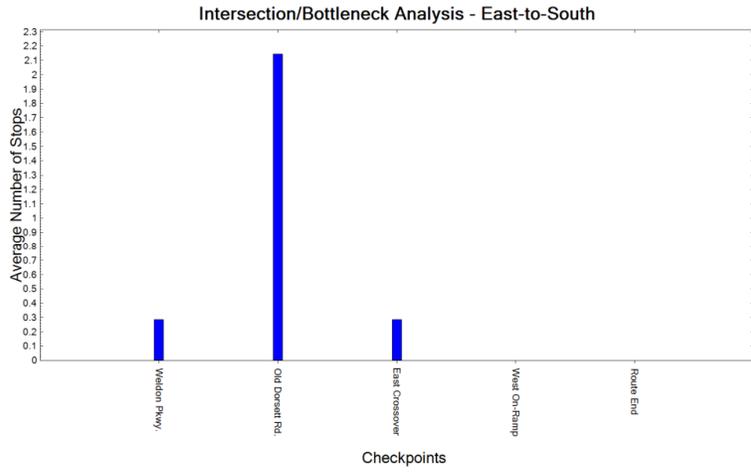


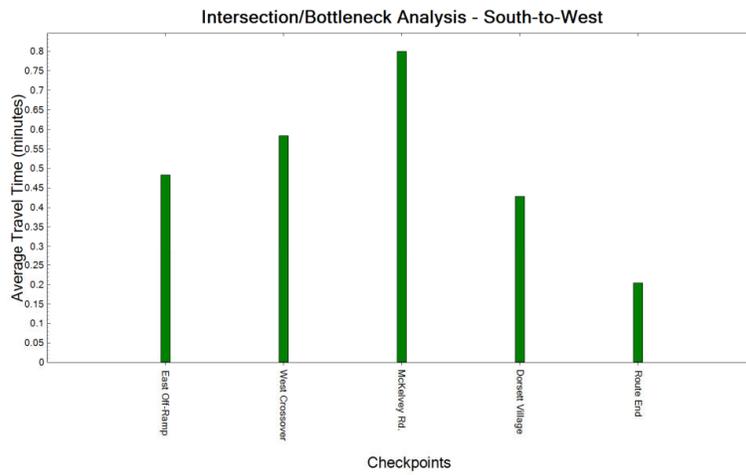
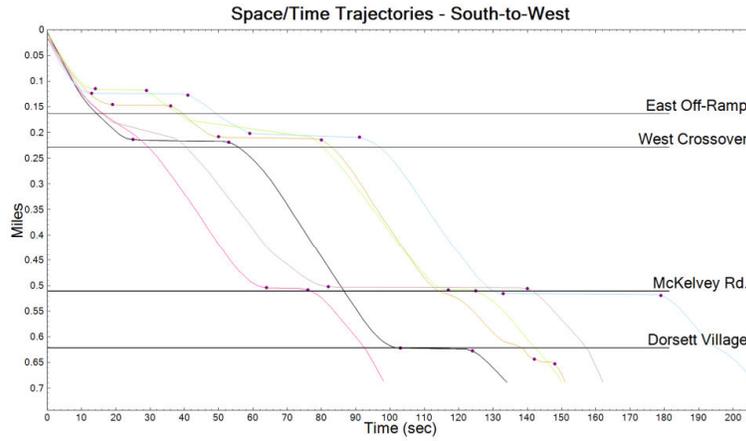


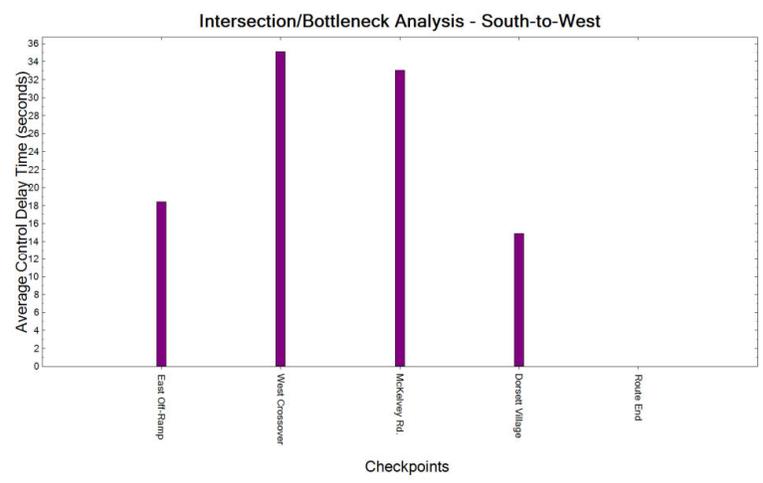
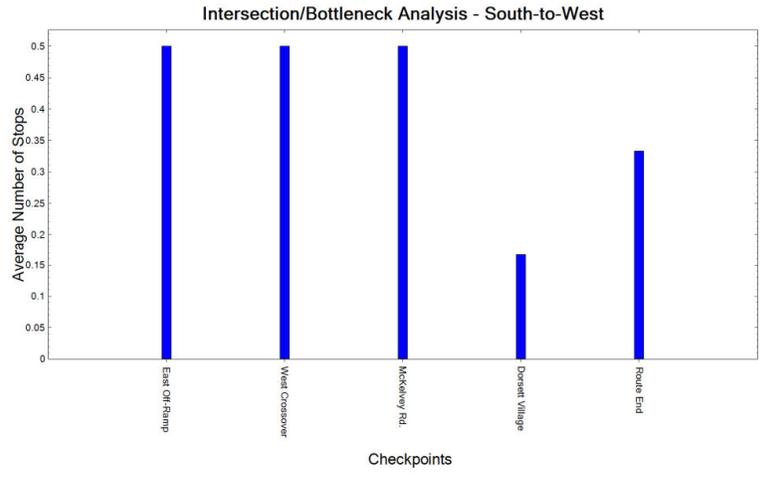


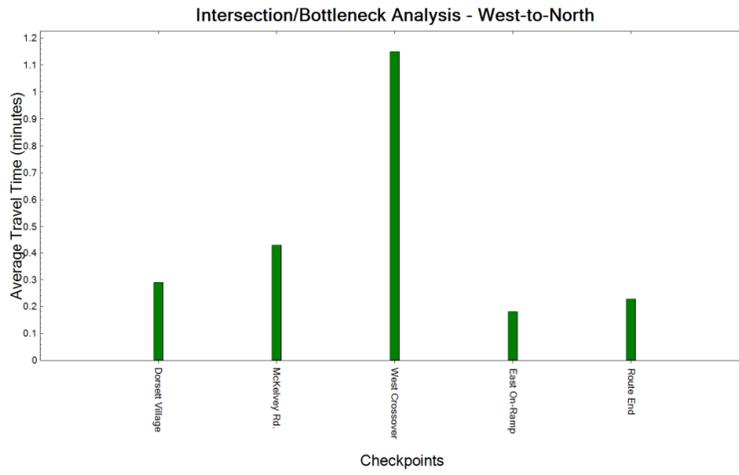
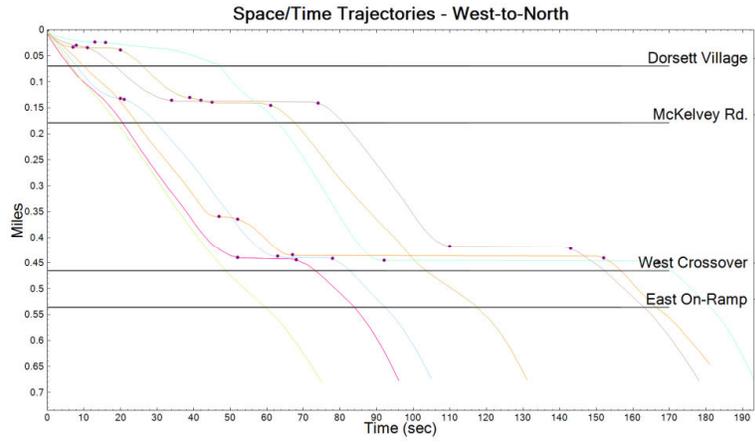


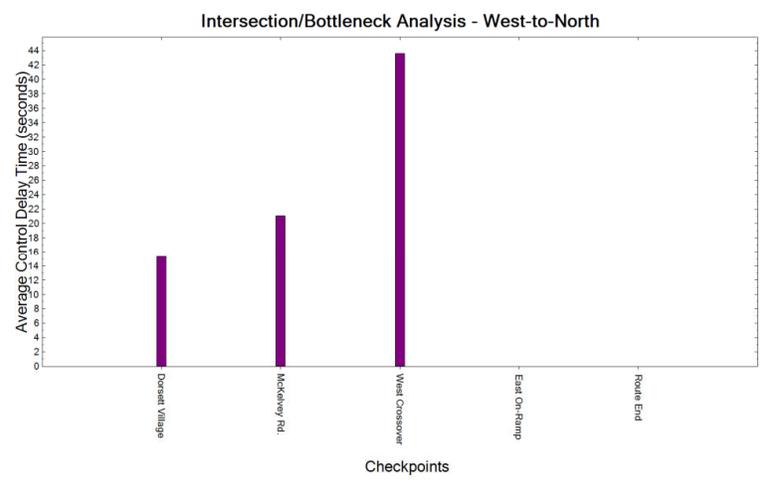
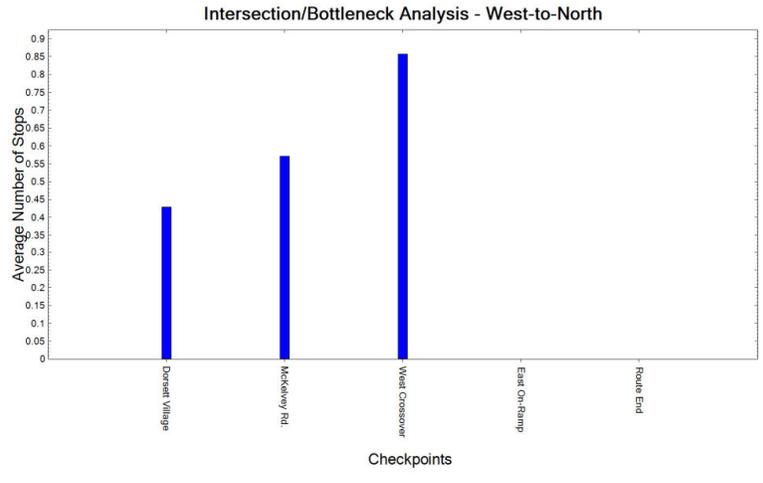


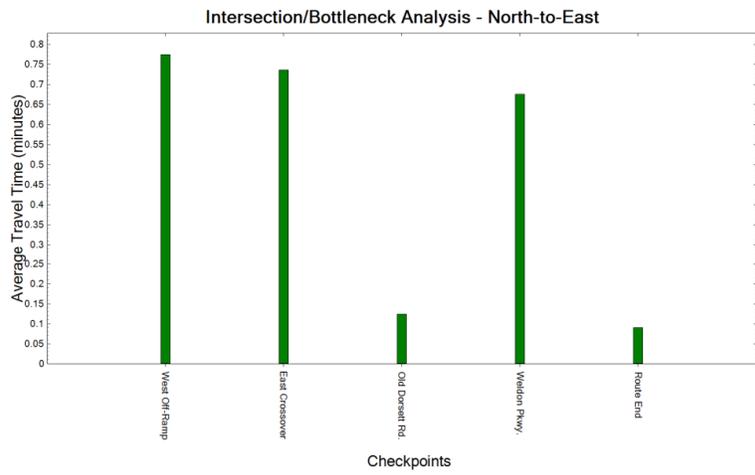
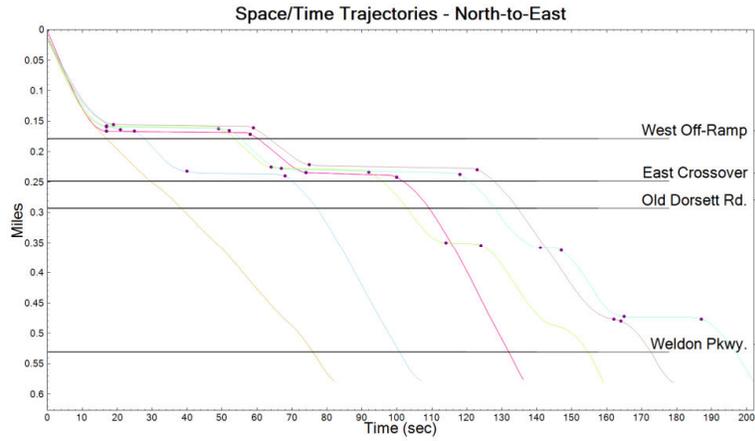


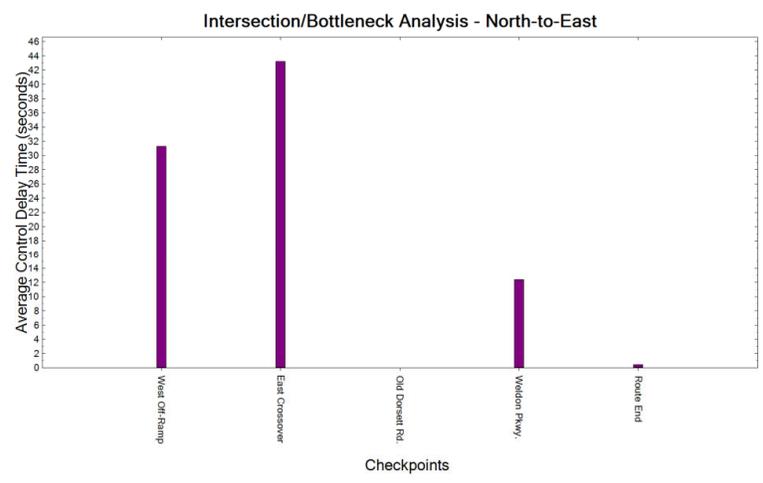
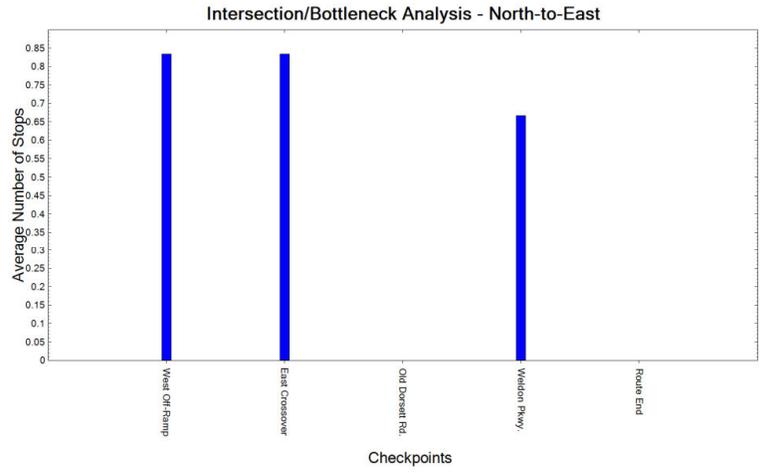












APPENDIX I:

SimTraffic Output from MO-13 DCD, Springfield, MO

Arterial Level of Service
MO-13 After Scenario

12/31/2011

Arterial Level of Service: NB #1

Cross Street	Node	Delay (s/veh)	Travel time (s)	Dist (mi)	Arterial Speed	Run 1 Speed	Run 1 Delay
I-44 EB Off-Ramp RT	24	3.8	16.0	0.1	30	29	4.3
MO-13 NB	6	6.7	9.1	0.0	9	8	7.8
I-44 WB On-Ramp LT	2	26.3	32.5	0.1	6	9	16.8
MO-13 SB #2	3	16.1	17.2	0.0	5	5	14.7
I-44 WB Off-Ramp RT	1	6.3	9.4	0.0	9	9	6.0
Norton Rd.	18	3.1	13.2	0.1	30	30	3.2
	11	1.0	9.3	0.1	30	30	1.1
Total		63.2	106.7	0.4	15	16	54.0

Arterial Level of Service: NB #1

Cross Street	Run 4 Speed	Run 4 Delay	Run 5 Speed	Run 5 Delay	Run 6 Speed	Run 6 Delay	Run 7 Speed
I-44 EB Off-Ramp RT	30	3.2	30	3.8	30	3.6	30
MO-13 NB	9	6.5	9	7.3	10	6.1	10
I-44 WB On-Ramp LT	5	33.9	5	39.3	7	23.1	7
MO-13 SB #2	5	17.9	5	17.4	5	17.0	5
I-44 WB Off-Ramp RT	9	6.5	9	6.4	9	6.0	9
Norton Rd.	30	3.2	29	3.5	31	2.9	30
	30	1.1	30	1.1	30	1.0	30
Total	14	72.1	13	78.6	16	59.6	16

Arterial Level of Service: NB #1

Cross Street	Run 4 Delay	Run 5 Speed	Run 6 Delay
I-44 EB Off-Ramp RT	3.5	29	4.0
MO-13 NB	6.5	10	5.9
I-44 WB On-Ramp LT	22.4	7	24.6
MO-13 SB #2	14.6	5	15.1
I-44 WB Off-Ramp RT	6.4	9	6.5
Norton Rd.	2.8	31	2.7
	0.9	30	0.9
Total	57.1	15	59.7

Educational Use Only

Arterial Level of Service
MO-13 After Scenario

12/31/2011

Arterial Level of Service: SB #2

Cross Street	Node	Delay (s/veh)	Travel time (s)	Dist (mi)	Arterial Speed	Run 1 Speed	Run 1 Delay
	13	763.0	773.5	0.1	2	2	876.2
Norton Rd.	19	109.3	116.9	0.1	2	2	111.1
I-44 WB On-Ramp RT	4	77.8	90.0	0.1	5	5	74.6
MO-13 SB #2	3	25.0	27.1	0.0	3	3	24.6
I-44 WB Off-Ramp LT	5	3.8	6.5	0.0	14	15	3.3
I-44 EB On-Ramp LT #	7	15.8	21.9	0.1	10	11	12.6
I-44 EB On-Ramp RT	26	2.1	4.2	0.0	17	19	1.7
Total		996.7	1040.0	0.4	4	4	1104.1

Arterial Level of Service: SB #2

Cross Street	Run 4 Speed	Run 4 Delay	Run 5 Speed	Run 5 Delay	Run 6 Speed	Run 6 Delay	Run 7 Speed
	2	917.7	2	980.6	3	532.8	2
Norton Rd.	2	117.1	2	121.9	3	99.8	2
I-44 WB On-Ramp RT	4	83.1	4	83.4	5	77.5	4
MO-13 SB #2	3	25.8	3	25.3	3	25.5	3
I-44 WB Off-Ramp LT	13	4.3	12	4.7	14	3.7	14
I-44 EB On-Ramp LT #	9	17.7	9	17.0	9	18.2	9
I-44 EB On-Ramp RT	18	1.9	17	2.1	15	2.5	17
Total	3	1167.7	3	1235.0	4	760.1	4

Arterial Level of Service: SB #2

Cross Street	Run 4 Delay	Run 5 Speed	Run 6 Delay
	892.2	3	455.7
Norton Rd.	110.4	3	98.3
I-44 WB On-Ramp RT	78.8	5	70.8
MO-13 SB #2	24.7	3	24.1
I-44 WB Off-Ramp LT	3.6	15	3.1
I-44 EB On-Ramp LT #	16.4	11	12.8
I-44 EB On-Ramp RT	2.1	18	1.9
Total	1128.2	4	666.7

Educational Use Only

Arterial Level of Service
MO-13 After Scenario

12/31/2011

Arterial Level of Service: NB MO-13 NB

Cross Street	Node	Delay (s/veh)	Travel time (s)	Dist (mi)	Arterial Speed	Run 1 Speed	Run 1 Delay
Evergreen St	32	4.8	15.0	0.1	25	25	4.5
I-44 EB On-Ramp RT	10	5.6	20.2	0.1	24	25	4.6
MO-13 SB	9	28.4	30.7	0.0	3	3	27.0
I-44 EB Off-Ramp LT	6	1.7	3.8	0.0	21	21	1.5
Total		40.5	69.7	0.3	14	15	37.6

Arterial Level of Service: NB MO-13 NB

Cross Street	Run 4 Speed	Run 4 Delay	Run 5 Speed	Run 5 Delay	Run 6 Speed	Run 6 Delay	Run 7 Speed
Evergreen St	25	4.5	24	5.3	24	5.1	24
I-44 EB On-Ramp RT	27	3.7	23	6.7	25	4.7	24
MO-13 SB	3	27.5	3	28.3	3	25.7	2
I-44 EB Off-Ramp LT	21	1.5	21	1.7	22	1.5	18
Total	15	37.1	14	42.0	15	36.9	14

Arterial Level of Service: NB MO-13 NB

Cross Street	Run 4 Delay	Run 4 Speed	Run 5 Delay
Evergreen St	5.0	25	4.6
I-44 EB On-Ramp RT	6.1	22	7.5
MO-13 SB	31.1	2	30.8
I-44 EB Off-Ramp LT	2.2	21	1.6
Total	44.3	14	44.6

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Arterial Level of Service: SB MO-13 SB

Cross Street	Node	Delay (s/veh)	Travel time (s)	Dist (mi)	Arterial Speed	Run 1 Speed	Run 1 Delay
MO-13 NB	9	5.6	6.7	0.0	12	14	4.6
I-44 EB Off-Ramp RT	8	2.2	5.1	0.0	15	15	2.2
Evergreen St.	31	13.6	27.6	0.1	17	17	14.3
Total		21.4	39.4	0.2	16	16	21.2

Arterial Level of Service: SB MO-13 SB

Cross Street	Run 4 Speed	Run 4 Delay	Run 5 Speed	Run 5 Delay	Run 6 Speed	Run 6 Delay	Run 7 Speed
MO-13 NB	12	5.9	12	5.7	11	6.5	11
I-44 EB Off-Ramp RT	15	2.2	16	1.9	14	2.5	14
Evergreen St.	18	12.7	17	13.8	17	13.8	17
Total	16	20.9	16	21.4	16	22.7	16

Arterial Level of Service: SB MO-13 SB

Cross Street	Run 1 Delay	Run 2 Speed	Run 3 Delay	Run 4 AM 123111 Delay
MO-13 NB	6.2	15	4.6	
I-44 EB Off-Ramp RT	2.5	16	2.0	
Evergreen St.	13.6	18	13.2	
Total	22.3	17	19.7	

Educational Use Only

Arterial Level of Service
MO-13 After Scenario

12/31/2011

Arterial Level of Service: NB #3

Cross Street	Node	Delay (s/veh)	Travel time (s)	Dist (mi)	Arterial Speed	Run 1 Speed	Run 1 Delay
Evergreen St.	32	4.8	15.0	0.1	25	25	4.5
I-44 EB On-Ramp RT	10	5.6	20.2	0.1	24	25	4.6
MO-13 SB #4	9	28.4	30.7	0.0	3	3	27.0
I-44 EB Off-Ramp LT	6	1.7	3.8	0.0	21	21	1.5
I-44 WB On-Ramp LT #	2	10.5	16.6	0.1	13	13	9.7
I-44 WB On-Ramp #3	15	4.0	6.3	0.0	12	18	2.0
Total		55.0	92.5	0.4	14	15	49.3

Arterial Level of Service: NB #3

Cross Street	Run 4 Speed	Run 4 Delay	Run 5 Speed	Run 5 Delay	Run 6 Speed	Run 6 Delay	Run 7 Speed
Evergreen St.	25	4.5	24	5.3	24	5.1	24
I-44 EB On-Ramp RT	27	3.7	23	6.7	25	4.7	24
MO-13 SB #4	3	27.5	3	28.3	3	25.7	2
I-44 EB Off-Ramp LT	21	1.5	21	1.7	22	1.5	18
I-44 WB On-Ramp LT #	12	11.6	13	10.2	12	10.6	12
I-44 WB On-Ramp #3	11	4.9	8	7.0	16	2.7	13
Total	14	53.7	13	59.2	15	50.2	13

Arterial Level of Service: NB #3

Cross Street	Run 13 Delay	Run 13 Speed	Run 13 Delay
Evergreen St.	5.0	25	4.6
I-44 EB On-Ramp RT	6.1	22	7.4
MO-13 SB #4	31.1	2	30.8
I-44 EB Off-Ramp LT	2.2	21	1.6
I-44 WB On-Ramp LT #	11.6	13	9.4
I-44 WB On-Ramp #3	3.8	13	3.8
Total	59.7	14	57.7

Educational Use Only

Arterial Level of Service
MO-13 After Scenario

12/31/2011

Arterial Level of Service: SB #4

Cross Street	Node	Delay (s/veh)	Travel time (s)	Dist (mi)	Arterial Speed	Run 1 Speed	Run 1 Delay
I-44 WB Off-Ramp RT	17	25.8	36.1	0.1	12	13	22.5
MO-13 SB #2	5	18.7	20.8	0.0	4	4	16.7
I-44 EB On-Ramp LT #	7	5.9	12.0	0.1	17	17	6.0
MO-13 NB #3	9	5.6	6.7	0.0	12	14	4.6
I-44 EB Off-Ramp RT	8	2.2	5.1	0.0	15	15	2.2
Evergreen St.	31	13.6	27.6	0.1	17	17	14.3
Total		71.7	108.4	0.4	13	13	66.5

Arterial Level of Service: SB #4

Cross Street	Run 4 Speed	Run 4 Delay	Run 5 Speed	Run 5 Delay	Run 6 Speed	Run 6 Delay	Run 7 Speed
I-44 WB Off-Ramp RT	13	19.6	10	36.5	10	32.8	11
MO-13 SB #2	4	18.7	4	19.8	3	20.1	4
I-44 EB On-Ramp LT #	17	6.0	17	6.3	17	5.9	16
MO-13 NB #3	12	5.9	12	5.7	11	6.5	11
I-44 EB Off-Ramp RT	15	2.2	16	1.9	14	2.5	14
Evergreen St.	18	12.7	17	13.8	17	13.8	17
Total	13	65.2	12	84.0	12	81.5	12

Arterial Level of Service: SB #4

Cross Street	Run 13 Delay	Run 13 Speed	Run 13 Delay
I-44 WB Off-Ramp RT	25.6	14	17.9
MO-13 SB #2	19.5	4	17.0
I-44 EB On-Ramp LT #	6.9	20	4.6
MO-13 NB #3	6.2	15	4.6
I-44 EB Off-Ramp RT	2.5	16	2.0
Evergreen St.	13.6	18	13.2
Total	74.4	14	59.3

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Arterial Level of Service: NB #1

Cross Street	Node	Delay (s/veh)	Travel time (s)	Dist (mi)	Arterial Speed
I-44 EB Off-Ramp RT	24	3.4	15.5	0.1	30
MO-13 NB	6	13.6	16.0	0.0	5
I-44 WB On-Ramp LT	2	8.2	14.2	0.1	15
MO-13 SB #2	3	4.1	5.2	0.0	16
I-44 WB Off-Ramp RT	1	3.2	6.2	0.0	14
Norton Rd.	18	12.2	23.2	0.1	17
	11	1.5	9.9	0.1	29
Total		46.1	90.1	0.4	18

Arterial Level of Service: SB #2

Cross Street	Node	Delay (s/veh)	Travel time (s)	Dist (mi)	Arterial Speed
	13	0.6	10.7	0.1	35
Norton Rd.	19	21.4	28.9	0.1	9
I-44 WB On-Ramp RT	4	65.2	77.4	0.1	5
MO-13 SB #2	3	24.1	26.2	0.0	3
I-44 WB Off-Ramp LT	5	2.3	5.0	0.0	18
I-44 EB On-Ramp LT #	7	19.4	25.5	0.1	8
I-44 EB On-Ramp RT	26	2.8	4.9	0.0	15
Total		135.7	178.7	0.4	8

Arterial Level of Service: NB MO-13 NB

Cross Street	Node	Delay (s/veh)	Travel time (s)	Dist (mi)	Arterial Speed
Evergreen St.	32	16.6	27.4	0.1	13
I-44 EB On-Ramp RT	10	24.5	39.1	0.1	12
MO-13 SB	9	21.5	23.8	0.0	3
I-44 EB Off-Ramp LT	6	2.0	4.1	0.0	19
Total		64.6	94.4	0.3	11

Arterial Level of Service: SB MO-13 SB

Cross Street	Node	Delay (s/veh)	Travel time (s)	Dist (mi)	Arterial Speed
MO-13 NB	9	17.3	18.4	0.0	8
I-44 EB Off-Ramp RT	8	6.0	8.9	0.0	9
Evergreen St.	31	4.1	12.6	0.1	38
Total		27.4	39.9	0.2	20

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Arterial Level of Service: NB #3

Cross Street	Node	Delay (s/veh)	Travel time (s)	Dist (mi)	Arterial Speed
Evergreen St.	32	16.6	27.4	0.1	13
I-44 EB On-Ramp RT	10	24.5	39.1	0.1	12
MO-13 SB #4	9	21.5	23.8	0.0	3
I-44 EB Off-Ramp LT	6	2.0	4.1	0.0	19
I-44 WB On-Ramp LT #	2	4.4	10.4	0.1	20
I-44 WB On-Ramp #3	15	0.8	3.1	0.0	25
Total		69.8	107.9	0.4	12

Arterial Level of Service: SB #4

Cross Street	Node	Delay (s/veh)	Travel time (s)	Dist (mi)	Arterial Speed
I-44 WB Off-Ramp RT	17	25.4	35.8	0.1	11
MO-13 SB #2	5	27.3	29.5	0.0	3
I-44 EB On-Ramp LT #	7	42.2	48.3	0.1	4
MO-13 NB #3	9	17.3	18.4	0.0	8
I-44 EB Off-Ramp RT	8	6.0	8.9	0.0	9
Evergreen St.	31	4.1	12.6	0.1	38
Total		122.3	153.4	0.4	9

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