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Developing Warrants for Designing Continuous Flow Intersection and Diverging Diamond Interchange

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DEVELOPING WARRANTS FOR DESIGNING CONTINUOUS FLOW INTERSECTION AND DIVERGING DIAMOND INTERCHANGE

by

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ABSTRACT

The main goal of this dissertation is to have better understanding of design and operation of the Continuous Flow Intersection (CFI) and Diverging Diamond Interchange (DDI) - as well as numerous factors that affect signalized intersection and interchange performance due to increased left-turn demand. The dissertation attempts to assess the need and justification to redesign intersections and interchanges to improve their efficiency. And to that end, an extensive literature review of existing studies was done with the prime aim of perceiving the principles of these innovative designs and determining the methodology to-be-followed, in order to reach the study's core. Accordingly, several DDI and CFI locations were selected as candidate locations, where the designs have already been implemented and the required data - to model calibration and validation - was collected. The micro-simulation software (VISSIM 8.0) was used for simulation, calibration and validation of the existing conditions - through several steps - including signal optimization and driving behavior parameter sensitivity analysis. Subsequently, an experiment was conceived for each design, aiming at examining several factors that affect each design's efficiency. The experiment comprised 180 and 90 different CFI & DDI scenarios and their conventional designs, respectively. Two measures of effectiveness were identified for result analysis: the average delay and capacity. Result analyses were performed to detect switching thresholds (from conventional to innovative designs. In addition, performance comparison studies of the CFI and DDI with their conventional designs were performed. The results and findings will serve as guidelines for decision-makers as to when they should consider switching from conventional to innovative design. Finally, decision support systems were developed to speed up the search for the superior design, in comparison with others.

Keywords: Innovative Design, Continuous Flow Intersection, Signalized Intersection, Traffic Operation, CFI, DDI

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At the end I would like express appreciation to my beloved wife Laila, who spent sleepless nights with and was always my support in the moments when there was no one to answer my queries.

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CHAPTER 1: INTRODUCTION

1.1 Background

The U.S. highway system has been designed to tolerate and accommodate traffic patterns that no longer exist, nor can be handled. Traffic demand for the past few decades, all over the world, has been continuously increasing due to intense exponential growth of the population, resulting in more people travelling farther and spending more time on roads, which contribute to road deterioration and traffic jams. Heavy traffic on highways will lead to congestion at many intersections and interchanges along the system. The traffic will build up and will not accommodate such new traffic pattern (Hummer, 1998; Hummer and Reid, 2000; Dhatrek et al., 2010; Abou-Senna et al., 2015). The congestion at intersections and interchanges is mainly caused by the high left-turns onto arterials or crossroads.

Transportation engineers, around the world, have been working on new alternatives and countermeasures to improve operational and safety performance at intersections and interchanges. Constrained by limited resources, researchers and professionals were forced to develop several countermeasures/solutions to relieve congestion and improve the level of service (LOS) at such designs (El Esawey and Sayed, 2013). The conventional countermeasures, which have been adopted by the transportation engineers around the world, are categorized as the first approach to mitigate congestion by double left-turn lanes, increasing signal cycle length, coordination and signal synchronization (Dhatrek et al., 2010). The implementation of these conventional solutions is limited as the modifications of any design - such as widening the right-of-way and improving alternative routes - are expensive and disruptive (Cheong et al., 2008; Hu et al., 2014). Adjustments to cycle lengths and signal coordination merely provide marginal improvements to saturated intersections (Dhatrek et al., 2010). When conventional measures are inadequate or countermeasures unfeasible, a grade separation may be considered as a second countermeasure. Considering the grade separation as a feasible countermeasure to traffic operation problems - at major intersections - has major improvements (Goldblatt et al., 1994). However, this conventional measure cannot be implemented in many cases, due to time demand and costs related to the construction of grade-separation design (Goldblatt et al., 1994). In addition, when the grade-separation structure (conventional interchange) is implemented and the location is really experiencing heavy traffic flows, the conventional grade-separated approach will not be suitable.

After attempting to solve such congestion nuisance, using the previous two countermeasures with no feasible or optimum solution, the transportation engineers and other professionals, for the past decades, have been forced to develop innovative/unconventional designs to remedy congestion at the signalized intersections and interchanges, which are categorized as the third countermeasure. Many innovative designs, such as Continuous Flow Intersection (CFI) and Diverging Diamond Interchange (DDI), have been developed by the transportation professionals to efficiently improve traffic performance and accommodate less-costly heavy-traffic patterns, typically by shifting the left-turn movement to the other direction, which would reduce the number of signal phases at the intersection or interchange. The principles of these two innovative designs have been proven by other researches, who were able to improve traffic operations and safety beyond the capabilities of other conventional designs (Goldblatt et al.,1994; Abou-Senna et al., 2015; Abou-Senna et al., 2016]. Other designs have exclusively studied through micro-simulation such as the Upstream Signalized Crossover (USC) and the Double Crossover Intersection (DXI) (Abou-Senna et al., 2015; Sayed et al., 2006; Autey et al., 2013). Since this study is mainly focused on CFI and DDI designs, a detailed section on understanding the two design principles and how they are working will be discussed later in this paper.

1.2 Research Objectives

For the past decades, there has been many worth-reading research studies, which analyzed and assessed CFI and DDI innovative designs' operational and safety performance (El Esawey and Sayed, 2013; Reid, 2000; Reid and Hummer, 1999; Dhatrek et al., 2010). Most of them were based on comparison between the innovative designs - with one another or with its own conventional design (Cheong et al., 2008; Sharma and Chatterjee, 2007; Kaisar et al., 2011; Abou-Senna and Radwan, 2016). Some studies tested these designs so that they could be used to guide authorities, when one design should be replaced by a better one, based on performance comparison between the new and conventional designs. However, these studies did not build their guidelines based on technical needs' assessment and justification seeking, when an intersection or interchange's redesigning is necessary.

The purpose of this dissertation is to better understand the innovative designs and the different factors that affect signalized intersection/interchange performance (due to increased left-turn demand) and assess the need/justification to redesign intersections/interchanges and improve their efficiency. Such guidelines will help transportation engineers whenever they need to apply CFI and DDI designs. The aspiration of this dissertation is met when the following questions are addressed:

- Do the current left-turn management strategies - at signalized intersections, street facilities and freeway interchanges - work well?
- Do the proposed special left-turn treatments have shown promise to improve the operation of such facilities (CFI and DDI)?
- Can the current evaluation tools (micro-simulation programs) be used, complemented with field data, to simulate conditions at such facilities?

- Is it possible to design an experiment that can give us a better understanding of the different factors that affect the efficiency of these facility designs?
- Can this experiment produce warrants for switching from a conventional intersection/interchange design to one of the proposed special treatments?
- Would these warrants be beneficial to decision-makers?

To address the first two questions, an extensive literature review was carried out and a fully detailed chapter was written. To address the rest of the dissertation questions, the following tasks were performed:

Task1: Select sites that are good candidates for field data collection

Task2: Use the selected micro-simulation tool to simulate conditions for selected field sites

Task3: Calibrate and validate the simulation models using field data

Task4: Design an experiment that utilizes one existing intersection and a second existing interchange with a variety of traffic demands, turning movements, geometric characteristics, and other critical factors

Task5: Use appropriate MOEs when searching for crossing points - from Diamond interchange to DDI and from conventional intersection to CFI

Task6: Use the developed warrants for each non-conventional design type and check against existing CFIs and DDIs.

1.3 Dissertation Organization

The dissertation's structure starts in Chapter 1, which introduces the innovative designs and proposal's objectives and organization. Chapter 2 presents a fully detailed literature review encompassing previous studies and research papers on Continuous Flow Intersections and Diverging Diamond Interchanges, including scopes, methodologies, analysis tools, results and future works of each literature. The methodology observed to reach the dissertation's purpose is covered in Chapter 3. Chapter 4 includes the steps (location and data collection, simulation tool selection, modeling, evaluation, MOEs, experiment design) that were followed in order to build the Diverging Diamond Interchange warrants. Chapter 4 and 5 documents the coding of candidate-sites and how to use field data to calibrate and validate the developed models, by using the appropriate micro-simulation tool for the Continuous Flow Intersection and Diverging Diamond Interchange. Both Chapters 4 and 5 include each design's analyses and results. Paper's conclusion and discussion are presented in Chapter 6.

CHAPTER 2: LITERATURE REVIEW

2.1 Continuous Flow Intersection (CFI)

Goldblatt et al. (1994) studied the effectiveness of applying the Continuous Flow Intersection (CFI) and how it enhances traffic operations by comparing CFI with conventional designs operated under multi-phased actuated control. The evaluation process of these two designs was done (using the TRAF-NETSIM simulation model) under three different traffic scenarios, 1500, 2000, and 3000 vehicles per hour (vph), respectively, on all four approaches. For the turning movements, the percentage of each movement for all approaches was set in 15% for left-turn movement, 11% for right-turn movement, and 5% of truck on the traffic stream. The evaluation and comparison were done based on 12 measures of effectiveness, being: vehicle-trips, total delay, moving/total time, delay per vehicle, average speed, storage, phase failure, fuel, HC emissions, NOX emissions, % demand and CO emissions. Results in Goldblatt et al. (1994) study show that CFIs outperformed the conventional design especially when (a) the demand volume 'about to reach' or 'exceed' the capacity and (b) the left-turn movement required protected phases, which is a promising improvement on the operational performance for those intersections experiencing congestions. The authors suggest performance-assessment, including the overall cost-benefit ratio, when options are the conventional and CFI designs; nevertheless, if options are CFI and grade-separation designs, it is worth considering CFI because it is the optimal solution for both (operational performance and cost-benefit ratio).

Hummer and Reid (2000) reviewed five unconventional intersection-designs (median U-turn, bowtie, superstreet, jughandle and continuous flow intersection) and summarized the information providing details on each alternative design, which is the main purpose of their work. The summary includes: an explanation of the innovative design, including a diagram, new research or implementation review for the new design (if available), a list of advantages/disadvantages of such design, when compared to its conventional version, and an in-brief 'when/where' authorities or agencies should consider the alternative. Previous designs have the same two main advantages, i.e.: (1) delay-reduction for the through movement; and (2) conflict-point reduction at the intersection, by rerouting some movements; The first principle is related to the operational performance and the second to safety. If delay (for through movement) is reduced, traffic progression along the arterial will be better; and if conflict-points are reduced, life-threatening conditions, which put the driver in danger, will also drop. Operational and safety advantages, at an intersection (when implementing the unconventional design) are related to signal-phases reduction, from four (assuming no overlaps) to just two phases. The paper suggests the implementation of the unconventional intersection, along the corridor, rather than an isolated intersection and the same design to prevent driver-confusion when travelling along the road.

The Cheong et al. (2008) study focused on assessing and comparing three innovative designs, (Continuous Flow Intersection (CFI), Parallel Flow Intersection (PFI) and Upstream Signalized Crossover (USC)), and comparison between each unconventional design and its regular design, from an operational point of view. The average delay of each intersection, for the through traffic only and left-turn traffic only was selected in this research to be the measure of effectiveness (MOE) to perform the comparison MOE in this study was affected by many factors, the authors designed an experiment to properly evaluate and compare the operational performance of intersections. The factors included in the experiment were: 1) various 'through' and 'left-turn' traffic volumes; 2) geometric design; 3) signal plans. For the balanced volume scenarios, the traffic volume for each unconventional intersection was set out to be 1000vph as low volume level, 1500vph as moderate volume level and 1800vph as high-volume level. For the conventional design, the traffic volume was set out to be 1000vph and 1200vph. The percentage of ‘right-turn’ volume, for both designs and scenarios, was fixed at 10% and various percentages of ‘left-turn’ volume (5%, 10%, 20% and 25%) were set as well. For the unbalanced volume scenarios, main arterial road’s volume was set in 2000vph and 2500vph as moderate and high-volume levels, respectively. And for the minor cross road the volume was set in 600vph, 900vph and 1300vph as the low, moderate, and high-volume levels, respectively. For the signal plans, the minimum cycle length was calculated by using the minimum cycle length equation. To analyze all the experimental designs, the authors selected VISSIM 4.1 with no change in the default parameters, drive characteristics, lane width, grades or vehicle distribution. Research result showed that the three unconventional designs significantly reduced the average control delay, if compared to the regular design under volume’s moderate level. The CFI design outperformed the PFI and USC, under all traffic volume scenarios; however, under minor road low and moderate scenarios, CFI and PFI showed very similar delay.

Dhatrak et al. (2010) evaluated and compared two unconventional intersections (Parallel Flow Intersection (PFI) and Displaced Left turn Intersection (DLT)). By using unique design layouts and three different traffic volume conditions, the study scope was to determine the maximum traffic flow of ‘through’ and ‘left-turn’ movements for each design and three different volume conditions. In this paper, the maximum throughput volumes for both, ‘through’ and ‘left turn’ movements, may serve as a selection guide when considering these two designs. The criteria used in this study, to determine the maximum traffic flow for any movement were (1) if model output volume for any movement was 150 vehicles lower than the input volumes; (2) if the travel delay for any movement reaches 80 seconds per vehicle. The VISSIM 5.10 traffic simulation tool was chosen due to its ability to model the innovative designs. The optimum cycle length in this study varied from 55 seconds to 80 seconds. After running the simulation (30 runs each) and obtaining the movements' run result, both designs performed the same in terms of throughputs and average intersection delays for the 'through' movement, but the 'left-turn' movement for the DLT outperformed PFI in the throughputs and delays.

The main scope of Olarte and Kaisar (2011) study was to compare three different innovative intersections (Left-turn Bypass, the Diverging Flow intersection and the Displaced Left-turn intersection) from an operational perspective. The first step was to assess the isolated unconventional intersection designs by using the microscopic simulation software VISSIM 5.10 and, then, to apply these intersections along an existing corridor in the state of Florida to test which of the innovative design intersections performed better in different scenarios. Two measures of effectiveness were used, in this study, to test the operational performance between the innovative designs, that is, average control delay time and total number of stops. Results were based on three different scenarios (balanced conditions, unbalanced conditions and an existing corridor, as a case study), which showed that the displaced left-turn intersection outperformed the other two innovative designs - in both delay and number of stops - for almost all the scenarios.

Autey et. al. (2012) compared the operational performance among four unconventional designs (the continuous flow Intersection (CFI), the upstream signalized crossover (USC), the double crossover intersection (DXI) and the median U-turn (MUT) and between the unconventional designs and conventional design, which is a research extension and was based on two unconventional schemes. The study compared each intersection's average control delay and overall capacity. In this study, the micro-simulation software VISSIM 5.10 was used to model and simulate these intersections, as per previous research. To obtain a fair comparison between the intersections, all of them had the same geometric design and traffic volumes scenarios (number of legs, lanes, exclusive left-turn lane (65 m long) and balanced and unbalanced volume scenarios). Results proved that all unconventional designs performed better than the conventional intersection designs and the CFI has always outperformed the other innovative designs in all volume scenarios.

Several studies were done on unconventional intersection designs and many suggest the use of those designs owing to their benefits in enhancing the operational and safety performances. Many studies suggest higher capacity, lower delay and fewer crashes can be obtained by switching from conventional intersection to unconventional design. El Esawey and Sayed (2012) did an extensive literature review of the existing studies that analyze innovative intersection designs' operational and safety performances. Each article is different in terms of scope, methodology, analysis tools, MOE and future research. This paper states that the average delay is the most common effectiveness measure to compare intersections in relation to operational performance. Furthermore, several approaches propose safety-performance alternatives for different innovative designs, which can be classified as (1) number of conflict-points, (2) before-and-after cross sectional analysis, (3) driver confusion and human behavior studies and (4) using safety assessment based on micro-simulation. Moreover, this study suggests other areas that need further investigation including pedestrian-movement analysis, cost-benefit assessments, environmental impacts and safety evaluations.

The major aim of the Continuous Flow Intersection (CFI) guidelines, introduced by the Utah Department of Transportation (UDOT) in 2013, is to accelerate CFI acceptance throughout the state and to identify the main design elements. The guidelines consolidate this goal by providing a detailed accounting of key concept principles, design variations, decision-making factors, evaluation standards, design standards and lessons learned from 11 CFI implementations throughout the state in six years (2007-2013). The UDOT recognizes that the CFI design can be provided for a very reasonable price (less than 10 million dollars), if compared to other designs and the CFI showed safety improvements that include fewer conflict-points and a 30% to 70% reduction in travel time and intersection delay. All these advantages are provided with minimal driver inconvenience, no out of direction travel and new opportunities in term of access management/consolidation.

Abou-Senna et. al. (2015) did a comprehensive review and assessment of several innovative designs, which mainly focused on eliminating the left-turn phase. They have assessed the benefits and challenges these designs from operational and safety aspects related to traffic, bicycles and pedestrians. The operational assessment was based on each alternative design's advantages and disadvantages and a variety of parameters that need to be taken into consideration when taking into account one of the alternative designs. They also assessed driver-confusion possibilities that might be provoked by the implementation of innovative designs and maintenance impacts.

An operational evaluation of 'partial crossover displaced left-turn (XDL)' *versus* 'full XDL intersection' was done by Abou-Senna and Radwan (2016). The study explained the CFI concept and how it works and what kind of traffic volume works better with such design. The research considered overall intersection-performance, of an existing intersection in Orlando, Florida, which has two heavy conflicting movements that operate near capacity. The results of this study showed that the CFI increased the capacity in 25%, reduced delay in 30-45 % (for the critical movements), and reduced queue length in 25-40 %.

After an extensive literature review, related to Continuous Flow Intersection, the following tables (1, 2, 3, 4, and 5) summarize the main objective, methodology, MOEs, research tools and results, for each literature:

Table 1. Summary of main objective of each research

Study	***Main objective of study***
Goldblatt et al. (1994)	Evaluate the performance of traffic at CFI designs
Hummer and Reid (2000)	Summarizing a full detailed information of five UAIDs
Cheong et al. (2008)	Evaluate the performance of three unconventional designs (CFI, PFI, and USC)
Dhatrak et al. (2010)	Determine the maximum traffic flow of through and left movements for PFI and DLT intersections and use them as a selection guide for those designs
Olarte and Kaisar (2011)	Evaluate the operational performance between three innovative designs
Autey et. al. (2012)	Evaluate the operational performance between 4 innovative designs (CFI, USC, DXI, and MUT)
El Esawey and Sayed (2012)	Summary many literature reviews about the UAIDs
UDOT CFI guideline (2013)	To accelerate acceptance of the CFI throughout the State, and to formalize the critical design elements to help foster acceptance
Abou-Senna et. al. (2015)	A comprehensive review and assessment for several innovative designs

Table 2. Summary of analysis methodology of each research

Study	***Analysis methodology of study***
Goldblatt et al. (1994)	Comparing the CFI with the conventional intersection under multi-phased actuated control under three traffic volume scenarios
Hummer and Reid (2000)	Explain the design, advantages and disadvantages of the design, lists of when implementing the design
Cheong et al. (2008)	Comparing the operational performance between the designs under different scenarios
Dhatrak et al. (2010)	If output volume < 150 vehicles lower than input volume, delay reaches 80 seconds, for any movement
Olarte and Kaisar (2011)	Comparing the performance of the new designs along the corridor under different traffic scenarios
Autey et. al. (2012)	Comparing the operational performance of the CFI, USC, DXI, and MUT under the same geometric design and same traffic volume scenarios
El Esawey and Sayed (2012)	Reviewing literature review about UAIDs and summarized the scope, methodology, analysis tools, MOEs, and future research
UDOT CFI guideline (2013)	Writing a detailed accounting of key concept principles, design variations, decision making factors, evaluation standards, design standards, and lessons learned from CFI implementations
Abou-Senna et. al. (2015)	Comparing the operation and safety aspects of each alternative design/benefit-to-cost ratio

Table 3. Summary of measure of effectiveness of each research

Study	***MOEs of study***
Goldblatt et al. (1994)	vehicle-trips, total delay, moving/total time, delay per vehicle, average speed, storage, phase failure, fuel, HC emissions, NOX emissions, % demand, and CO emissions
Hummer and Reid (2000)	Operational and safety performance
Cheong et al. (2008)	Average control delay
Dhatrak et al. (2010)	Delays and Capacity
Olarte and Kaisar (2011)	Average control delay and number of stops
Autey et. al. (2012)	Average control delay and overall capacity
El Esawey and Sayed (2012)	N/A
UDOT CFI guideline(2013)	Delay for all the movement
Abou-Senna et. al. (2015)	Operational and Safety benefits/Benefit-to-cost Ratio

Table 4. Summary of analysis tool of each research

Study	***Analysis tool of study***
Goldblatt et al. (1994)	TRAF-NETSIM simulation tool
Hummer and Reid (2000)	N/A
Cheong et al. (2008)	VISSIM 4.1
Dhatrak et al. (2010)	VISSIM 5.10
Olarte and Kaisar (2011)	VISSIM 5.10
Autey et. al. (2012)	VISSIM 5.10
El Esawey and Sayed (2012)	N/A
UDOT CFI guideline(2013)	VISSIM microscopic simulation tool
Abou-Senna et. al. (2015)	N/A

Table 5. Result summary of each previous study

Study	***Results of study***
Goldblatt et al. (1994)	CFI outperformed the conventional design with high volume demand
Hummer and Reid (2000)	The new designs reducing the delays and conflict points
Cheong et al. (2008)	All the three designs significantly reduced the average delay especially under high level flows
Dhatrak et al. (2010)	Both designs performed the same in throughput and delay for through movement, but left-turn movement for DLT outperformed PFI
Olarte and Kaisar (2011)	The CFI outperformed the other two designs
Autey et. al. (2012)	All the innovative intersections performed better than the conventional intersections and the CFI design outperformed the others.
El Esawey and Sayed (2012)	The average delay is the most used MOE, safety performance classified into 4-classififcations, and more areas need more investigation
UDOT CFI guideline (2013)	Providing a detailed accounting of key concept principles, design variations, decision making factors, evaluation standards, design standards, and lessons learned from CFI implementations throughout the State
Abou-Senna et. al. (2015)	These designs enhance the operational and safety performance with no need to change the existing infrastructure

2.2 Diverging Diamond Interchange (DDI)

Goldblatt et al. (1994) studied the effectiveness of applying the Continuous Flow Intersection (CFI) and how it enhances traffic operations by comparing CFI with conventional designs operated under multi-phased actuated control. The evaluation process of these two designs was done (using the TRAF-NETSIM simulation model) under three different traffic scenarios, 1500, 2000, and 3000 vehicles per hour (vph), respectively, on all four approaches. For the turning movements, the percentage of each movement for all approaches was set in 15% for left-turn movement, 11% for right-turn movement, and 5% of truck on the traffic stream. The evaluation and comparison were done based on 12 measures of effectiveness, being: vehicle-trips, total delay, moving/total time, delay per vehicle, average speed, storage, phase failure, fuel, HC emissions, NOX emissions, % demand and CO emissions. Results in Goldblatt et al. (1994) study show that CFIs outperformed the conventional design especially when (a) the demand volume 'about to reach' or 'exceed' the capacity and (b) the left-turn movement required protected phases, which is a promising improvement on the operational performance for those intersections experiencing congestions. The authors suggest performance-assessment, including the overall cost-benefit ratio, when options are the conventional and CFI designs; nevertheless, if options are CFI and grade-separation designs, it is worth considering CFI because it is the optimal solution for both (operational performance and cost-benefit ratio).

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Results proved that all unconventional designs performed better than the conventional intersection designs and the CFI has always outperformed the other innovative designs in all volume scenarios.

Several studies were done on unconventional intersection designs and many suggest the use of those designs owing to their benefits in enhancing the operational and safety performances. Many studies suggest higher capacity, lower delay and fewer crashes can be obtained by switching from conventional intersection to unconventional design. El Esawey and Sayed (2012) did an extensive literature review of the existing studies that analyze innovative intersection designs' operational and safety performances. Each article is different in terms of scope, methodology, analysis tools, MOE and future research. This paper states that the average delay is the most common effectiveness measure to compare intersections in relation to operational performance. Furthermore, several approaches propose safety-performance alternatives for different innovative designs, which can be classified as (1) number of conflict-points, (2) before-and-after cross sectional analysis, (3) driver confusion and human behavior studies and (4) using safety assessment based on micro-simulation. Moreover, this study suggests other areas that need further investigation including pedestrian-movement analysis, cost-benefit assessments, environmental impacts and safety evaluations.

The major aim of the Continuous Flow Intersection (CFI) guidelines, introduced by the Utah Department of Transportation (UDOT) in 2013, is to accelerate CFI acceptance throughout the state and to identify the main design elements. The guidelines consolidate this goal by providing a detailed accounting of key concept principles, design variations, decision-making factors, evaluation standards, design standards and lessons learned from 11 CFI implementations throughout the state in six years (2007-2013). The UDOT recognizes that the CFI design can be provided for a very reasonable price (less than 10 million dollars), if compared to other designs and the CFI showed safety improvements that include fewer conflict-points and a 30% to 70% reduction in travel time and intersection delay. All these advantages are provided with minimal driver inconvenience, no out of direction travel and new opportunities in term of access management/consolidation.

Abou-Senna et. al. (2015) did a comprehensive review and assessment of several innovative designs, which mainly focused on eliminating the left-turn phase. They have assessed the benefits and challenges these designs from operational and safety aspects related to traffic, bicycles and pedestrians. The operational assessment was based on each alternative design's advantages and disadvantages and a variety of parameters that need to be taken into consideration when taking into account one of the alternative designs. They also assessed driver-confusion possibilities that might be provoked by the implementation of innovative designs and maintenance impacts.

An operational evaluation of 'partial crossover displaced left-turn (XDL)' *versus* 'full XDL intersection' was done by Abou-Senna and Radwan (2016). The study explained the CFI concept and how it works and what kind of traffic volume works better with such design. The research considered overall intersection-performance, of an existing intersection in Orlando, Florida, which has two heavy conflicting movements that operate near capacity. The results of this study showed that the CFI increased the capacity in 25%, reduced delay in 30-45 % (for the critical movements), and reduced queue length in 25-40 %.

After an extensive literature review, related to Continuous Flow Intersection, the following tables (1, 2, 3, 4, and 5) summarize the main objective, methodology, MOEs, research tools and results, for each literature:

Table 6. Main objective summary of previous literature

Study	***Main objective of study***
Speth (2008)	Evaluate the operational performance of CDI, SPUI, and DDI
Chlewicki (2003)	Develop new intersection and interchange designs
Bared et al. (2005)	Evaluate the performance of two unconventional designs (DXI and DDI)
Sharma and Chatterjee (2007)	Evaluate the effectiveness of new alternative solution DDI with CDI

Table 7. Summarized analysis methodology of previous literature

Study	***Analysis methodology of study***
Speth (2008)	Comparing the operation performance of three unconventional interchanges under four traffic volume scenarios
Chlewicki (2003)	Develop and test the operation of the proposed new designs compared to their original designs
Bared et al. (2005)	Comparing the performance of the new designs with their conventional design under different traffic volume scenarios
Sharma and Chatterjee (2007)	Comparing the operational performance and cost-benefit of the new design under different volume scenarios

Table 8. Measure of effectiveness of each previous literature

Study	***MOEs of study***
Speth (2008)	Average vehicle delay, number of vehicle served, average number of stops per vehicle, and total number of stops
Chlewicki (2003)	Total delay, stop delay, and total stops
Bared et al. (2005)	Capacity, average delay, stop time, and queue length
Sharma and Chatterjee (2007)	delay per vehicle, queue length, and capacity

Table 9. Summary of analysis tool of each research

Study	***Analysis tool of study***
Speth (2008)	Synchro 7, SimTraffic, and VISSIM 4.2
Chlewicki (2003)	Synchro 5.0, SimTraffic
Bared et al. (2005)	VISSIM microsimulation model and Synchro
Sharma and Chatterjee (2007)	Microscopic simulation tool - VISSIM 4.3

Table 10. Summarized result of previous literature

Study	***Results of study***
Speth (2008)	DDI outperformed the conventional design with all volume scenarios
Chlewicki (2003)	DDI is 3-times lower than CDI for total delay, 4-times less stop delay, 2-times lower total stops
Bared et al. (2005)	The DDI and CDI have identical performance under low and medium volume, but DDI designs offers higher capacity, lower average delay per vehicle, lesser number of stops, lower stop time, fewer conflict points, and shorter queue length with higher traffic flows
Sharma and Chatterjee (2007)	DDI performed better than the CDI in all traffic volume scenarios, with the largest different at the high flow levels, but the difference was identical at the low and medium flows for both alternatives and reduced the time cost and the vehicle operation cost experienced by the driver

Most literature was either about evaluating the operational and safety performance of the innovative designs or comparing the performance among more than two designs. There is also literature about summarizing information on innovative designs and how to evaluate them. The methodologies observed in these studies are almost the same in MOEs, analysis tools and studies' results. The most popular MOEs in literature are delays, capacity, number of stops, travel times, and vehicle delays. The VISSIM application was the most widely used software in previous studies due to its ability to imitate those innovative designs. All studies showed that DDI and CFI designs are outperforming their conventional designs and they hold great promise in enhancing the operational and safety performance at such intersections and interchanges.

However, no literature brings a study that develops thresholds to switch any intersection/interchange from their conventional designs to the new innovative designs. And, consequently, this dissertation is unique regarding DDI and CFI designs. The thresholds these studies found are going to help transportation engineers to make their decisions and consider the innovative designs over conventional designs. The following chapter explains the methodology that is going to be followed in this dissertation.

CHAPTER 3: METHODOLOGY

To reach the study's main goal, a clear methodology was built to guide the researcher to perform the analysis. The main procedure, in the analysis methodology, is shown in figure 1 below. The first step - to start the analysis - is to understand the innovative designs. This was done through previous literature and report (on these designs) review and identification of the most significant parameters that may affect the designs' operation such as lane configurations, geometric features, traffic volumes and signal phasing and timing (Autey et al., 2013; Cheong et al., 2008). There are several new intersection/interchange designs implemented in the United States, but there are not many published studies based on field data related to such innovative designs. Accordingly, there is a need to identify several candidate-locations that have CFI and DDI designs within the U.S. and the required data that need to be collected. The candidate-locations and data-collection are explained in detail. After understanding the innovative designs and collect field data, it is necessary to select the appropriate tools that can run detailed analyses at microscopic level [25]. There are many traffic micro-simulation models, broadly used by many professionals and researchers, such as VISSIM, CORISM, AMISUN, SIMTRAFFIC, PARAMICS, INTEGRATION, among others. Nonetheless, not all of them are able to perfectly simulate the innovative designs. To choose the appropriate micro-simulation package, it is necessary to considered factors such as ability to imitate the innovative designs, simulate signal control plans and/or import signal plans from other tools, and the capability to easily run simulations for different replications and random seeds and other factors (El Esawey and Sayed, 2013). There are many simulation parameters that need to be considered such as the number of replications, simulation period, seeding number and driving parameters, which are going to be dealt with during calibration and validation steps, in the simulation models

(Schroeder et al., 2014; Manjunatha et al., 2013; Lownes and Machemehl, 2006). To minimize error between model input and output, when calibrating and validating the models, it will be necessary to optimize signal timing plans and perform a sensitivity analysis regarding driving-behavior parameters (Hu et al., 2014; Chu et al., 2003).

When crucial parameters, related to the designs, are identified and models are calibrated and validated, the two designs can be compared to each other by means of specific measures of effectiveness (MOEs) that have been broadly used on previous literature such as average delay, travel time, queue length, and capacity (maximum throughput of movements). Those MOEs for these innovative designs are affected by significant factors such as traffic volumes, geometric designs, lane configurations and traffic signal plans, all necessary to design an experiment that considers all these parameters. So, the next two logical steps, after the calibration and validation, are designing the experiment and identifying the proper MOEs. The last two steps will allow the comparison between the two designs' operational performance and will establish the switching point (warrants) at which the design's operation becomes more efficient to convert the conventional designs into unconventional designs.

Hence, the research investigates two designs and each design has its own parameters to be included on the experiment, it will be necessary to design two separate experiments, one for DDI and other one for CFI. These experiments will have some similar parameters - such as the number of lanes, volume levels - but different geometric parameters and signal plans, which will lead to different number of runs and scenarios for each experiment. Traffic volume, to be used in the experiments, is the number of vehicles per hour, per lane and levels that are different in each experiment. Each experiment and related parameters are going to be explained in detail in the respective design's chapter.

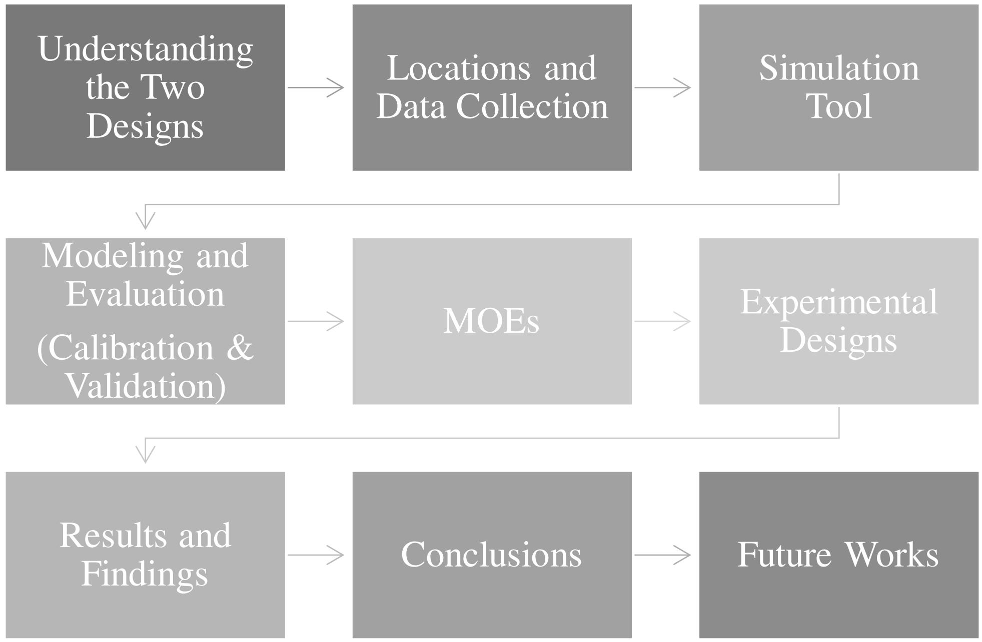


Figure 1 Dissertation Methodology Procedure

The previous methodology was followed to achieve the research's goal. Detailed information on DDI and CFI designs will be explained on the following section, where the fundamental concept for the two designs will be explained.

3.1 Understanding the Diverging Diamond Interchange (DDI)

The design is mainly The Diverging Diamond Interchange design, also known as double crossover diamond (DCD), which was introduced by Chlewicki in his paper "New Interchange and Intersection Designs: The Synchronized Split-Phasing Intersection and the Diverging Diamond Interchange, 2003" and the first DDI in the United States was built in Springfield, Missouri, 2009 (UDOT, 2016). The idea behind the DDI is to use the crossing-over movement in an interchange design, developed from the concept of the synchronized split phasing design, in order to better accommodate left-turn movements and potentially eliminate one phase in the signal cycle (Chlewicki, 2003). To get there and reach DDI's goal, the 'through' and 'left-turn' traffic movements, on the right side of the road, are shifted to the left side - prior the interchange - by intersecting the road mainline by the protected phase (see figure 2). This shifting makes vehicles, at the crossroad, making left turns onto or off ramps, not conflict with vehicles approaching from another direction (Schroeder et al., 201). The highway portion does not change but the left-turn movement 'off ramps' is changed. DDI design allows to operate the interchange's 'through' and 'left-turn' movements easily with two simple signal phases and process the traffic flow very efficiently, especially for interchanges with high left-turn demand volume to and from the highway. Moreover, it improves safety by reducing the number of conflict-points (see figure 3 and 4), which makes DDI a popular option and a cost-benefit treatment to replace the over-loaded unconventional interchanges (Abou-Senna et al., 2015; Schroeder et al., 2014).

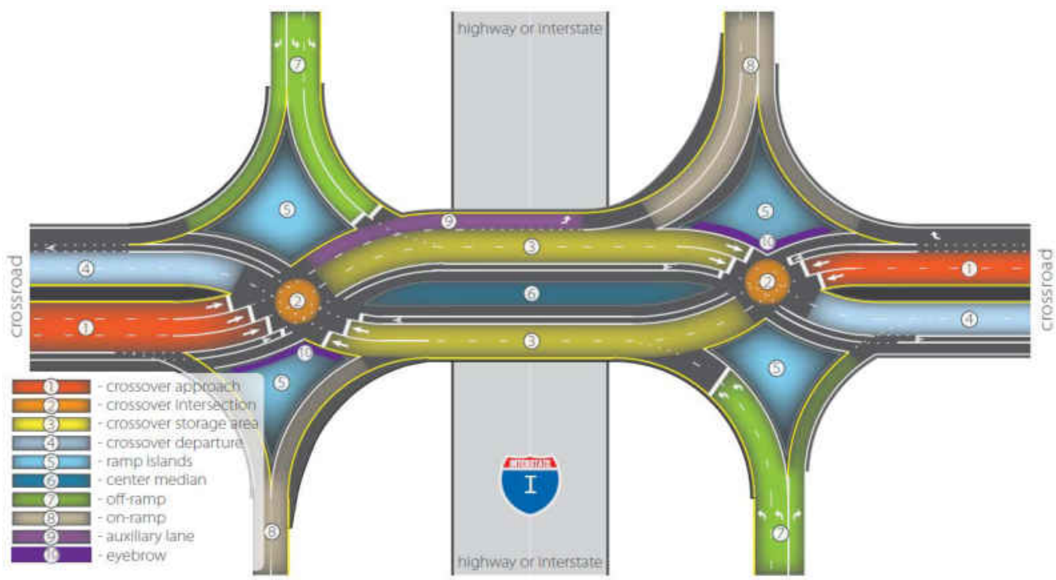


Figure 2 Layout of Diverging Diamond Interchange (DDI) (UDOT DDI Guidelines, 2014)

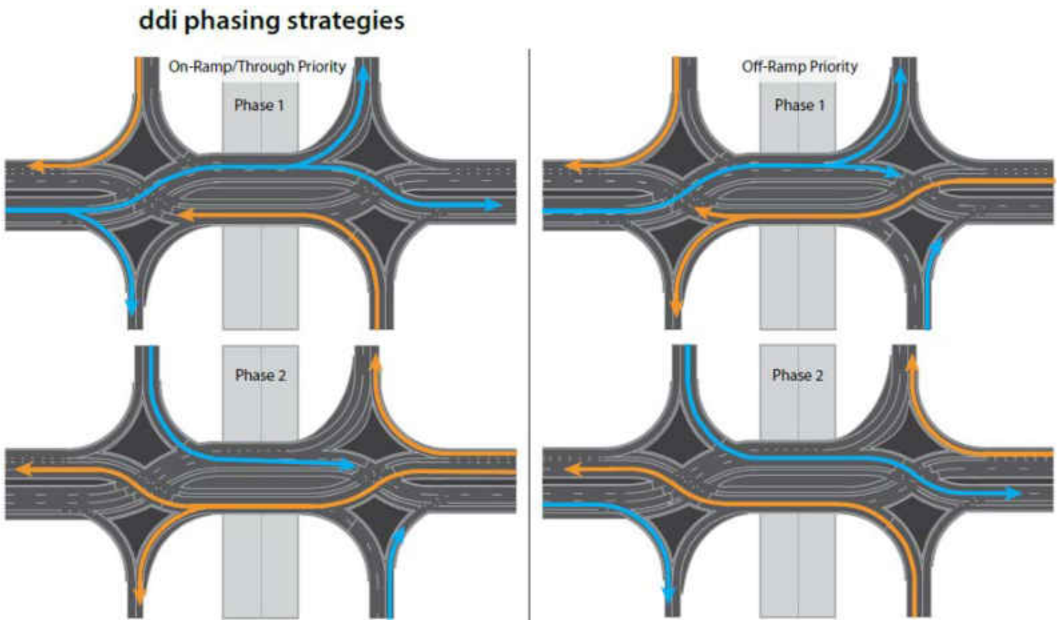


Figure 3 the DDI Signal Phasing Schemes (UDOT DDI Guidelines, 2014)

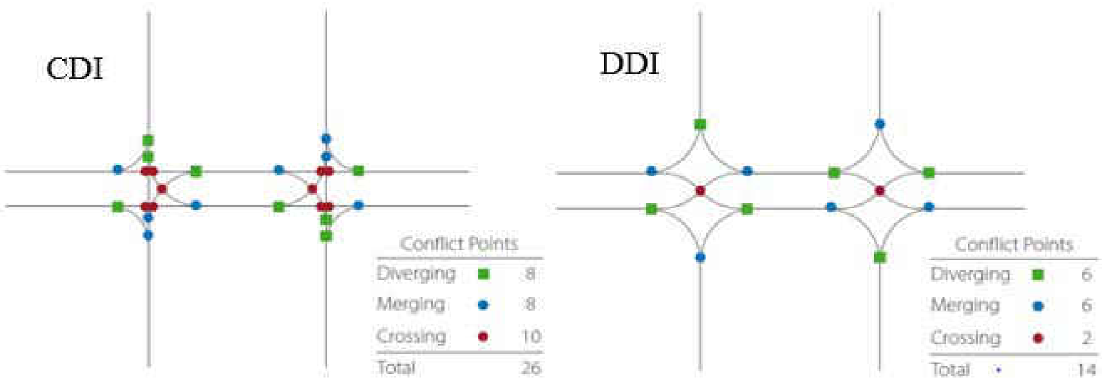


Figure 4 DDI Conflict Points Compared to CDI (UDOT DDI Guidelines, 2014)

There are many potential benefits that can be reached by implementing DDI and these advantages, relate to operational and safety perspectives, are:

- Increased capacity;
- Two phase signals with short-time cycle lengths;
- Substantial reduction of conflict points;
- Reduced construction time;

- Cost effective;
- Fewer collision and reduced collision severity;
- Improved pedestrian safety;
- Minimization of right-of-way impacts.

There is no system or design that has no disadvantages, but they can be acceptable if compared to benefits obtained from such innovative design, some disadvantages are:

- Driver Confusion;
- Problematic for high-speed arterial;
- Operational issues with closed space intersections;
- Pedestrian may require two-stage crossing.

In conclusion, DDI design's fundamental concept (1) reduces the number of conflicts between 'left-turn' and 'through' movements - shifting one or more movements to the opposite side of the road; (2) reduces the number of phases from four or three phases to three or two phases (Cheong et al., 2008; Abou-Senna et al., 2015; UDOT, 2016).

3.2 Understanding the Continuous Flow Intersection (CFI)

The first continuous flow intersection in the United States, with ramps in a single quadrant at a T-intersection, was opened in 1994 on Long Island, New York, at an entrance to Dowling College (Chlewicki, 2003). The main idea of the continuous flow intersections, also known as the Crossover Displaced Left turn (XDL) and Displaced Left turn (DLT)(Dhatrek et al., 2010; Steyn et al., 2014), is to shift the left-turn lanes from the main intersection to a left-turn bay that placed to the left side of the road by crossing the oncoming through lanes during a protected phase. This shifting is accomplished by adding a signal controller and mid-block intersection on the approach around 300 feet or so feet upstream of the main intersection (see figure 5 & 6) (UDOT, 2013; Cheong et al., 2008; Schroeder et al., 2014). Three phases intersection will be operated if one set of paired sub-intersections is implemented. If the CFI was implemented with 4 sub-intersections ahead of the primary intersection, the intersection will be operated with 2 signal phases which reduces the conflicts between the movements (see figure 7 and 8), improves the intersection capacity, and reduces the delay (UDOT, 2013).

Figure 5 Layout of 2-Leg Continuous Flow Intersection (CFI) (UDOT CFI Guidelines, 2013)

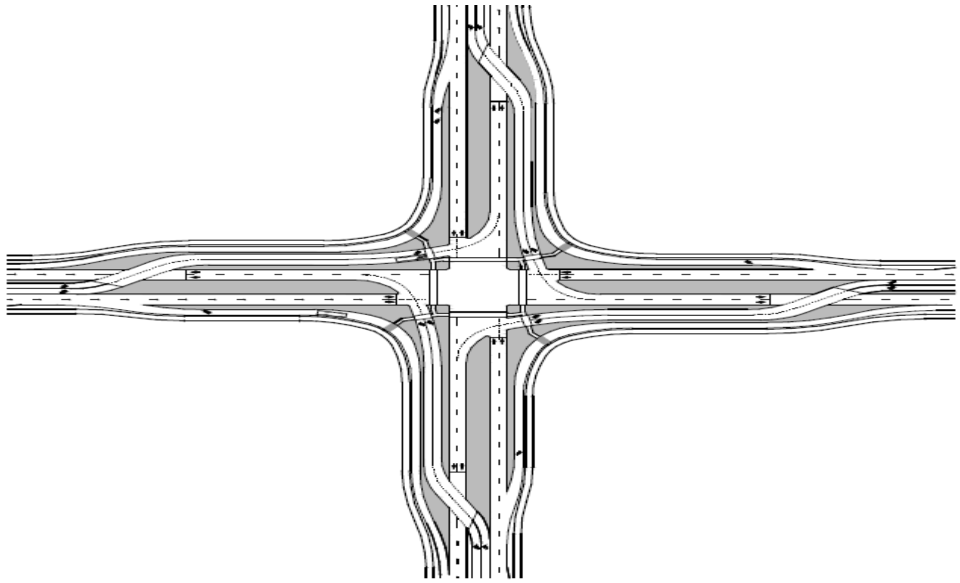


Figure 6 Layout of 4-Leg Continuous Flow Intersection (CFI) (FHWA-SA-14-068, 2014)

The ‘through’ and ‘left-turn’ movements, at the main intersection are allowed to operate simultaneously without conflict with the oncoming traffic by using two-phase signal (Autey et al., 2013). A channelized right-turn lane allows the right-turn traffic to bypass the main intersection and merge into the mainstream traffic, which will allow the through, left-turn, and right-turn movements to be served simultaneously without any potential conflict at the main intersection. The additional green time, reduced delay and reduced conflicts can potentially improve the capacity of an intersection between 30% and 70%, as identified in operational and observational studies performed by UDOT (UDOT, 2013; Abou-Senna and Radwan, 2016). The results of CFI implementation will improve traffic operations and safety performance. More bicycles and pedestrians through any CFI as well as through a conventional intersection (see figure 9)

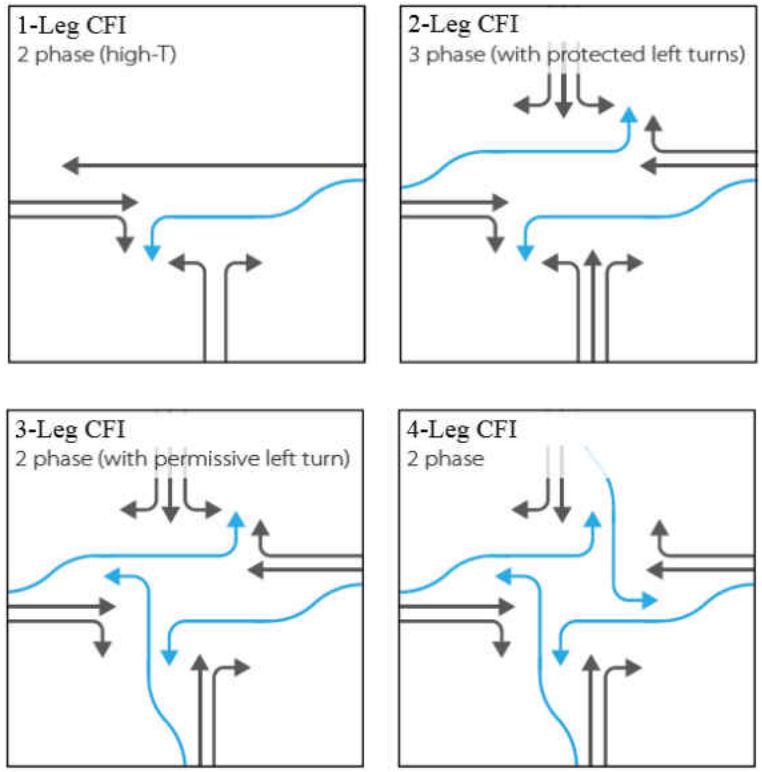


Figure 7 the CFI Signal Phasing Schemes (UDOT CFI Guidelines, 2013)

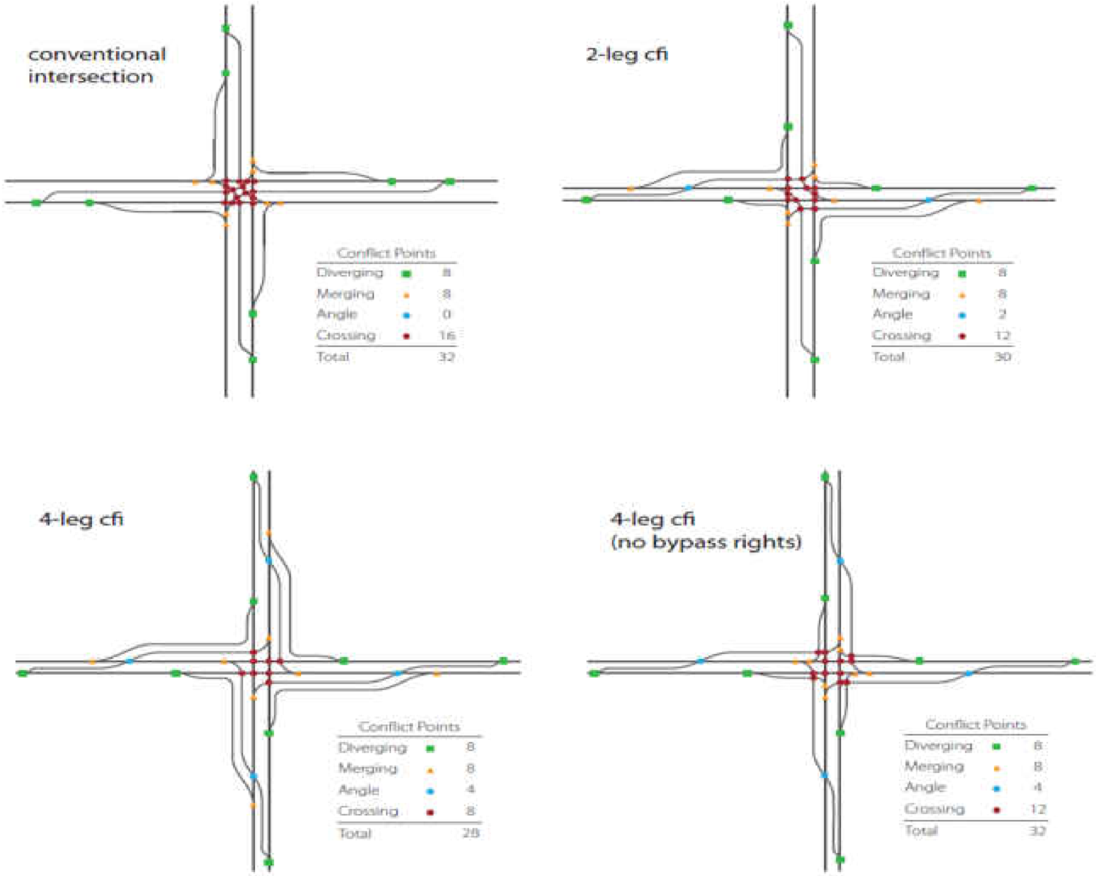


Figure 8 Conventional Intersection Conflict Points compared to 2 & 4 legs CFI (UDOT CFI Guidelines, 2013)

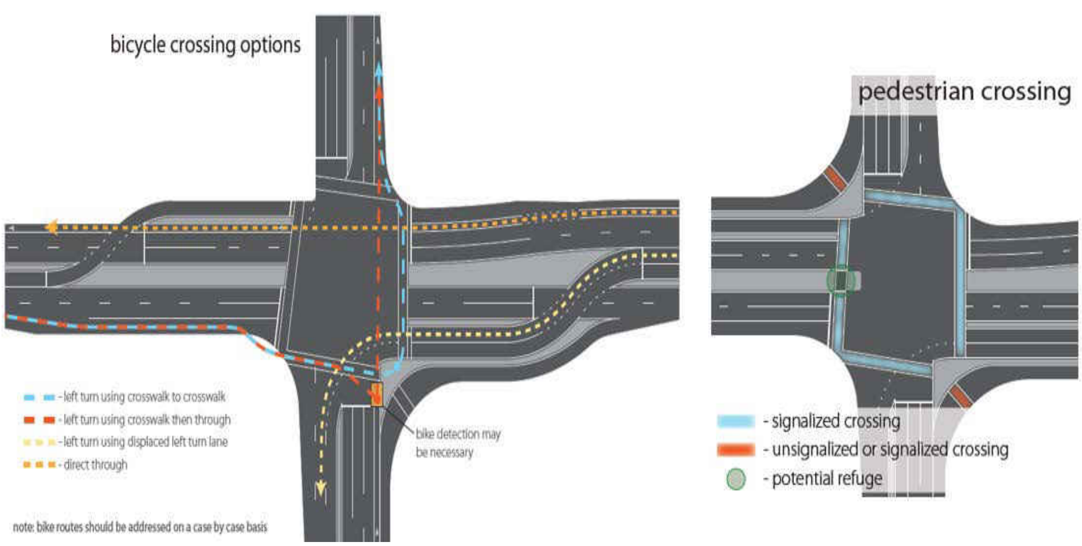


Figure 9 CFI Bicycle and Pedestrian Crossing Options (UDOT CFI Guidelines, 2013)

There are many benefits and disadvantages that earned by implementing the CFI design summarized in some points:

Advantages:

- Reduced delay and travel time for all the movements;
- Reduced number of stops for through arterial traffic;
- Increased capacity
- Lower cost than some alternatives
- Better progression for all movements
- Improved the intersection safety performance

Disadvantages

- Confusion for the driver and pedestrian;
- Prohibited U-turn possibilities;
- Pedestrian cross the intersection in two or more stages;
- Additional right-of-way;
- Lack of access control
- More cost than some alternatives

When to consider implementing the CFI?

There are two considerations that were agreed upon by most literatures regarding the construction of the CFI on an arterial road. The first consideration is when the volume demand is at or over the intersection capacity, and the second is when there is additional right-of-way available along the arterial road near the intersection (Kim et al., 2007; Steyn et al., 2014; Hummer and Reid, 2000; Toledo et al., 2003).

CHAPTER 4: DIVERGING DIAMOND INTERCHANGE

The previous methodology was followed to achieve the research's goal. Detailed information about the DDI design was explained on the previous section to understand DDI's fundamental concept.

4.1 Candidate Locations and Data Collections

The DDI concept is new and there are no innovative interchange designs implemented in Florida. Thus, candidate-locations are located outside the state. However, the Florida Department of Transportation (FDOT) is planning 26 DDIs, which will be in place by 2030, in the State of Florida. There are several DDI design locations all over the U.S., but there is not sufficient data available on such locations or access to data is not easy. So, finding locations, with DDI already in place and getting data (collected for the implemented designs by agencies or authorities, interested in sharing the data) was a huge challenge. Nevertheless, the Federal Highway Administration (FHWA) represented by Dr. David Yang and Dr. Wei Zhang, proposed four locations:

(1) I-285 & Ashford Dunwoody RD, Atlanta, GA (DDI-Implemented)

(2) I-85 and Pleasant Hill, Atlanta, GA. (DDI-Implemented)

(3) I-66 and Hwy 15, Haymarket, VA (DDI-Under-construction)

(4) I-75 and University Pkwy, Sarasota, FL (DDI-Under-construction)

Also, only the first two DDI designs were implemented and the collected data for these constructed locations were shared in a detailed report – mostly assignment lines, origins and destinations (O-D) matrices, turning movement counts, average travel times, average speeds and calculated delay.

4.1.1 Ashford Dunwoody RD and I-285, Atlanta, GA

The first DDI in Georgia is the first candidate-location (Ashford Dunwoody RD and I-285) in this study. Located in Atlanta, it was opened on June 3, 2012. Figure 10 shows the DDI lane configuration with four lanes in each direction, two through, one left and one dedicated on-ramp right turn-lane before the crossover. The two off-ramps consist of two left-turn lanes and two right-turn lanes. One left-turn and one right turn-lane led to the on-ramp. The southbound traffic allows a right turn movement to the ramp before the crossover. This right turn ramp will merge with the left turn movement, from the northbound direction to provide one ramp to the westbound direction. After the crossover for the southbound movement, traffic will come in from the westbound direction that wants to head southward. This traffic will come from the left side of the southbound traffic. An exit will then be provided on the left for left turn movements to the eastbound movement after the highway passes under the I-285.

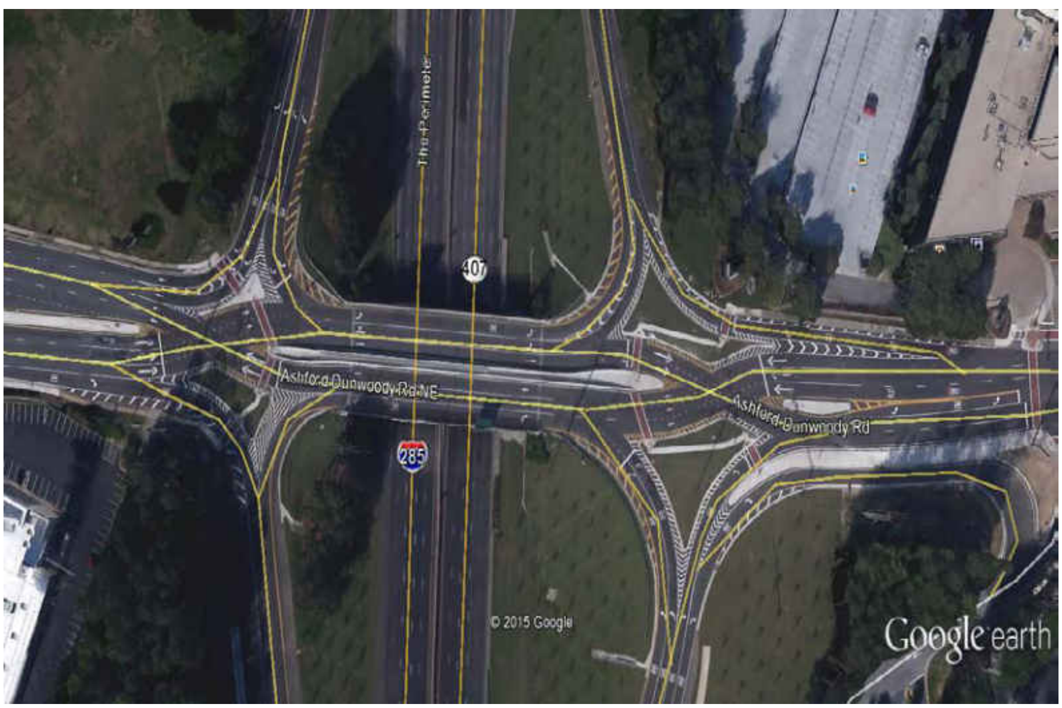


Figure 10 Lane Configuration of I-285 & Ashford Dunwoody RD, Atlanta, GA (Source: Google Earth 2015)

The crossover will then occur again to get the southbound traffic to the right side and finally receive the traffic from the eastbound right-turn movement. The DDI design is symmetrical for the northbound traffic. Two signal lights are needed for the left-turn crossover, one at each crossover (see figure 11), but usually the conventional diamond interchange is operated using a three-phase signal control. These signals are two-phase signals, with each phase dedicated to the alternative opposing movements. The ramp phase will be combined with the non-conflicting flow of traffic for the south/north road. There are two signals for the right and left turn movements, for each off-ramp.

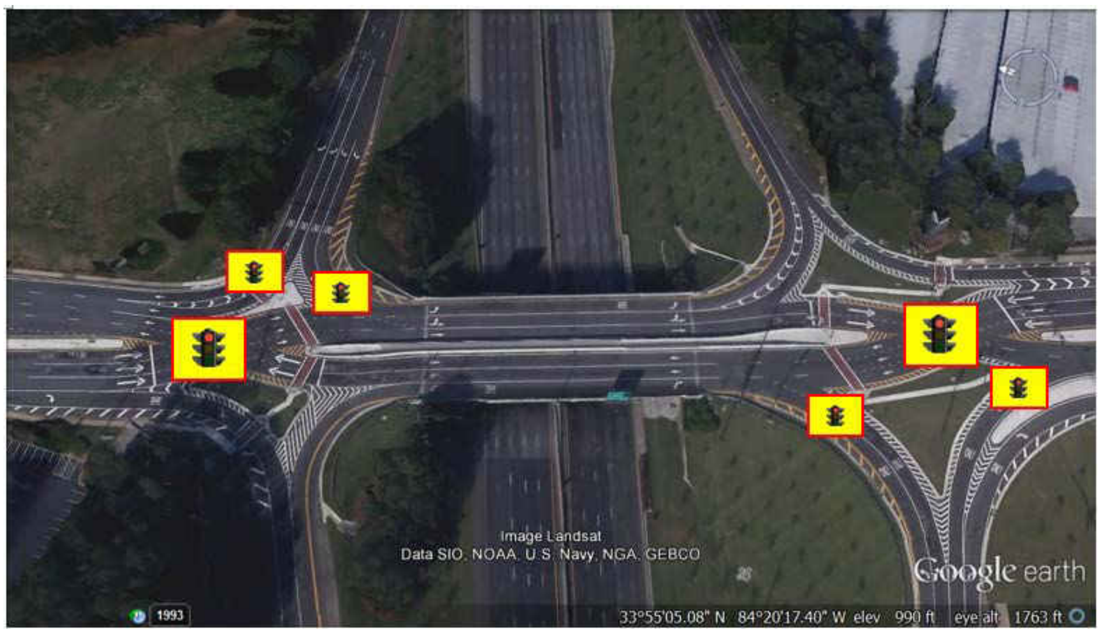


Figure 11 Traffic Signals of I-285 & Ashford Dunwoody RD, Atlanta, GA (Source: Google Earth 2016)

The provided data for this location are assignment lines, origins and destinations (O-D) matrices, turning movement counts, average travel times, average speeds and calculated delay. The assignment lines and origins/destinations are fully described as well as the way the data was collected, not to mention the starting point for each origin and ending point for each destination (see figure 12 and 13). Allocated volumes (vehicles) based on O-D percentages and average travel times (in seconds) by O-D pair are collected for this location. A sample of the volumes and average travel times is shown in figure 14 and 15. In addition, turning movement counts (TMC) for the AM/PM periods and their locations are provided for each movement and direction, for several locations along the interchange corridor (see figure 16). Also, calculated travel times, speeds and delays were provided for the AM/PM periods.

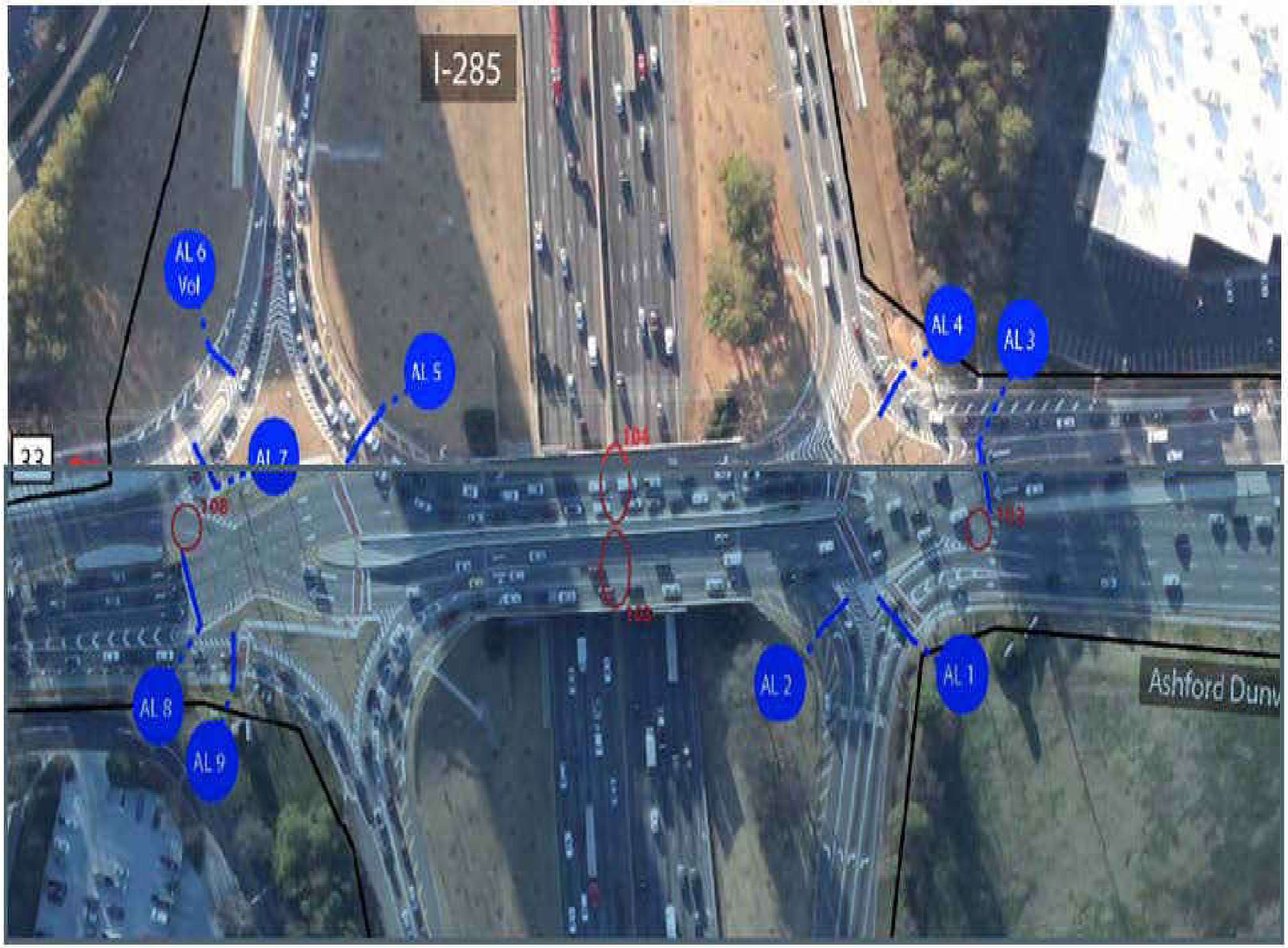


Figure 12 Assignment Lines and Central Route Markers, (FHWA, 2015)

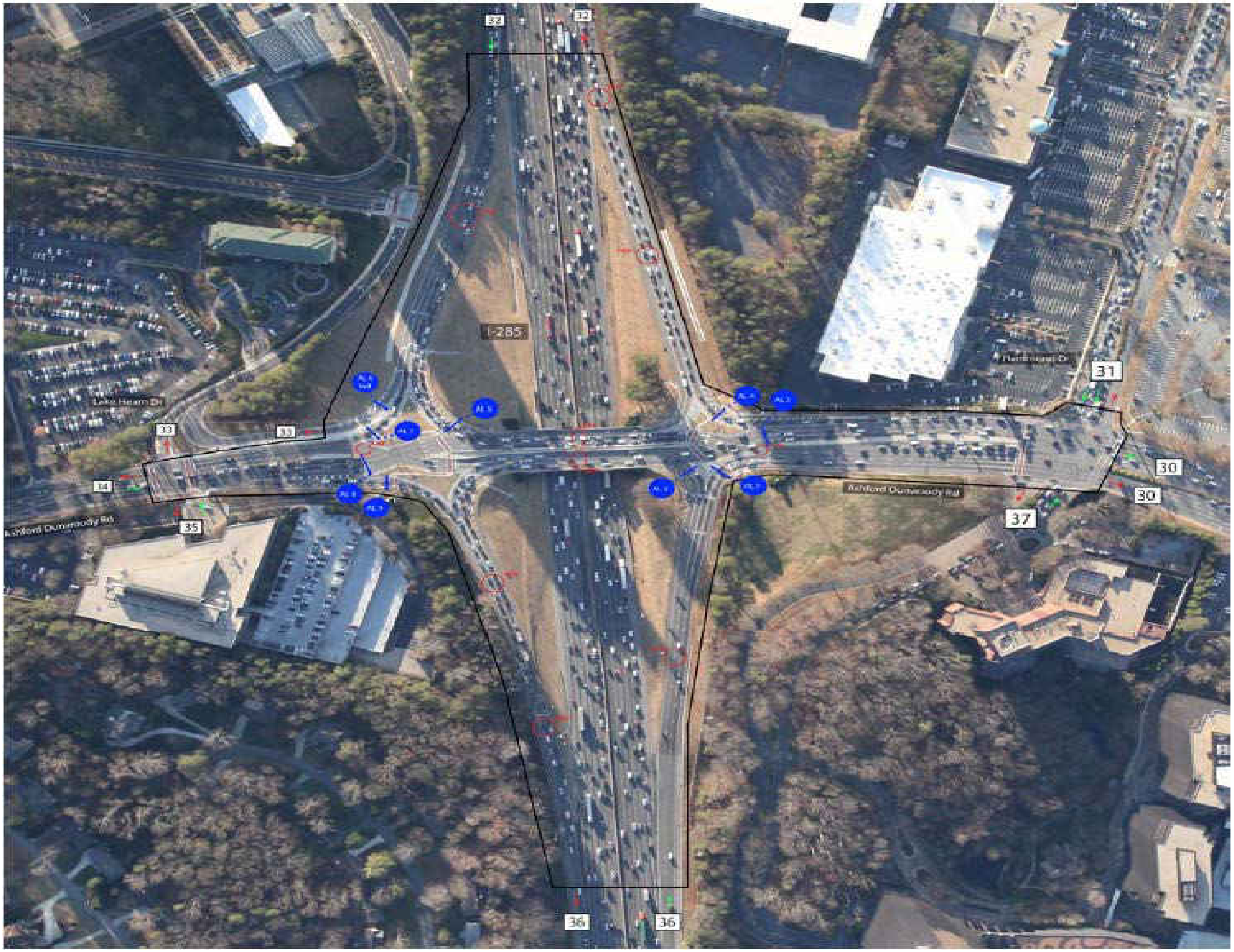


Figure 13 Overlay codes applied to survey area for O-D tracing, (FHWA, 2015)

FHWA Site 1
Atlanta, GA: I-285 at Ashford-Dunwoody Rd
AM Survey Period 7:45 - 8:45 AM

TABLE AM3 (20%): Allocated Volumes Based on O-D Percentages 20 Percent Sample

Assignment Line	Origin	Counted Volume	DESTINATIONS 30	31	32	33	34	35	36	37
AL1	36	1727	1143	425	0	0	0	0	0	159
AL2	36	490	0	0	0	362	91	27	11	0
AL3	30	612	0	0	0	74	161	9	368	0
AL3	31	106	0	0	0	0	18	0	87	0
AL3	32	5	0	0	0	0	0	0	5	0
AL3	37	18	5	0	0	0	9	0	5	0
AL3 Total		741	5	0	0	74	189	9	465	0
AL4	30	590	0	0	590	0	0	0	0	0
AL4	31	51	0	0	51	0	0	0	0	0
AL4	37	41	0	0	41	0	0	0	0	0
AL4 Total		682	0	0	682	0	0	0	0	0
AL5	32	1523	1083	250	20	0	0	0	0	169
AL6	32	598	0	0	0	598	0	0	0	0
AL7	32	344	0	0	0	5	296	42	0	0
AL8	32	5	0	0	0	0	0	0	0	5
AL8	34	747	284	41	394	0	0	0	0	27
AL8	35	5	5	0	0	0	0	0	0	0
AL8 Total		756	289	41	394	0	0	0	0	32
AL9	34	118	0	0	0	0	0	0	118	0
		Summed total	2520	717	1096	441	576	78	594	360
		Applicable TMC count	2495	695	1130	440	592	86	591	406
		GEH	**0.5**	**0.8**	**1.0**	**0.1**	**0.7**	**0.9**	**0.1**	**2.4**

Blue Cells indicate calculated volumes based on the sum of the relevant ramps.

NOTE: Table 3 provides a way to check the internal consistency (quality) of the O-D findings. As described above, Table 3 provides the expected volumes for each O-D pair. These volumes can be summed by column (destination) to provide the expected total hourly volume at each destination (not of all vehicles at that destination, but just of vehicles that passed through the interchange across an AL). If the O-D table is accurate, these estimates will be close to turning movement counts acquired at the destinations (for movements of traffic that earlier had passed through the interchange). Both the volume estimates and counts have been provided on separate lines at the bottom of each Table 3. The GEH test of closeness was then used to compare the volumes; normally a GEH value of less than 5.0 indicates a close match (green); a GEH value between 5 and 10 indicates a match of marginal acceptability (yellow, if applicable); and a GEH value greater than 10 indicates that major error is probably present (red, if applicable). A quality standard published by the Wisconsin DOT applicable to microsimulation modeling is that 75% (or 85%, depending on definition used) of GEH scores should be less than 5.0 (green), and none greater than 10 (no reds) (see "GEH" section at http://www.wisdot.info/microsimulation/index.php?title=Model_Calibration).

Figure 14 An (AM) Sample of Allocated Volumes Based on O-D percentages, (FHWA, 2015)

FHWA Site 1
Atlanta, GA: I-285 at Ashford-Dunwoody Rd
AM Survey Period 7:45 - 8:45 AM

TABLE AM4 (20%): Average Travel Times (in seconds) by O-D Pair 20 Percent Sample

			DESTINATIONS							
Assignment Line	Origin		30	31	32	33	34	35	36	37
AL1	36		117	145						99
AL2	36					77	62	79	58	
AL3	30					85	73	124	73	
AL3	31						119		95	
AL3	32								210	
AL3	37		196				151		83	
AL3 Total										
AL4	30				99					
AL4	31				104					
AL4	37				153					
AL4 Total										
AL5	32		122	146	158					140
AL6	32									
AL7	32					67	52	56		
AL8	32									187
AL8	34		98	139	119					94
AL8	35		73							
AL8 Total										
AL9	34								39	

Figure 15 An (AM) Sample of Average Travel Times (in seconds) by O-D Pair, (FHWA, 2015)

Turning Movement Counts Summary Graphic (AM/PM)
Atlanta, GA
Site 1: Ashford-Dunwoody Rd at I-285

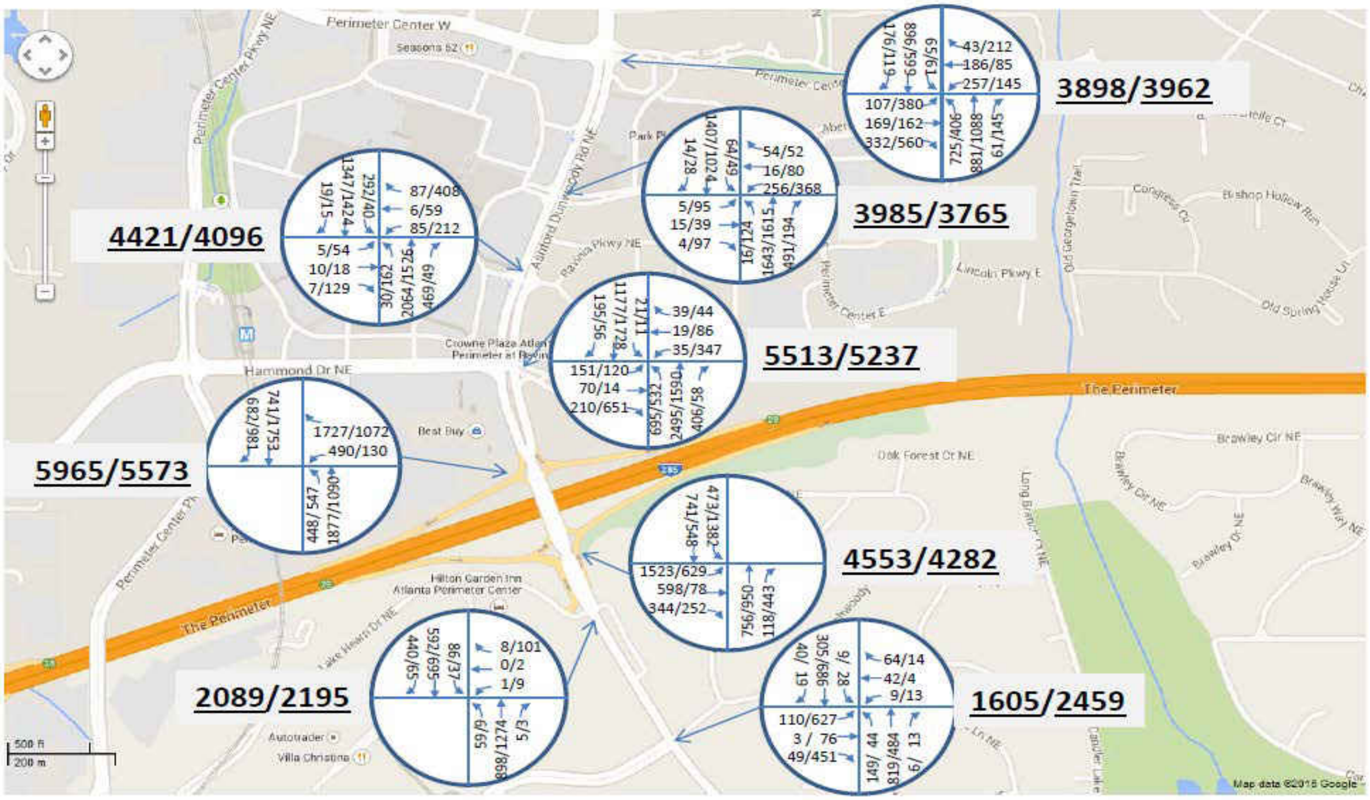


Figure 16 Turning Movement Counts Summary and Their Locations (AM/PM), (FHWA, 2015)

4.1.2 Pleasant Hill Rd and I-85, Atlanta, GA

The second DDI in Georgia and the first operated DDI in Gwinnett County, Atlanta (I-85 and Pleasant Hill) is the second DDI candidate-location in this study, which is one of only a dozen that have been constructed in the U.S.

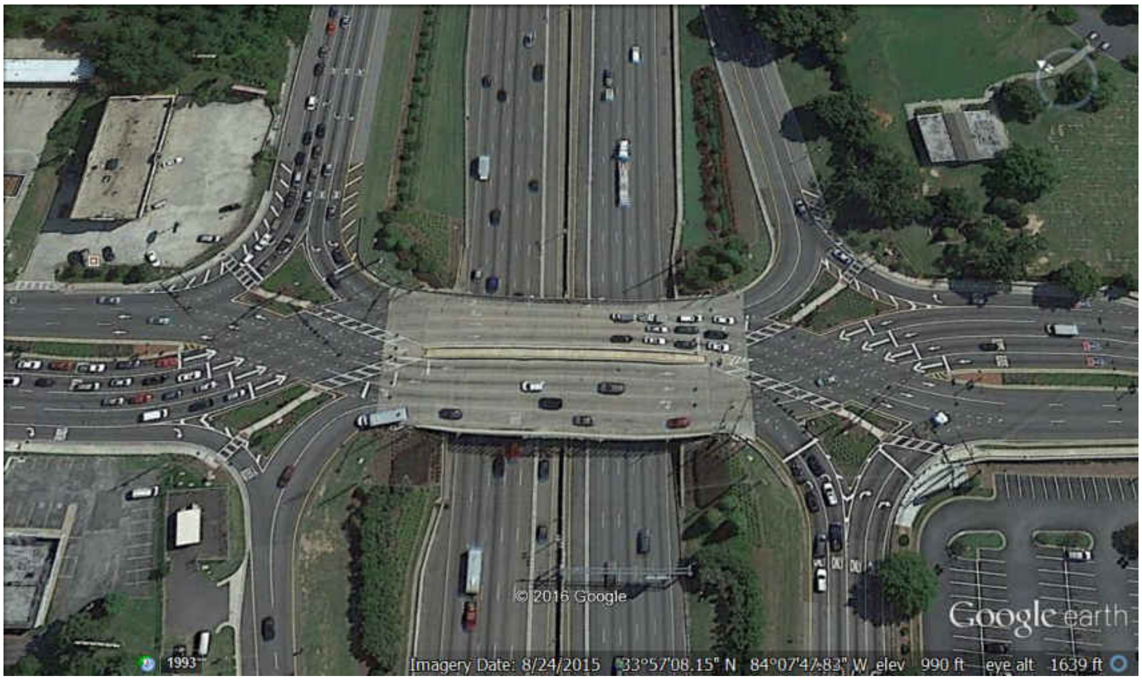


Figure 17 Lane Configuration of Pleasant Hill Rd & I-85, Atlanta, GA (Source: Google Earth 2016)

Figure 17 shows the Pleasant Hill Road Diverging Diamond Interchange (DDI) lane configuration which has five-lanes in each direction, two through lanes, one left-turn lane, one shared left and through lane and one dedicated on-ramp right turn lane before the crossover. The two off-ramps consist of four lanes, two for the left-turn traffics and two for the right-turn traffics. For the on-ramp, there are one right-turn lane and two left-turn lanes that led to the on-ramp. The eastbound traffic allows a right turn movement to the on-ramp before the crossover. The right turn ramp will merge with the two left-turn movements from the westbound direction to provide one ramp to the southbound direction. After the crossover for the eastbound movement, the traffic that will come from the southbound direction will merge with the crossing traffic that comes from the eastbound and heads southward. An exit is provided on the left side after the highway passes above the I-85 for the traffic that wants to go to the northbound direction. The crossover will, then, occur again to take the eastbound traffic to the right side and finally receive the traffic from the northbound right-turn movement. The design is symmetrical for the westbound traffic. This design requires two traffic signals only for the left-turn crossover, one at each cross over (see figure 18). These signals are usually operated by two phases and each phase is dedicated to the alternative opposing movements. The phase for the ramp will be combined with the non-conflicting traffic flow to the east/west road. There are two signals, one to the right and one to the left-turn movement, for each off-ramp (see figure 18).

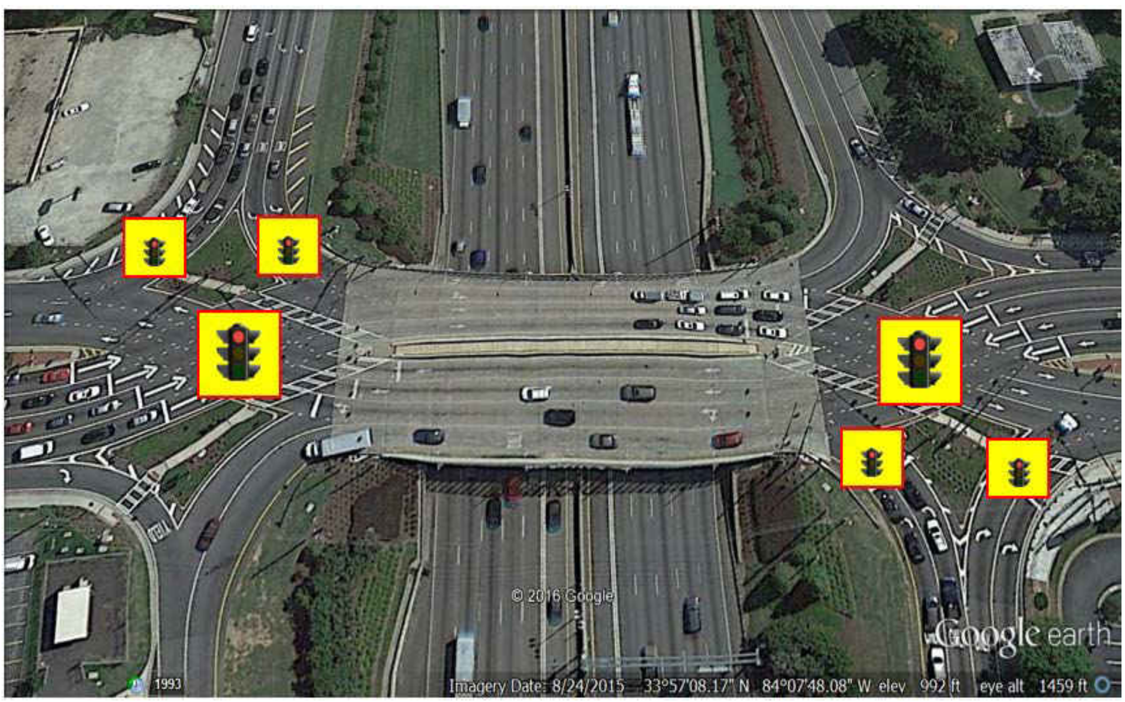


Figure 18 Traffic Signals of Pleasant Hill Rd & I-85, Atlanta, GA (Source: Google Earth 2016)

The data for this location was shared by the Federal Highway Administration (FHWA) on May, 17, 2016. This data is a long and detailed report done by Skycomp to conduct time-lapse aerial photographic (TLAP) surveys of the diverging diamond interchange at Pleasant Hill Rd and I-85 in Atlanta, GA. Surveys were collected by flights on the 1st of April 2015, from approximately 7:30 am to 9:00 am, and from 5: 00 pm to 6:30 pm. The survey resulted in different data as follows:

- Origin-destination matrices with travel times;
- Turning movement counts (TMC);
- Queue length profiles;
- Vehicle speed profiles.

Samples of FHWA shared data are shown in the following figures. Figure 19 shows a sample of the shared O-D matrices data. Manual turning movement counts were provided in the report for 15-minute periods and as counted, the vehicles were classified into one of the following categories: tractor-trailer, single-unit truck, bus or auto/other (see figures 20 and 21).

FHWA Site 2
Pleasant Hill Rd at I-85
Atlanta, GA **04-01-2015, 7:40 AM - 8:40 AM**

TABLE 3: Allocated Volumes Based on O-D Percentages — 10 Percent Sample

Assignment Line	Origin	Volumes	DESTINATIONS 30	31	33	37	39	40	42	43	44	46	48	50	51	52
AL1	46	1490	1068	36	341	9	0	0	0	0	0	0	9	9	9	9
AL2	46	692	0	0	0	0	28	0	303	341	9	9	0	0	0	0
AL3	30	873	0	0	0	0	0	0	57	332	19	465	0	0	0	0
	31	19	0	0	0	0	0	0	9	0	0	9	0	0	0	0
	32	19	0	0	0	0	0	0	0	0	0	19	0	0	0	0
	33	142	0	0	0	0	0	0	0	0	0	142	0	0	0	0
	34	19	0	0	0	0	0	0	0	9	9	0	0	0	0	0
	50*	9	0	0	0	0	0	0	9	0	0	0	0	0	0	0
	52	47	0	0	0	0	0	0	0	19	0	28	0	0	0	0
AL3 Total		1129	0	0	0	0	0	0	76	361	28	664	0	0	0	0
AL4	30	109	0	0	0	109	0	0	0	0	0	0	0	0	0	0
	32	18	0	0	0	18	0	0	0	0	0	0	0	0	0	0
	33	9	0	0	0	9	0	0	0	0	0	0	0	0	0	0
	35	9	0	0	0	9	0	0	0	0	0	0	0	0	0	0
	46*	27	0	0	0	18	0	0	0	9	0	0	0	0	0	0
AL4 Total		173	0	0	0	164	0	0	0	9	0	0	0	0	0	0
AL5	37	483	357	0	19	0	0	0	0	0	0	0	10	77	0	19
AL6	37	400	0	0	0	0	0	10	20	234	137	0	0	0	0	0
AL7	30*	9	0	0	0	0	0	0	0	0	0	0	9	0	0	0
	42	18	9	0	0	9	0	0	0	0	0	0	0	0	0	0
	43	1252	925	9	45	236	0	0	0	0	0	0	0	27	0	9
	44	200	64	0	9	118	0	0	0	0	0	0	0	0	0	9
	46*	9	0	0	0	0	0	0	0	0	0	0	9	0	0	0
AL7 Total		1489	999	9	54	363	0	0	0	0	0	0	18	27	0	18
AL8	30*	9	0	0	0	0	0	0	0	0	0	9	0	0	0	0
	42	70	0	0	0	0	0	0	0	0	0	70	0	0	0	0
	43	565	0	0	0	0	0	0	0	0	0	565	0	0	0	0
	44	35	0	0	0	0	0	0	0	0	0	35	0	0	0	0
AL8 Total		678	0	0	0	0	0	0	0	0	0	678	0	0	0	0
		Summed total	2424	45	415	536	28	10	399	945	175	1352	37	113	9	46
		Counted total	2351	32	510	559			333	966	215	1316		98		73
		GEH	**1.5**	**2.1**	**4.4**	**1.0**			**3.4**	**0.7**	**2.9**	**1.0**		**1.5**		**3.4**

NOTE: Table 3 provides a way to check the internal consistency (quality) of the O-D findings. As described above, Table 3 provides the expected volumes for each O-D pair. These volumes can be summed by column (destination) to provide the expected total hourly volume at each destination (not of all vehicles at that destination, but just of vehicles that passed through the interchange across an AL). If the O-D table is accurate, these estimates will be close to turning movement counts acquired at the destinations (for movements of traffic that earlier had passed through the interchange). Both the volume estimates and counts have been provided on separate lines at the bottom of each Table 3. The GEH test of closeness was then used to compare the volumes; normally a GEH value of less than 5.0 indicates a close match (green); a GEH value between 5 and 10 indicates a match of marginal acceptability (yellow, if applicable); and a GEH value greater than 10 indicates that major error is probably present (red, if applicable). A quality standard published by the Wisconsin DOT applicable to microsimulation modeling is that 75% (or 85%, depending on definition used) of GEH scores should be less than 5.0 (green), and none greater than 10 (no reds) (see "GEH" section at http://www.wisdot.info/microsimulation/index.php?title=Model_Calibration).

Figure 19 A Sample of the O-D Matrices Data for the DDI at Pleasant Hill Rd & I-85, Atlanta, GA (FHWA- Reduced Data, 2015)

Figure 20 A Sample of Turning Movement Counts Summary and Their Locations for the DDI at Pleasant Hill Rd & I-85, Atlanta, GA (AM/PM), (FHWA- Reduced Data, 2015)

Project:	FHWA Site 2 (Pleasant Hill Rd / I-85)
Assignment:	Intersection 1
Location:	Pleasant Hill Rd at Shackleford Rd / Breckinridge Blvd
Date / Time:	April 1, 2015 / 7:30-9:00 a.m.

Note: The morning survey began at 7:38 a.m.; counts were extrapolated for this 15-minute time period.

TURNING MOVEMENT COUNT SUMMARY

TIME PERIOD	VEHICLE CLASS.	Northbound			Southbound				Eastbound			Westbound		
		NB-L	NB-T	NB-R	SB-L	SB-T	SB-U	SB-R	EB-L	EB-T	EB-R	WB-L	WB-T	WB-R
7:30 AM	Total Vehicles	64.3	589.3	19.3	49.3	231.4	6.4	109.3	19.3	23.6	10.7	10.7	186.4	55.7
	Passenger Car	62.1	585.0	19.3	47.1	227.1	6.4	107.1	19.3	23.6	10.7	10.7	184.3	55.7
	Truck	2.1	2.1	0.0	2.1	2.1	0.0	2.1	0.0	0.0	0.0	0.0	2.1	0.0
	Tractor-Trailer	0.0	0.0	0.0	0.0	2.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
7:45 AM	Bus	0.0	2.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
7:45 AM	Total Vehicles	43	541	28	47	256	2	88	29	48	13	8	160	56
	Passenger Car	43	535	28	47	248	2	86	29	45	10	8	155	56
	Truck	0	5	0	0	5	0	0	0	2	3	0	3	0
	Tractor-Trailer	0	1	0	0	0	0	2	0	0	0	0	2	0
8:00 AM	Bus	0	0	0	0	3	0	0	0	1	0	0	0	0
8:00 AM	Total Vehicles	34	505	22	49	239	2	68	33	50	13	14	160	57
	Passenger Car	34	498	21	46	234	2	67	33	47	12	12	151	57
	Truck	0	3	1	0	3	0	0	0	1	1	1	6	0
	Tractor-Trailer	0	3	0	1	1	0	0	0	0	0	1	2	0
8:15 AM	Bus	0	1	0	2	1	0	1	0	2	0	0	1	0
8:15 AM	Total Vehicles	41	394	28	60	245	3	93	29	50	22	13	164	63
	Passenger Car	41	391	27	60	232	3	91	28	45	22	12	161	63
	Truck	0	2	1	0	7	0	1	1	2	0	0	1	0
	Tractor-Trailer	0	0	0	0	4	0	1	0	1	0	0	2	0
8:30 AM	Bus	0	1	0	0	2	0	0	0	2	0	1	0	0
8:30 AM	Total Vehicles	27	324	23	59	226	2	84	36	51	15	26	145	58
	Passenger Car	26	318	23	59	219	2	84	32	49	14	25	142	56
	Truck	0	6	0	0	4	0	0	3	2	0	1	1	0
	Tractor-Trailer	0	0	0	0	2	0	0	0	0	0	0	1	0
8:45 AM	Bus	1	0	0	0	1	0	0	1	0	1	0	1	2
8:45 AM	Total Vehicles	29	404	34	59	244	5	71	27	53	15	22	142	53
	Passenger Car	28	397	34	57	236	5	70	27	50	15	21	135	53
	Truck	1	4	0	1	8	0	1	0	1	0	1	0	0
	Tractor-Trailer	0	2	0	1	0	0	0	0	0	0	0	3	0
9:00 AM	Bus	0	1	0	0	0	0	0	0	2	0	0	4	0
Total Vehicles		238	2757	154	323	1441	20	513	173	276	89	94	957	343
Total Passenger Car		234	2726	152	316	1396	20	505	168	260	84	89	928	341
Total Truck		3	21	2	3	29	0	4	4	8	4	3	13	0
Total Tractor-Trailer		0	6	0	2	9	0	3	0	1	0	1	10	0
Total Bus		1	4	0	2	7	0	1	1	7	1	1	6	2
% Passenger Cars		98%	99%	99%	98%	97%	100%	98%	97%	94%	94%	95%	97%	99%
% Heavy Vehicles		2%	1%	1%	2%	3%	0%	2%	3%	6%	6%	5%	3%	1%

Figure 21 A Sample of the Turning Movement Counts Summary for the DDI at Pleasant Hill Rd & I-85, Atlanta, GA (FHWA, Reduced Data, 2015)

4.2 Simulation Tool

There is a need to select an appropriate tool that has the ability to perform a detailed analysis at the microscopic level (Dhatrek et al., 2010). After reviewing many research and studies, there were many traffic microsimulation tools that have been used, but the most commonly used microsimulation software in the previous studies is VISSIM. VISSIM (version 8) is the tool that was used in the simulation and evaluation of the candidate-locations. VISSIM tool is a time-based, stochastic simulation of individual vehicles, which has many functions; ability of imitating the innovative designs, ability of simulating signal control plans and/or import signal plans from other tools, and the capability of easily running the simulation for different replications and random seeds and other factors. The software also has the ability to collect system wide measurements as well as movement, approach, link, route, area and other MOE possibilities (Hummer and Reid, 2000; Chu et al., 2003). The software also has the ability to develop animated graphics that can be displayed in 2-D or 3-D. There are many simulation parameters that need to be considered like the number of replications, simulation period, seeding number and calibration and validation of the simulation model. For the replication numbers, there are no exact number of replications that agreed, but based on most studies 10 replications are acceptable. However, in this research only one replication will be considered once the factorial experiment is used. For the simulation period, different period times have been used by other studies varying between 15 and 360 minutes, and this study was run for an hour and 15 minutes (El Esawey and Sayed, 2013). The first 15 minutes to warm up the system and ensure it is fully operational. Running the model with different replications and seeding numbers leads to reliable simulation outputs.

4.3 Modeling the Diverging Diamond Interchange

There is no tool that can confidently handle the innovative design variations, driver behaviors, travel paths, queues and signal timing implications. Nevertheless, it was used the most reliable and flexible tool able to simulate and evaluate the various unique elements of these innovative designs and traffic characteristics. When calibrating and validating any model that has been designed by VISSIM, there is a need to consider specific parameters that make the model more reliable to mimic the actual DDI design, which are the number of replications, simulation period, seeding number and driver behavior parameters (Toledo et al., 2003; Schroeder et al., 2014; Manjunatha et al., 2013). So, previous papers on how to determine the minimum and maximum number of each parameter and the accurate way to calibrate and validate such DDI design models were reviewed. The replication number is very helpful when improving the accuracy of the designed models and minimizing error between field and model observations. However, in this study, there was no need to replicate running the model because of the factorial design that was used when designing the experiment. Different simulation times have been used by other studies varying between 15 and 360 minutes, but an hour and fifteen minutes has been found to be enough simulation time to run the model. That is due to the first 15 minutes of the simulation time, ensuring the system is fully operational, the model is reliable and can be used *(*El Esawey and Sayed, 2013*)*. The driver-behavior parameters are one of the effective parameters on simulating any innovative design by using the microsimulation software. Maintaining the simulation parameters throughout the models leads to more reliable simulation outputs.

The geometric design and traffic characteristics for the Ashford Dunwoody RD & I-285, Atlanta, GA, DDI and its conventional diamond interchange design were needed before starting to code the DDI and CDI designs. The geometric design was obtained by using Google Earth application. The traffic characteristics were given in the data collection. The signal timing plans were also needed but as they were not available, it was necessary to run a signal optimization to determine the optimal signal plans and ensure model outputs matched field data.

This was the first time with the VISSIM software and it took approximately three months to learn how to use VISSIM and drawing such design. To start developing the model, the software required either to add a background, which is the image or drawing of the location or Bing Maps or VISSIM v.8 new service offers the actual map of the location. In this study, Bing Maps was used in coding the DDI design. Figures 22 and 23 show the coded DDI in this study. Also, the conventional diamond Interchange (CDI) of this location was coded because it is going to be needed in the comparison between the operational performance and the developing of the warrant (see figure 24), which was coded based on imagery background that was taken from Google Earth on 10/16/2011, before implementing the DDI.

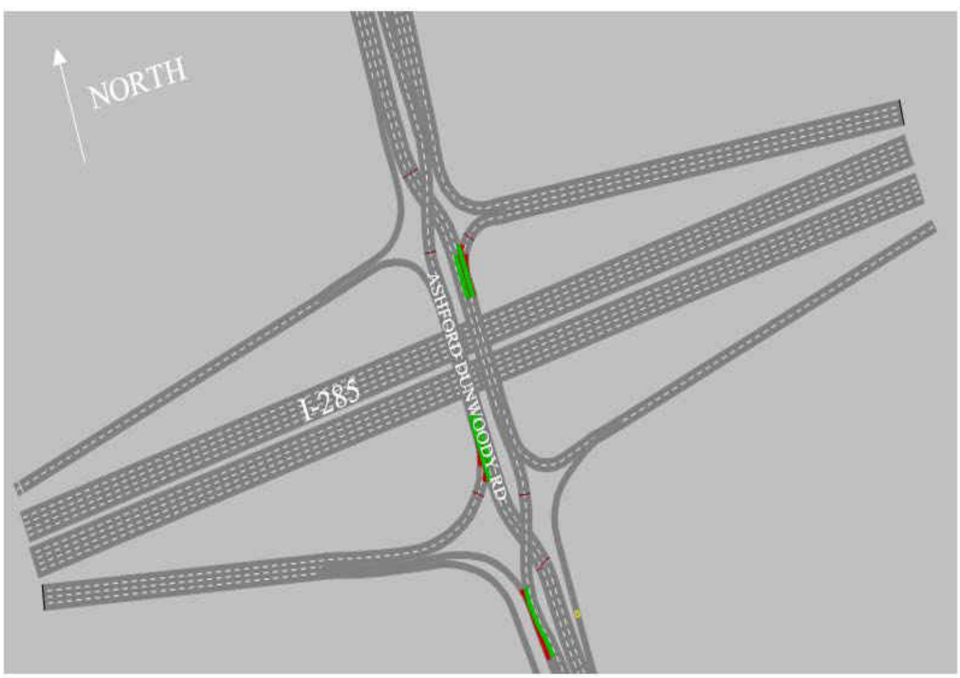


Figure 22 VISSIM- 8.0 Model for the Ashford Dunwoody Rd and I-285 DDI, Atlanta, GA

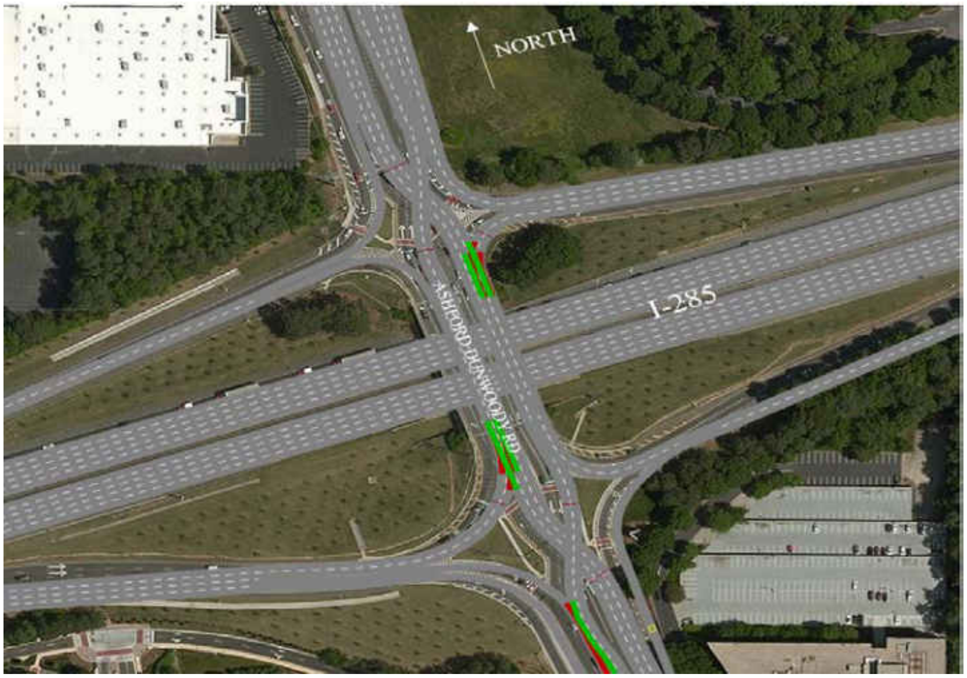


Figure 23 VISSIM- 8.0 Model for the DDI at Ashford Dunwoody Rd and I-285, Atlanta, GA

Figure 24 VISSIM- 8.0 Model for the CDI at Ashford Dunwoody Rd and I-285, Atlanta, GA

To test the coded locations and see how the models work, they were done with the VISSIM default values and assumed volume and signal timing values. Figures 25 and 26 show the DDI and CDI models running. The DDI has two signals at each crossover with two phases each (see figure 25). Phase 1 proceeds the southbound through traffics and the eastbound off-ramps left turn traffics. Phase 2 will, then, proceed the opposing movements coming from the southbound and westbound. The CDI usually runs with three-phase signals, phase 1 runs the northbound/southbound through traffic, phase 2 runs the off-ramps eastbound/westbound left-turn traffics and phase 3 runs the left turn traffics coming from the southbound towards the on-ramp eastbound direction. Since both DDI and CDI have been modeled, they will be calibrated and validated by using field data to ensure model outputs are a 95% match with field data.

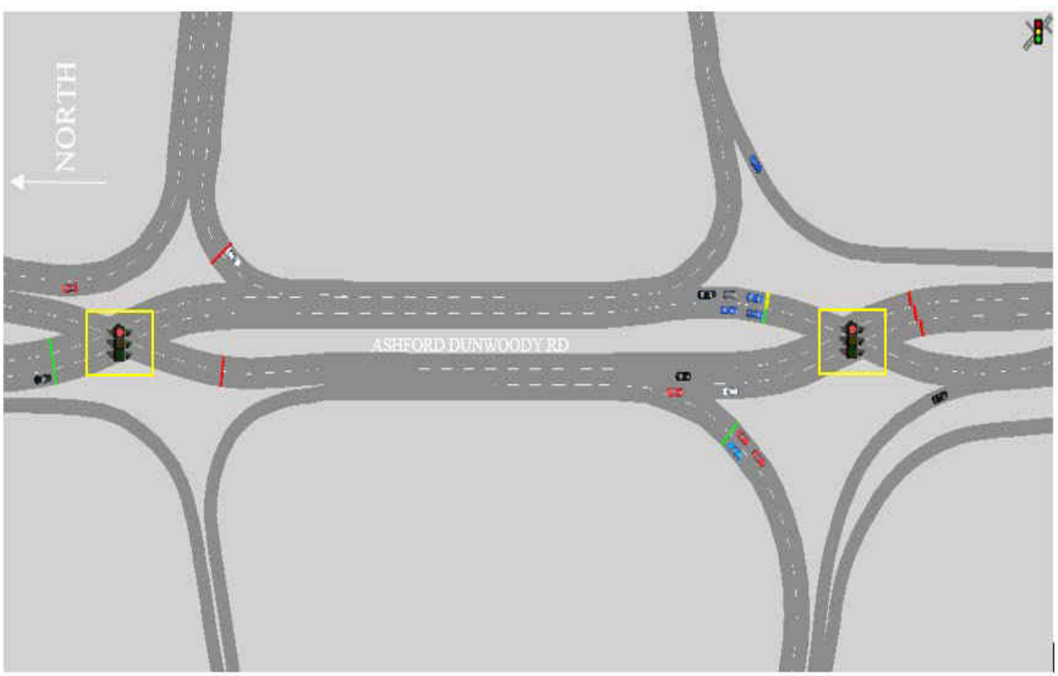


Figure 25 the Signals and Phases of the Model for the Ashford Dunwoody Rd & I-285 DDI, Atlanta, GA

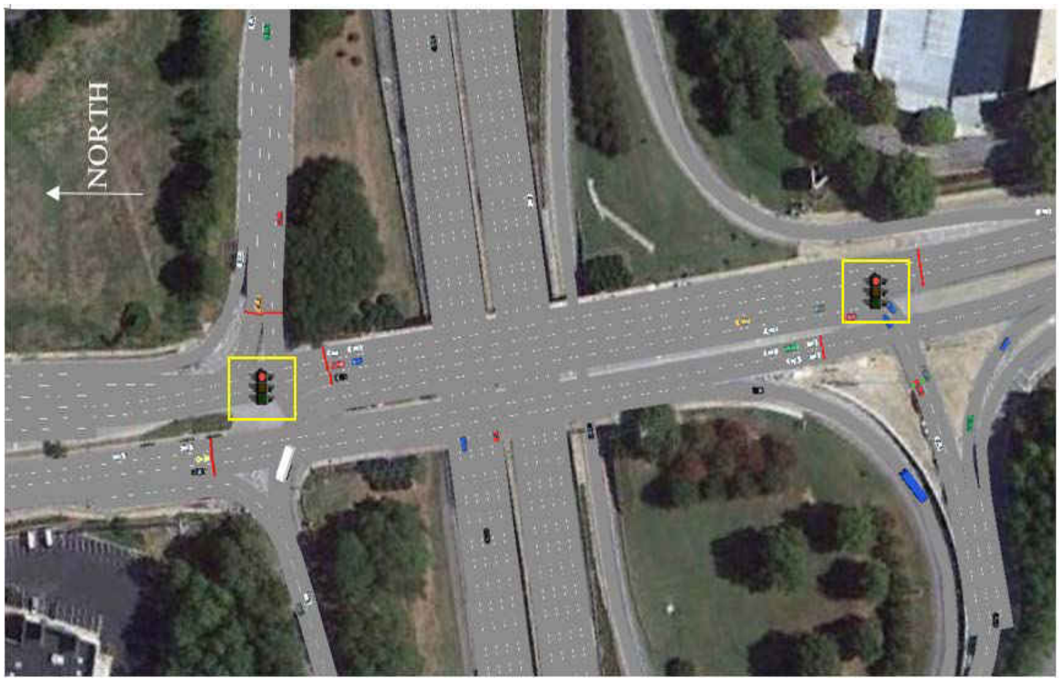


Figure 26 the Signals and Phases of the Model for the Ashford Dunwoody Rd & I-285 CDI, Atlanta, GA

4.4 Calibrating and Validating the Diverging Diamond Interchange

The most affected parameters, when using VISSIM software have been considered in coding the innovative design, however the models need to be calibrated and validated to match model outputs and field data, with 95% confidence interval. The calibration was done through two steps, which are signal timing optimization and driving behavior sensitivity analysis.

4.4.1 Signal Timing Optimization

The most affected parameters, when using VISSIM software have been considered in coding the innovative design, however the models need to be calibrated and validated to match model outputs and field data, with 95% confidence interval. The calibration was done through two steps, which are signal timing optimization and driving behavior sensitivity analysis.

4.4.1 Signal Timing Optimization

Since the signal timing for the studied location was not available, it was necessary to optimize the signal to determine the optimal signal plans that ensured the model-output in 95% or more (matching field data). The signal timing plans for the Diverging Diamond Interchange can be optimized by using either Synchro software or manually. When trying to use the signal optimization software to get the optimal signal plan that will be a 95 % match, or more of the VISSIM output with the observed data, at the proposed interchange, it was found that synchro does not perform signal optimization by a specific function. However, the signal optimization was performed by Synchro on other researches by considering the two DDI crossovers as two separate intersections (Yang et al., 2014; Hu et al., 2014). Consequently, the signal optimization in this study was performed manually, using VISSIM. Since the DDI design can be operated with two-phase control - each phase dedicated to the alternative opposing movements, the signal plan has used a fixed time plan. As shown in Figure 27 & 28, DDI is operated with one signal controller and two phases (Olarte and Kaisar, 2011; Goldblatt et al., 1994). Phase 1 allows the northbound traffic to cross the south and north crossovers and west off-ramp traffic to make left and right-turn without any conflict. The same happens to the south and east bound traffics with Phase 2. The purpose of this study is to evaluate the performance of the left-turn movement and, consequently, the pedestrian phase will not be considered.

Seven different cycle length scenarios were used to optimize the signal plan under different volumes. Three of these scenarios had 60-second cycles, one a 75-second cycle, one a 80-second cycle and two 90-second cycles with different green times (g/c), that is, 30/60, 40/60, 35/60, 40/70, 40/80, 45/90 and 54/90, respectively. The signal timing scenario with shortest left-turn delay, highest capacity and highest match percentage (between the input and output data) were selected as the optimized signal plans (as per comparison index). For the delay, the 60-second cycle plan presented the shortest average delay per vehicle. For the capacity, the signal plans with 30/60 and 35/60 g/c ratio presented the highest capacity, which excluded the 40/60 signal plan. Since this research was looking for better performance, for all approaches and no preference was given to any movement, the 60-second cycle with an 30-second green time for each phase was selected to be the optimized signal plan, although some cycles had better performances for certain movements, which other studies also suggested and used. For the conventional diamond interchange (CDI), the same steps have been followed to optimize the signal timing with three-phases, two-signal controller and 12 candidate-cycle scenarios (60, 90 and 120 second) with different g/c ratios. The selection criteria for the optimal signal plan was the same DDI criteria and the 90-second cycle was selected as the optimal signal timing, which showed the highest throughput and shortest delay (see Appendix A).

The optimum signal plans that ensure the shortest delay and highest capacity - for DDI and CDI designs - were selected. The next section will look at the driving behavior parameters.

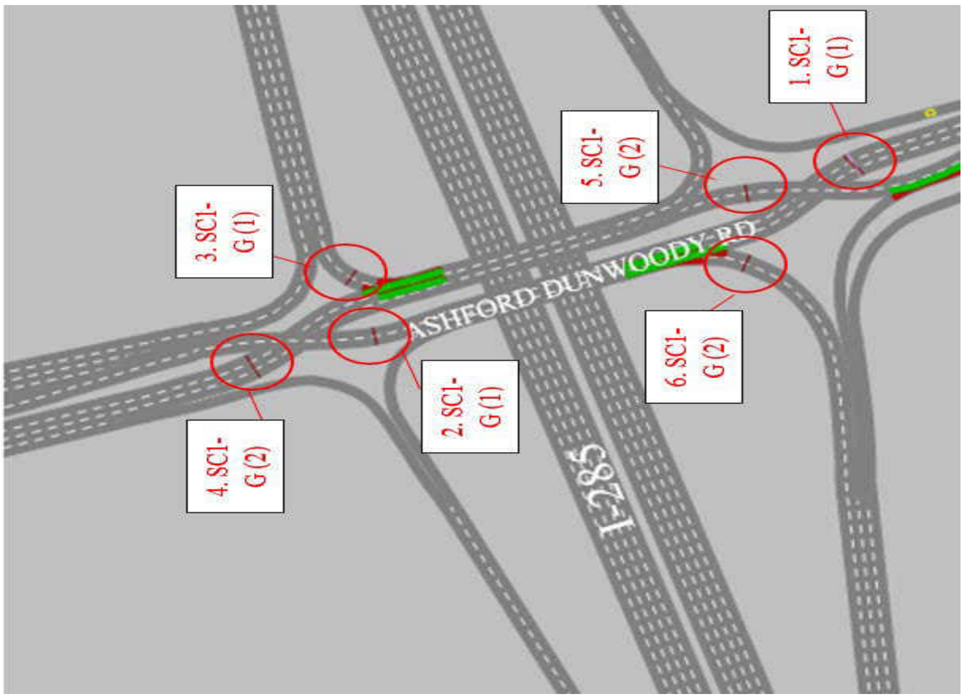


Figure 27 Signal Controller Locations for DDI

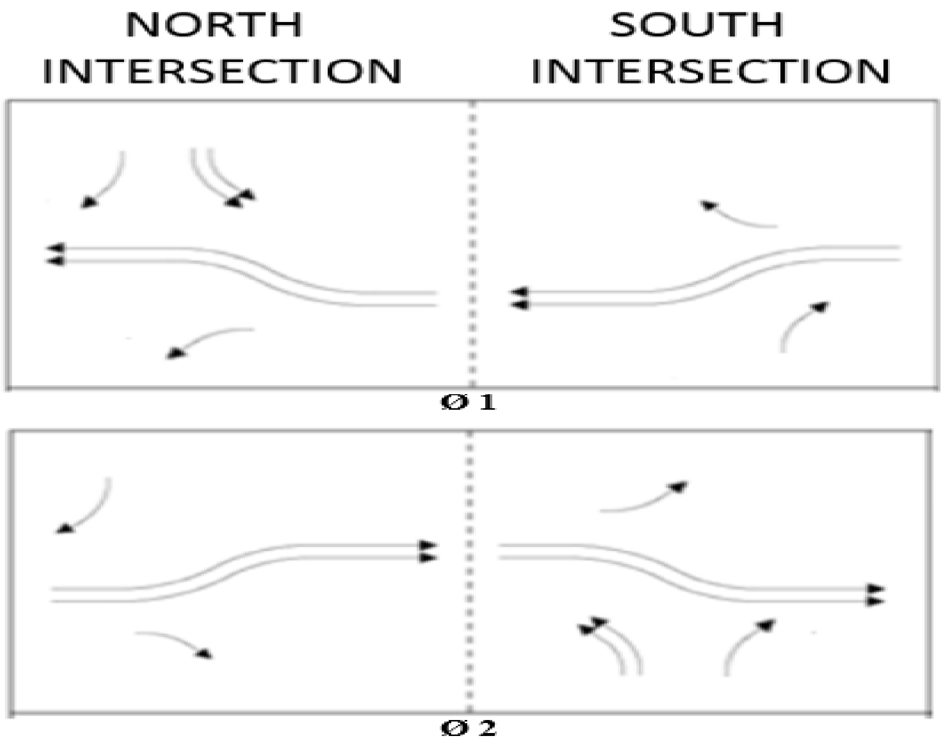


Figure 28 Signal Phasing for DDI

4.4.2 Driver Behavior Parameters

In order to calibrate the coded models and improve their reliability when mimicking field conditions, it was necessary to choose a set of driver-behavior parameters and run the model under different parameter values and, then, select the best set of parameters that will provide a 95% or more match between input demands and model outputs. Five parameters under Wiedmann 99 have been identified as the most influential parameters: CC0, CC1, CC2, CC7 and CC8 (Toledo et al., 2003; Russo, 2008; Lownes and Machemehl, 2006) (see Table 13). A sensitivity analysis was preformed to closely examine these 5 parameters, in order to calibrate the models and obtain the optimal set of parameters that offer the lowest error margin (5% or less) between model input and output. The implementation of such method allowed the identification of the set of parameters with the most substantial impact on interchange design, which is hard to identify in the real life. The

VISSIM model was run with parameters' default values, which are shown in red (Table 14) as initial evaluation and the percentage between the input and output data was 84%, which meant that further alterations to parameters were required for calibration. If error percentage - for the initial evaluation using the default values - was inferior to 5%, then, further calibration would be unnecessary (Toledo et al., 2003; Tarko et al., 2008). Multiple scenarios with different Wiedmann 99 parameter values were run by sensitivity analysis (trial and error) until model calibration was completed. Each parameter value in driver's behavior parameters was changed two levels high and two levels low and run one at a time when all other parameter values were constant, and, in each case, model capacity output was observed and if it varied from the input value for 25 iterations (See Table 14). The parameter value with the smallest difference between the input and output was identified as the most significant value: average standstill distance (CC0) at value 1.64 ft., desired headway at value 0.7 seconds and following variation at value 6.56 ft. Default values for the other two parameters have been found as the best values to be used for the models. Once the optimal driving behavior parameter set has been identified, the model was run under such set of parameters. The model ran under the optimal set of driving behavior parameters and error between actual data and model output was inferior to 3 %.

Models were calibrated, but it was still necessary to validate the developed models so that they became more reliable. This way, the collected data for the other interchange location was used as demand input and matched the calibrated model outputs (traffic volume and MOE) within 95% confidence interval. Error between the actual data and model outputs was 4%, which falls within the confidence interval.

Table 11Wiedemann 99 Parameters

Category	VISSIM Code	Description	Default Value
Thresholds for DX	CC0	**Standstill distance:** Desired distance between lead and following vehicle at v = 0 mph	4.92 ft
	CC1	**Headway Time:** Desired time in seconds between lead and following vehicle	0.90 sec
	CC2	**Following Variation:** Additional distance over safety distance that a vehicle requires	13.12 ft
	CC3	**Threshold for Entering 'Following' State:** Time in seconds before a vehicle starts to decelerate to reach safety distance (negative)	-8.00 sec
Thresholds for DV	CC4	**Negative 'Following' Threshold:** Specifies variation in speed between lead and following vehicle	0.35 ft/s
	CC5	**Positive 'Following Threshold':** Specifies variation in speed between lead and following vehicle	0.35 ft/s
	CC6	**Speed Dependency of Oscillation:** Influence of distance on speed oscillation	11.44
Acceleration Rates	CC7	**Oscillation Acceleration:** Acceleration during the oscillation process	0.82 ft/s^2
	CC8	**Standstill Acceleration:** Desired acceleration starting from standstill	11.48 ft/s^2
	CC9	**Acceleration at 50 mph:** Desired acceleration at 50 mph	4.92 ft/s^2

Table 12 Wiedemann 99 Parameters Sensitivity Analysis Scenarios

Wiedemann 99 Parameters				
CC0	**CC1**	**CC2**	**CC7**	**CC8**
1.64	0.9	13.12	0.82	11.48
3.28	0.9	13.12	0.82	11.48
4.92	0.9	13.12	0.82	11.48
6.56	0.9	13.12	0.82	11.48
8.20	0.9	13.12	0.82	11.48
4.92	0.7	13.12	0.82	11.48
4.92	0.8	13.12	0.82	11.48
4.92	0.9	13.12	0.82	11.48
4.92	1	13.12	0.82	11.48
4.92	1.1	13.12	0.82	11.48
4.92	0.9	6.56	0.82	11.48
4.92	0.9	9.84	0.82	11.48
4.92	0.9	13.12	0.82	11.48
4.92	0.9	16.40	0.82	11.48
4.92	0.9	19.69	0.82	11.48
4.92	0.9	13.12	0.49	11.48
4.92	0.9	13.12	0.66	11.48
4.92	0.9	13.12	0.82	11.48
4.92	0.9	13.12	0.98	11.48
4.92	0.9	13.12	1.15	11.48
4.92	0.9	13.12	0.82	4.92
4.92	0.9	13.12	0.82	8.20
4.92	0.9	13.12	0.82	11.48
4.92	0.9	13.12	0.82	14.76
4.92	0.9	13.12	0.82	18.04

4.5 Measures of Effectiveness (MOE)

There were many measures of effectiveness (MOE) used in previous studies to compare between the distinctive designs (DDI and CDI), however two MOE were identified to be used in this study, that is, average delay and capacity (maximum throughput of movements) (El Esawey and Sayed, 2013; Dhatrek et al., 2010). These MOEs are affected by many factors such as traffic volumes, geometric designs and signal plans and it is logical to design an experimental design that takes into consideration these parameters. Since DDI design has been especially designed to improve the left-turn movement, only left-turn delay and capacity will be used in the analysis. Since the delay and capacity of DDI and CDI designs are going to be compared to each other and the effectiveness of the innovative design on the interchange's operation will be checked, there are two left-turn movements on the DDI design that need to be measured and combined to be able to conduct the comparison. The delay of left-turn movements is measured from two points at the interchange (see figure 29). For DDI, the left-turn delay is measured from point 1 to point 4 and from point 5 to point 3 (figure 29). Then, these two delay measures are added to each other - considered as the DDI left-turn delay. The total number of vehicles that passed these two points, while running the model for an hour, are counted and considered as the maximum DDI throughput (capacity). For the CDI, the left-turn delay is measured from point 1 to point 4 and from point 5 to point 3 (figure 30). Then, these two delay measures are added to each other and considered as the CDI left-turn delay. The total number of vehicles that passed these two points, while running the model for an hour, are counted and considered as the maximum CDI throughput (capacity).

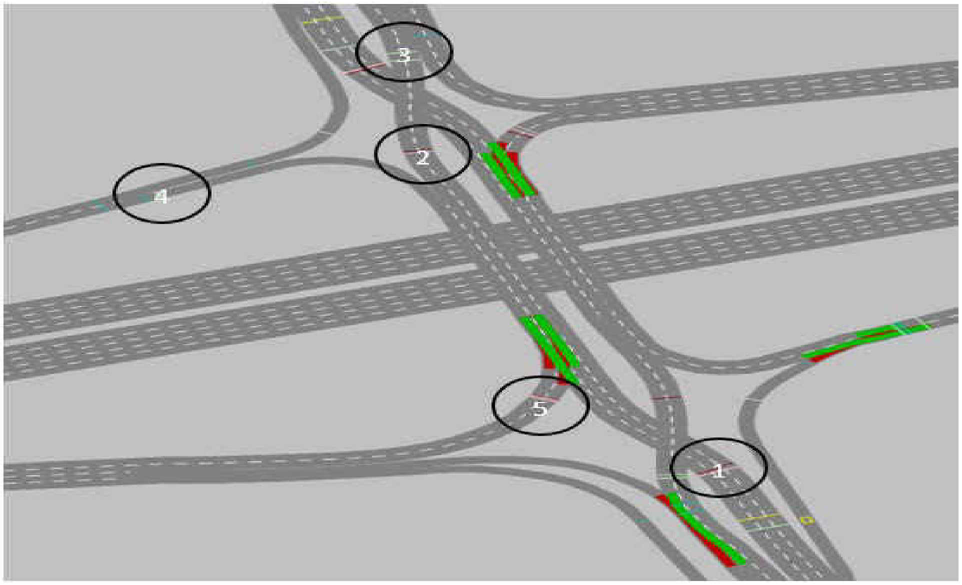


Figure 29 The DDI Left-turn Delay and Capacity Measures of Effectiveness

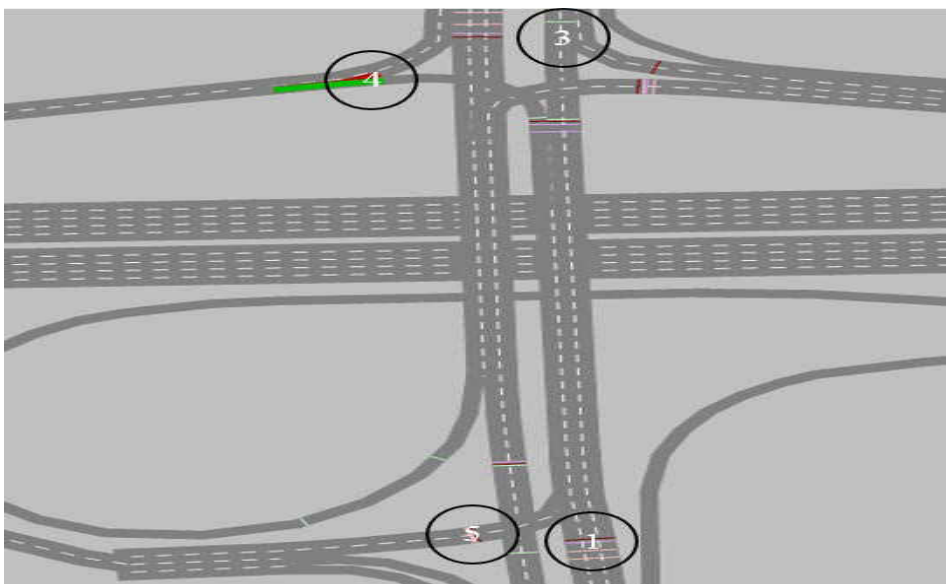


Figure 30 The CDI Left-turn Delay and Capacity Measures of Effectiveness

4.6 The Design of Experiment

The measures of effectiveness are affected by varied factors such as spacing distance, number of lanes and traffic volume levels, which were included in the experimental design and led to 90 different scenarios for each design (DDI and CDI) (see table 15). Table 15 shows three different scenario groups, which are categorized based on the spacing distance between the two crossovers of the interchange, that is, 850, 1200, and 1550 feet. Also, two and three levels of number of lanes for the left (LT) and through (Thru) movements, respectively. One and two number of lanes for the left turn movement and two, three, and four number of lanes for the through movement. Five volumes per lane levels shown in table 15, that is, 500, 750, 1000, 1250, and 1500 vehicles per hour, per lane and under each level, showing the total volume, per approach, for each scenario, including 5% right-turn volume.

Table 13 Experimental Design Parameters and scenarios for DDI and CDI

Scenario Group	Sub Group Iteration	Spacing Distances, (ft)	Number of Lanes		Volume Levels, (Vehicle per hour/Lane)				
			LT	Thru	500	750	1000	1250	1500
1	1.1	850	1	2	1579	2408	3237	4066	4895
	1.2		1	3	2105	3210	4316	5421	6526
	1.3		1	4	2632	4013	5395	6776	8158
	1.4		2	2	2105	3210	4316	5421	6526
	1.5		2	3	2632	4013	5395	6776	8158
	1.6		2	4	3158	4816	6473	8131	9789
2	2.1	1200	1	2	1579	2408	3237	4066	4895
	2.2		1	3	2105	3210	4316	5421	6526
	2.3		1	4	2632	4013	5395	6776	8158
	2.4		2	2	2105	3210	4316	5421	6526
	2.5		2	3	2632	4013	5395	6776	8158
	2.6		2	4	3158	4816	6473	8131	9789
3	3.1	1550	1	2	1579	2408	3237	4066	4895
	3.2		1	3	2105	3210	4316	5421	6526
	3.3		1	4	2632	4013	5395	6776	8158
	3.4		2	2	2105	3210	4316	5421	6526
	3.5		2	3	2632	4013	5395	6776	8158
	3.6		2	4	3158	4816	6473	8131	9789

4.7 Analyses and Results

After models' calibration and validation, the scenarios based on the three groups of experimental design were built. Eighteen different models were coded for different number of lanes and spacing distances. Figures 31, 32 and 33 show examples of different scenarios for the crossover distances and number of lanes. All the scenarios for DDI and CDI will be included in Appendix [B]. Each scenario was run under 5 levels of traffic conditions to simulate peak and off-peak traffic and search for the crossing point that makes the DDI design superior to the CDI design. This experiment resulted in 90 scenarios of simulation runs for each design (DDI and CDI). In addition, the models were used to evaluate DDI operational performance, if compared to its conventional design in terms of delay and left-turn movement capacity.

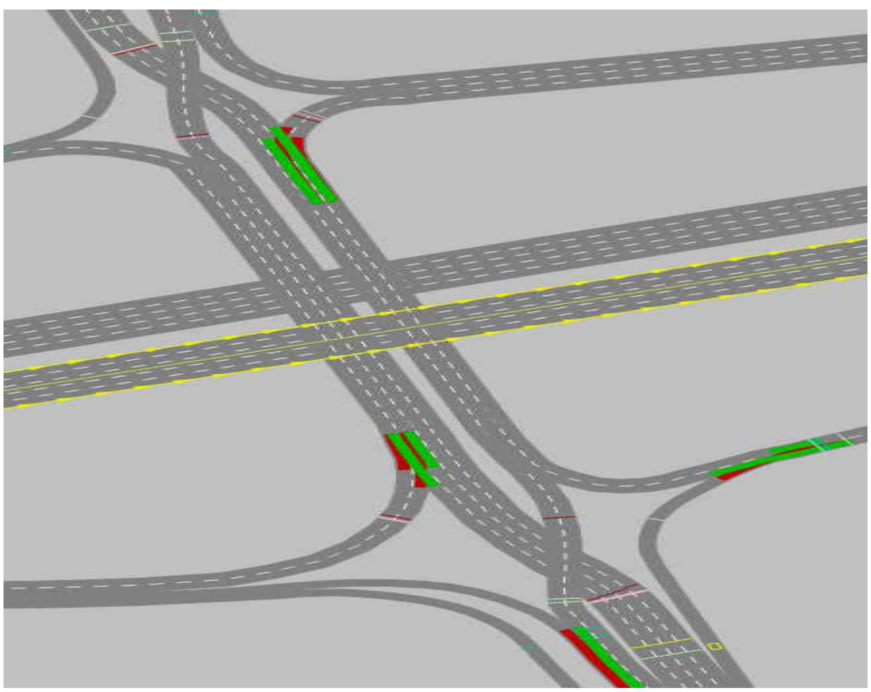


Figure 31 DDI Scenario with 2Thru - 2LT and 850 ft Crossover Distance

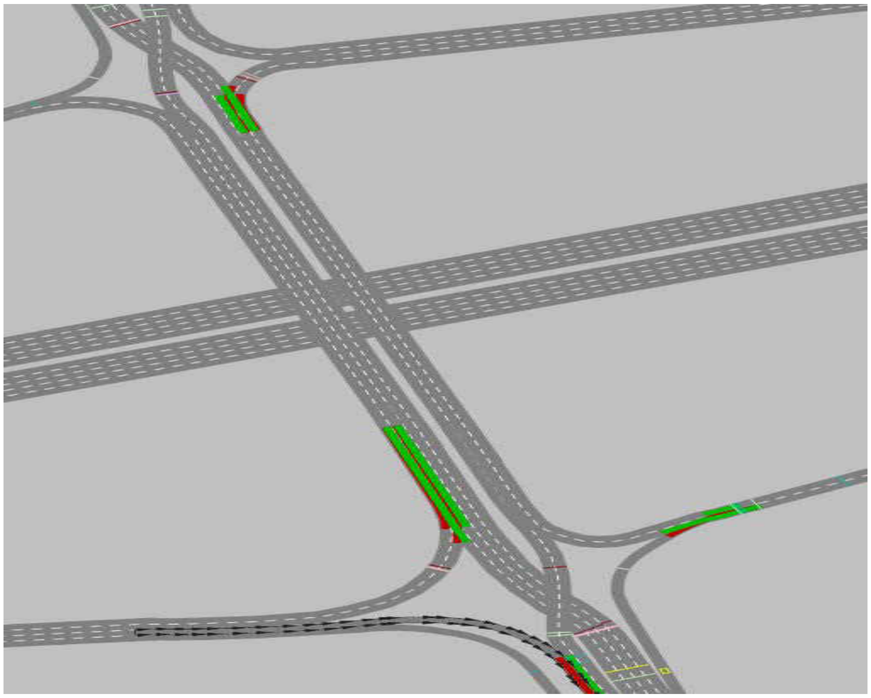


Figure 32 DDI Scenario with 2Thru - 2LT and 1200 ft Crossover Distance

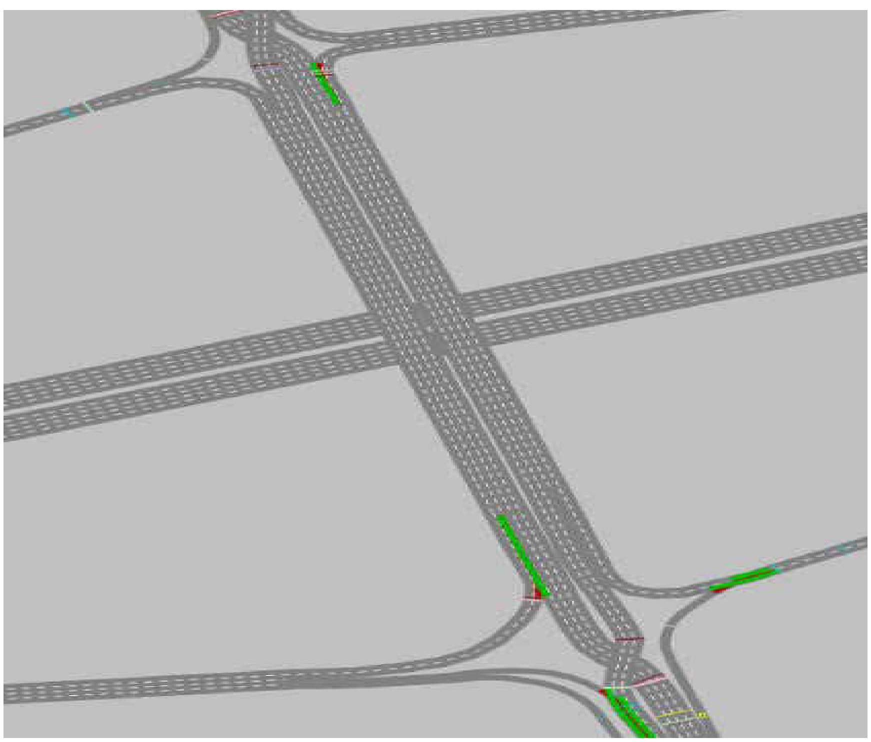


Figure 33 DDI Scenario with 3Thru - 2LT and 1550 ft Crossover Distance

The first group's analysis, sharing the same crossover distance (850ft), but with a different number of lanes and volume levels, was conducted using charts to compare each design; having the same parameters, that is, DDI and CDI having the same number of lanes, spacing distance and volume level and compared to each other by plotting graphs that show both design (DDI and CDI) performance at the same time. In terms of left-turn delay, the DDI design outperformed the CDI at all traffic volume levels, in 850 ft. crossover distance with more evident superiority at higher traffic volumes, as shown in Figure 34. For the left-turn capacity, the DDI design outperformed the CDI design and all scenarios had a cross point from one design to the other, but this cross happened at different volumes, ranging from 500 to 750 vehicles per hour/lane, as depicted in Figure 35. It is observed that as the traffic volume exceeds 750 vehicles/hour/ln, the difference between the two designs in terms of delay and capacity increases tremendously. It was also obvious that as the through volume grew - by the increasing number of through lanes with both left-turn lane scenarios - the left-turn capacity for the DDI and CDI diminished owing to the addition of more volume to the design. However, adding more left-turn lanes that would improve DDI's capacity - more than CDI's capacity when the number of through lanes is constant.

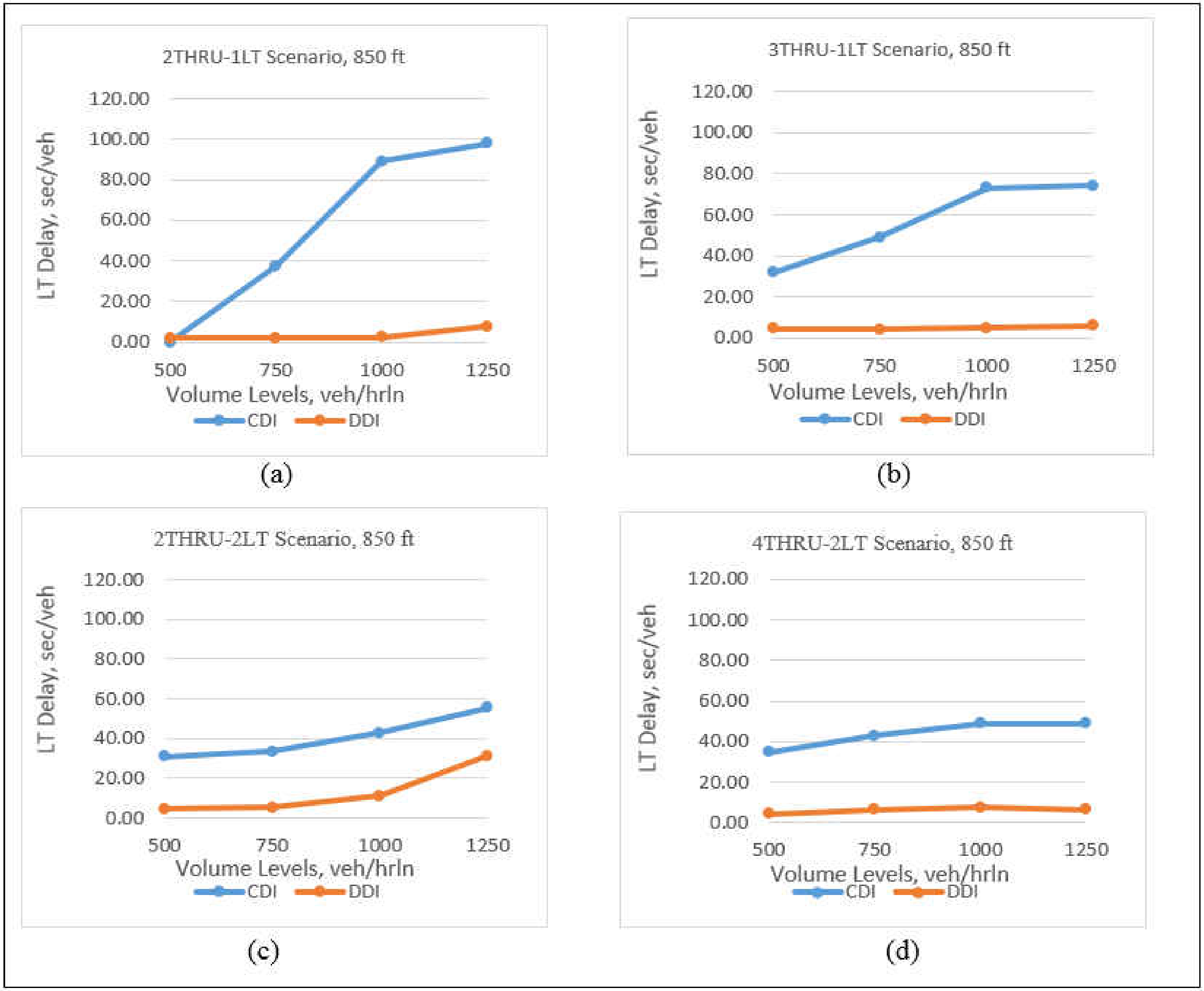


Figure 34Average Left-turn Delay Graphs for Different Scenarios for 850 ft crossover

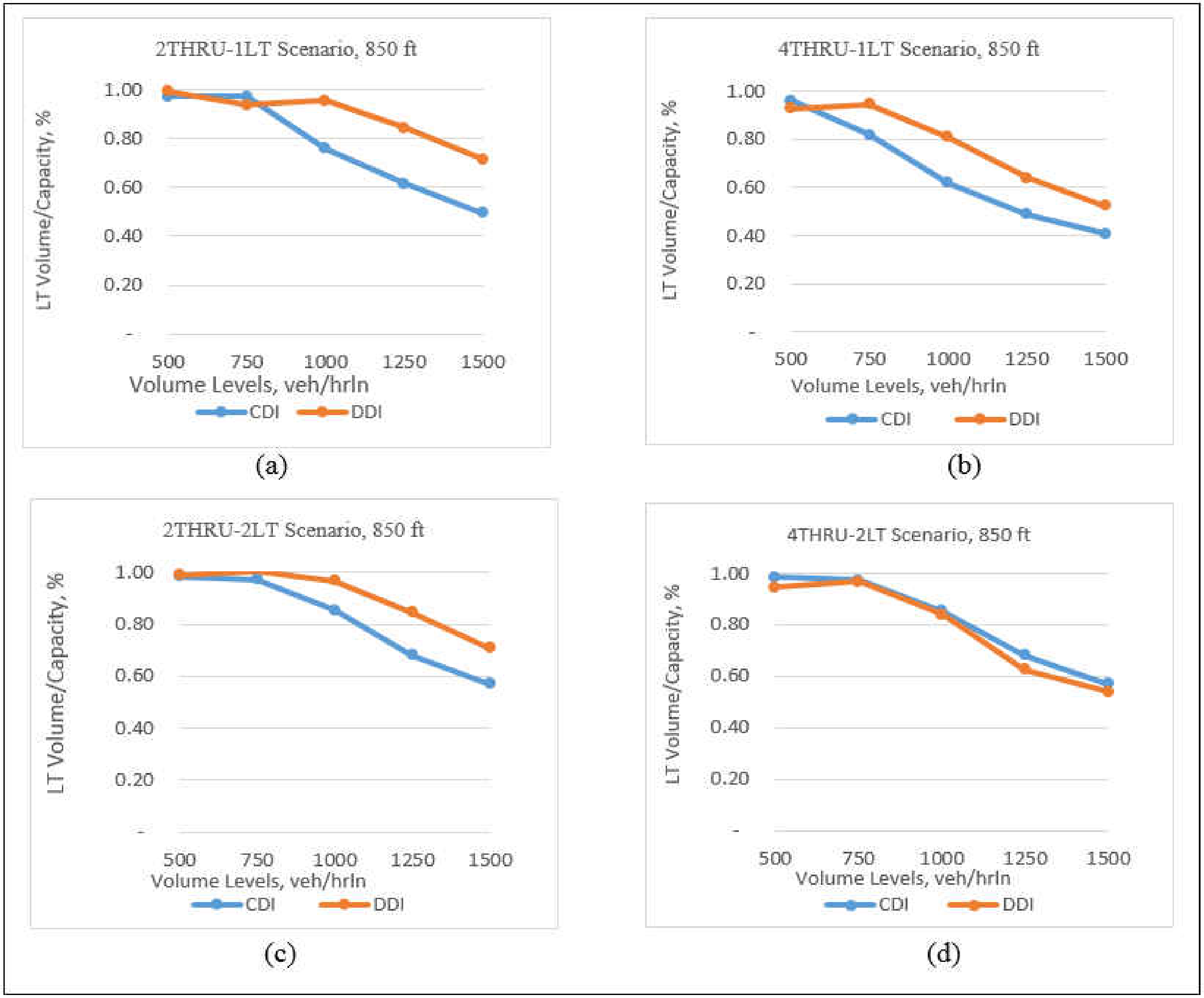


Figure 35Left-turn capacity Graphs for Different Scenarios for 850 ft crossover

In the second group's analysis, where the crossover distance is 1200 feet, results showed that DDI outperformed CDI in terms of delay and capacity. The left-turn delay did not have any cross point between the CDI and DDI designs; however, all the left-turn capacity scenarios have the cross points between the two designs (see Figure 36). The cross point for the scenario with one left-lane and 2 thru lanes occurred at 750 vehicles/hour per lane, but the cross point went down to 500 vehicles/hour per lane while increasing the through lanes and holding the number of left-turn lanes. That was caused by the addition of more through traffic when increasing the number of through lanes, which has more effect on the efficiency of CDI, if compared to DDI (see figure 37). While the tipping point (for the 2 left and 2, 3 and 4 through lanes) crossed around 500 to 600-vehicle level, due to left-turn volume increase, owing to the increase of the number of left-turn lanes, which reduced CDI left-turn performance (see figure 37). At the lowest volume level CDI and DDI performed the same and at some scenarios CDI performed better than DDI.

The analysis of the third group - in which all scenarios had the same crossover distance (1550 ft.), under different traffic volume levels and number of lanes - lead the same results seen in the other groups in terms of left-turn delay and capacity. The cross point happened at 750-vehicle level for the 1 left and 2 through lane scenario and the threshold dropped to 600 vehicles/hour per lane as the through lanes increased. For the 2 left-lane scenarios, the tipping point was observed at lowest volume level and the point goes up to 600 volume level when the number of through lanes reaches 4 lanes.

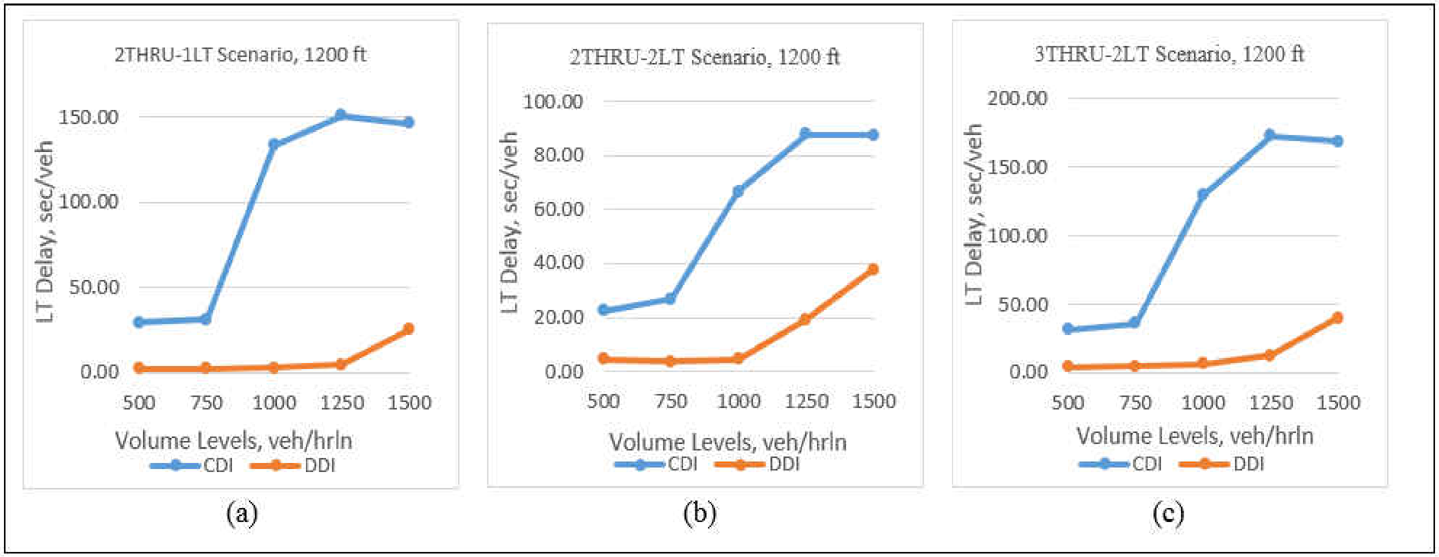


Figure 36Average Left-turn Delay Graphs for Different Scenarios of 1200 ft crossover

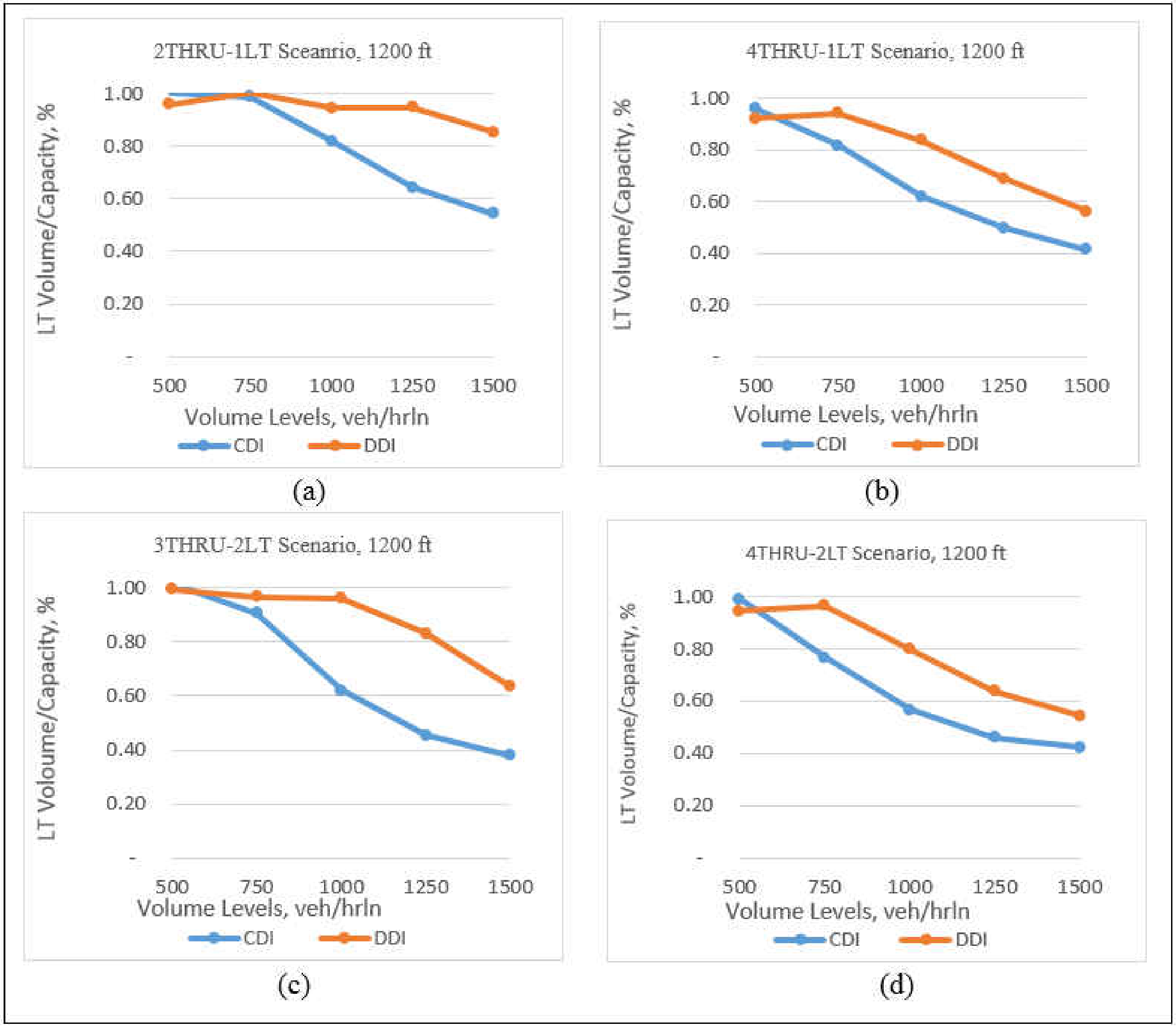


Figure 37Left-turn Capacity Graphs for Different Scenarios of 1200 ft crossover

CHAPTER 5: CONTINUOUS FLOW INTERSECTION

The increased left-turn demand is the main cause of congestion at the signalized intersections, and the available countermeasures are not adequate to relieve this congestion (Goldblatt et al.,1994; Reid and Hummer, 1999; Abou-Senna and Radwan, 2016). The innovative design called Continuous Flow Intersection design seems promising on improving the operational performance at the signalized intersections and that was proven by previous literature (UDOT, 2013; Abou-Senna and Radwan, 2016). This section looks at the justifying threshold to redesign a signalized intersection from a Conventional Intersection (CI) to a Continuous Flow Intersection (CFI) and elaboration of performance guidelines to help decision-makers and professionals when considering the alternative. To accomplish the study’s objective, a complete understanding of CFI was done by reviewing previous literatures and documented in the methodology chapter. The candidate-locations and data collection (selected and used for the analysis) are described below. The chapter also includes the simulation tool that has been selected and used to develop and evaluate the innovative design models. The modeling and evaluation are followed by two sections that include the measures of effectiveness and the experiment that has been specifically designed to examine the parameters that substantially affect the CFI design. The last section in this chapter includes the study’s results and analyses.

5.1 Locations and Data Collections

CFI design is relatively new, there were no innovative intersections implemented in Florida when this study started. The candidate-locations, under consideration, are outside the state of Florida. There are several new locations in various U.S. regions, which have already implemented different innovative designs. However, not all of them had sufficient data available and it was difficult to obtain data on such locations. The candidate pool, thus, shifted to locations that had, a while back, implemented CFIs. Mainly locations where agencies or authorities that had collected data when implementing these designs, which they were willing to share. The Federal Highway Admiration - FHWA), represented by Dr. David Yang and Dr. Wei Zhang, was, then, contacted to provide some suggestions for locations to innovative designs and were kind enough to recommend a list of distinct locations. However, only two of these locations had CFIs, and both were still under construction, and were not going to be ready within two years. A professor at Utah University was then contacted, and he was able to provide five different CFI locations along the Utah State Route 152 (Bangerter Highway):

1. 3100 South in West Valley City, Utah (Implemented)
2. 3500 South (SR-171) in West Valley City, Utah (Implemented)
3. 4100 South in West Valley City, Utah (Implemented)
4. 4700 South in Taylorsville and West Valley City, Utah (Implemented)
5. 5400 South (SR-173) in Taylorsville, Utah (Implemented)

The only intersection that has a 4-leg CFI is Bangerter Highway and 4100 S. Rd, this location was chosen as the candidate for the study.

5.1.1 Bangerter Highway and 4100 S. Rd, West Valley, Utah

The first CFI location in this study is located along the Bangerter Highway, Utah which has several implemented CFIs. This CFI was built in 2011 and it was the first 4-leg CFI in the U.S. The geometric configuration of this location has two through lanes, one left turn bay, and one through and right shared lane for the eastbound and westbound (EB/WB) approaches. For the northbound and southbound (NB/SB), it has three through lanes and one dedicated right-turn lane and one left-turn lane bay for the northbound and two left-turn bays for the southbound approach.

This intersection is operated as a full CFI because all the left-turn lanes on all four approaches are shifted from the right side of the road to the left of the opposing roadway, which results in eliminating the left-turn phase at the main intersection (see figures 38 and 39). This shifting occurs by crossing the left-turn traffics, the opposing through lanes at new signalized intersections that are located 500 to 700 feet from the main intersection. Then, the shifting left turn traffics travel on the roadway that parallel to opposing lanes and make the left-turn at the main intersection simultaneously with the through traffic at the main intersection. The shifting at the four approaches will result in five traffic signals, one at the main intersection and four at the secondary intersections as shown in figures 38 and 39. Three signal controllers (SC) were needed for the CFI design. The first signal controller (SC1) at the main intersection, the SC2 was at the east and west secondary intersections and the last controller SC3 was at the north and south secondary intersections. The intersection is operated with only two phases. Phase 1, the through and the left-turn traffics of the northbound and southbound at the main intersection proceeds simultaneously. At the same phase the through traffic at the north and south secondary intersections and the left turn traffic at the east and west secondary intersections get the green. The second phase, all the remaining movements proceed.

The provided data for this location are as follows:

- Turning movement counts (TMC);
- Average travel time;
- Origins and designations;
- The calculated network performance;
- Average calculated delay;
- Traffic volume.

A sample of the volumes, turning movement counts (TMC) and calculated average travel times (Avg. TT) are shown in figures.

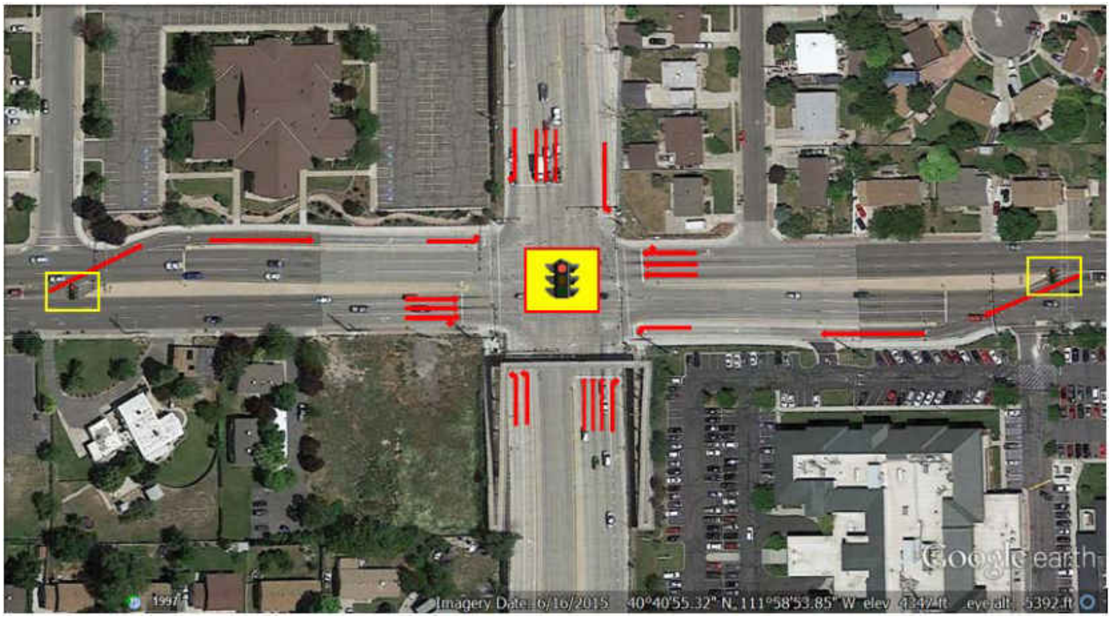


Figure 38 Lane Configuration and Signals of the CFI at the Bangerter Highway and 4100 S. Rd, West Valley, Utah (Source: Google Earth, 2016)-Part1

Figure 39 Lane Configuration and Signals of the CFI at the Bangerter Highway and 4100 S. Rd, West Valley, Utah (Source: Google Earth, 2016)-Part2

FIELD COUNTS FROM UDOT														
TIME PERIOD		Southbound			Westbound			Northbound			Eastbound			TOTAL
FROM:	TO:	SBL	SBT	SBR	WBL	WBT	WBR	NBL	NBT	NBR	EBL	EBT	EBR	VOLUMES
5:00 PM	6:00 PM	110	2604	418	272	1234	68	322	890	280	108	822	202	7330
Total		110	2604	418	272	1234	68	322	890	280	108	822	202	7330

Figure 40 A sample of Turning Movement Counts (TMC) for the CFI at Bangerter Highway and 4100 S. Rd, West Valley, Utah (Source: UDOT , 2013)

CAR	No.	Segment	Avg. TT VISSIM	GPS	ST dev VISSIM	ST dev GPS
SB	1	3100 S-3500 S	45.2	28.0	12.51	2.4
	2	3500 S-4100 S	83.7	65.0	16.34	6.6
	3	4100 S-4700 S	80.2	82.0	24.83	10.2
	4	4700 S-5400 S	81.1	83.0	18.04	7.8
	5	5400 S-6200 S	97.9	85.0	25.93	9.6
NB	6	6200 S-5400 S	80.1	84.0	23.05	12.6
	7	5400 S-4700 S	76.8	76.0	7.53	9.0
	8	4700 S-4100 S	80.4	75.0	18.81	12.0
	9	4100 S-3500 S	56.0	59.0	11.48	3.6
	10	3500 S-3100 S	66.7	37.0	27.34	12.6

Figure 41 A Sample of Calculated Average Travel Times for the all the Provided CFIs along the Bangerter Highway, Utah (Source: UDOT , 2013)

Date	Northboun	Southboun	Eastboun	Westboun	NB hr	SB hr	EB hr	WB h
4/2/2013 14:35	83	44	33	40	996	528	396	480
4/2/2013 14:40	81	44	32	45	972	528	384	540
4/2/2013 14:45	83	68	35	46	996	816	420	552
4/2/2013 14:50	75	65	46	43	900	780	552	516
4/2/2013 14:55	92	58	37	48	1104	696	444	576
4/2/2013 15:00	75	32	32	43	900	384	384	516
4/2/2013 15:05	72	55	38	44	864	660	456	528
4/2/2013 15:10	57	52	49	44	684	624	588	528
4/2/2013 15:15	71	51	34	45	852	612	408	540
4/2/2013 15:20	74	62	34	45	888	744	408	540
4/2/2013 15:25	90	71	35	39	1080	852	420	468
4/2/2013 15:30	80	51	50	45	960	612	600	540
4/2/2013 15:35	63	49	32	48	756	588	384	576
4/2/2013 15:40	65	57	29	51	780	684	348	612
4/2/2013 15:45	68	65	42	41	816	780	504	492
4/2/2013 15:50	74	85	38	44	888	1020	456	528

Figure 42 A Sample of Volumes for the CFI at Bangerter Highway and 4100 S. Rd, Valley City, Utah (Source: UDOT , 2013)

5.2 Simulation Tool

There are many micro-simulation tools for traffic analysis, however, none of them can accurately handle the CFI design variations, travel paths, signal timing implications, driver behaviors and queues. One of the most commonly used micro-simulation software is VISSIM, and it was selected mainly for its reliability and flexibility. VISSIM V.8 is a microscopic time-based, behavior-based, stochastic simulation tool. It has the ability to:

- Imitate innovative designs;
- Simulate signal control plans and/or import signal plans from other tools;
- Be easily replicated;
- Run the simulation for random seeds and other factors;
- To collect various measurements throughout the network, allowing a closer look at different measure of effectiveness;
- Develop animated 2-dimensional and 3-dimensional models.

There are numerous simulation parameters that were taken into consideration while simulating, calibrating and validating the CFI and CI designs. One of these parameters is the simulation period. Previous studies have used simulation periods that vary between 15 and 360 minutes, in this study however 60 minutes was used as the simulation period and it was the most used period plus 15 minutes in the beginning to warm up and ensure the system is fully operational and to simulate real-life situations. In order to produce reliable simulation outputs, the models were run using varying replication and seeding numbers (Kim et al., 2007). The models should be run using varying replication and seeding numbers, however one replication number was enough for this study because due to the factorial design. All these parameters will ensure a reliable model to perform the analysis.

5.3 Modeling of the Continuous Flow Intersection

The way of coding any design is similar, but there are some different geometric elements from one design to another that poses unique difficulties to each design's coding. CFI geometric configuration and traffic characteristics at Bangerter Highway and 4100 South Rd. were discussed earlier in this chapter. The geometric design has been obtained by Google Earth application. The signal timing plans are not available for this location, which will require the optimization of the signal timing plans for the CFI and CI to ensure the highest percentage match between the input data and the model output. The traffic characteristics on this location were included in the data collection. Previous information is needed before starting to code CFI and CI. With the aid of VISSIM and images found on Google Earth, it was possible to build two initial models for the location at Bangerter Highway and 4100 S. Rd. The first model was for the Conventional Intersection (CI), using images from 6/17/2010, before that location was converted into a CFI (see figure 43 and 44). The second model was for the Continuous Flow Intersection (CFI), using images from 7/8/2016 after that location was converted into a CFI (see Figure 45 and 46). To test the coded locations and see how the design works, the model run with VISSIM default values and assumed volume and signal timing values. Figure 46 shows the CFI design while it was running. This CFI four-approach has five traffic signals, one at the main intersection and four at the secondary intersections as shown in figure 47. Three signal controllers and only two phases will be needed to operate this intersection (see figure 47). Phase 1, the through and the left-turn traffics of the northbound and southbound at the main intersection proceed simultaneously. At the same phase the through traffic at the north and south secondary intersections and the left turn traffic at the east and west secondary intersections get the green. In the second phase, all remaining movements proceed.

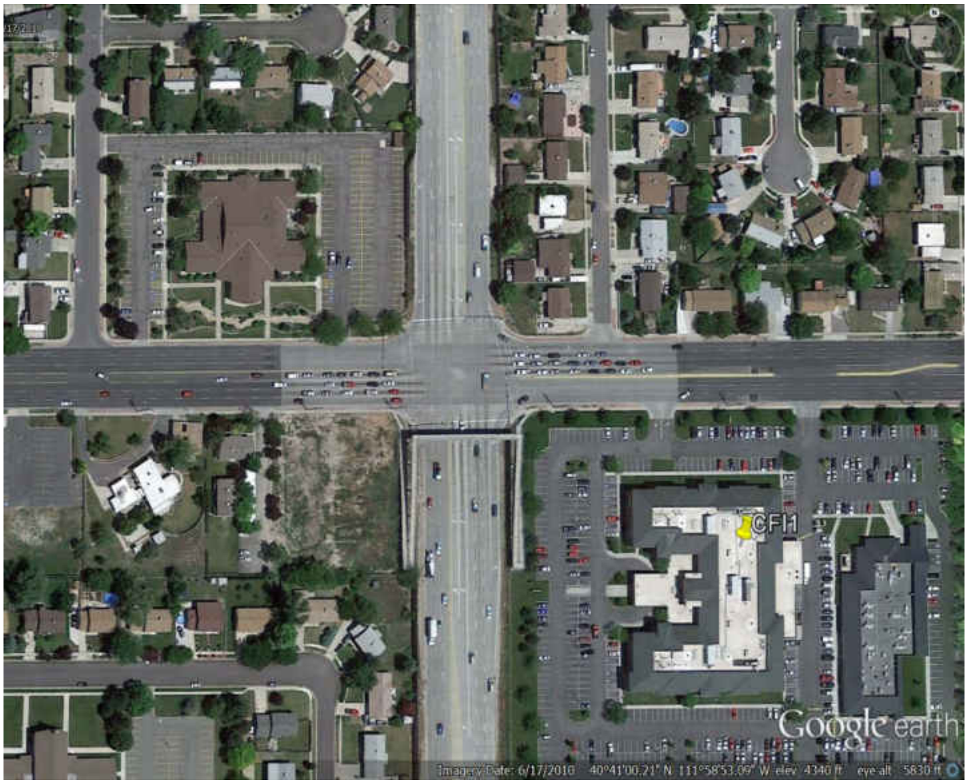


Figure 43 the Conventional Intersection at Bangerter Hwy and 4100 S. Rd, Utah, (Google Earth, 2016)

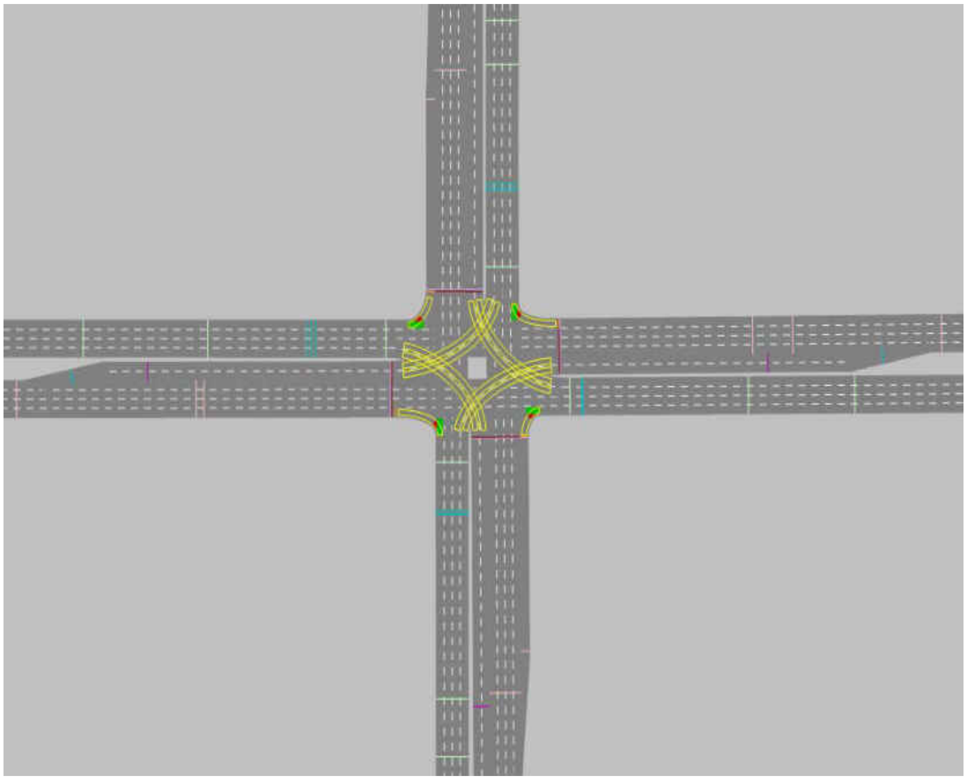


Figure 44 the Coded CI at Bangerter Hwy and 4100 S. Rd, Utah,

Figure 45 the Continuous Flow Intersection (CFI) at Bangerter Hwy and 4100 D. Rd, Utah, (Google Earth, 2016)

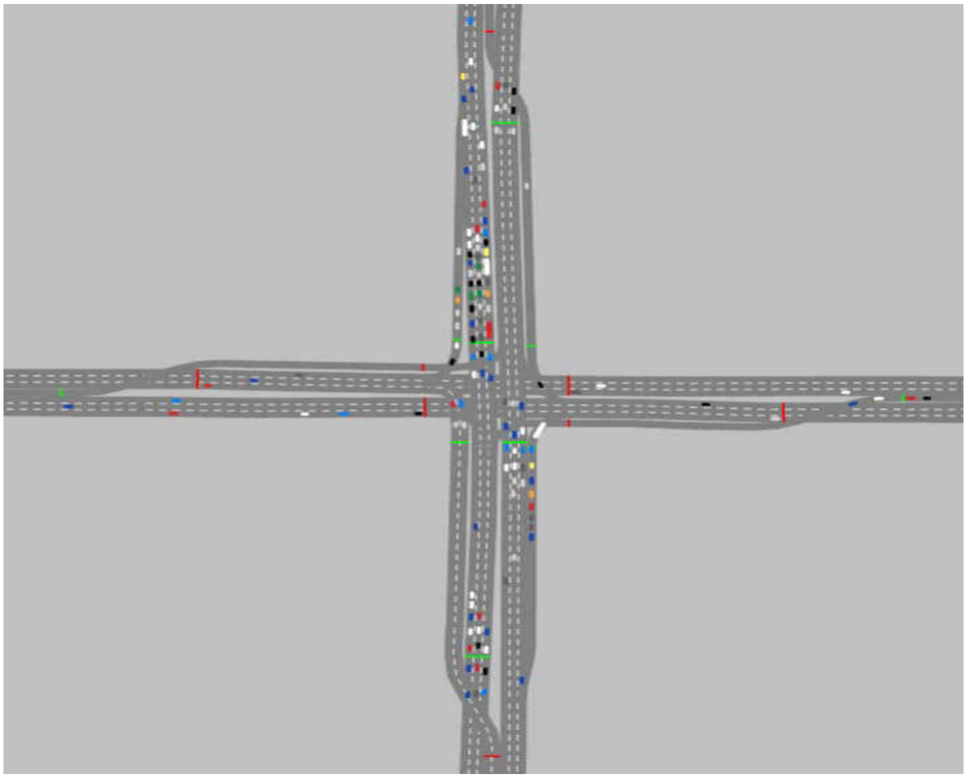


Figure 46 the Coded CFI at Bangerter Hwy and 4100 S. Rd, Utah,

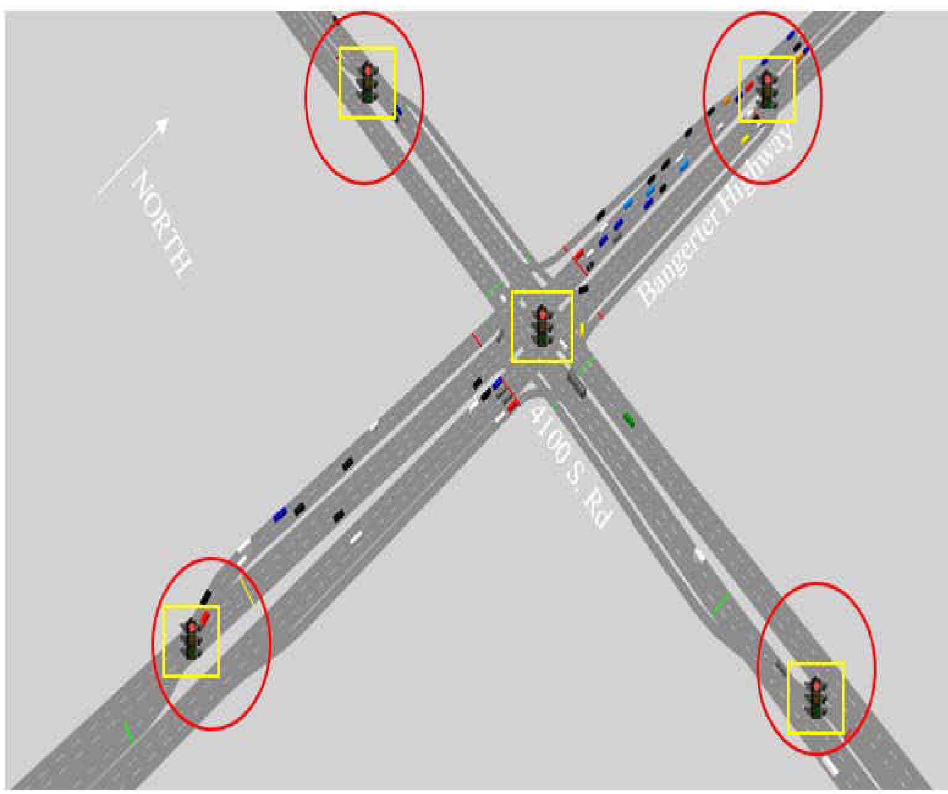


Figure 47 Signals and Phases of the Model for the full-approach CFI at Bangerter Highway and 4100 S. Rd, Utah

5.4 Calibrating the Continuous Flow Intersection

These microsimulation coded models are not considered credible to imitate the existing conditions for the studied designs. The calibration and validation of these models are crucial in order to make them more precise in simulating these innovative designs. There are many characteristics of each intersection design, lane geometrics, TMC, O-D matrices, driver behaviors, transition area and signal timing that need to be taken into consideration while calibrating and validating the models. The calibration was obtained through several steps to ensure the model outputs are a 95% or higher match to field data. There are numerous simulation parameters that need to be taken into considerations while simulating, calibrating and validating the CFI and CI designs. One of these parameters is the simulation period. In this study, 60 minutes was used as the simulation period and it was the most common period in previous studies plus 15 minutes in the beginning to warm up and ensure system is fully operational and to simulate the real life. In order to produce reliable simulation outputs, the models were run using varying replication and seeding numbers (Kim et al., 2007; Olarte and Kaisar, 2011). The models should be run using varying replication and seeding numbers, however one replication number was enough for this study because of the factorial design. All these parameters will ensure a reliable model in the analysis. These parameters were identified by previous literature and tested while running the coded models. There are more parameters that were used to calibrate the model, using field data to ensure produced models are a 95% or higher match to the model outputs - such as the signal timing plans and driving behavior parameters. As signal timings of the existing location were not available, a signal optimization step was necessary while calibrating the models.

5.4.1 Signal Plans Optimization

The signal timing plans were not available, and it was necessary to optimize the signal timing plans manually - using VISSIM to ensure the maximum percentage match between field data and model outputs. Signal-plan optimization was done through running the simulation models, using different cycle lengths and different splits and paying attention to the delay-time and capacity. Taking into consideration these two parameters and comparing each simulation run against another, the signal time splits with the shortest delay and highest capacity were picked. For the CI, 5 signal timings were picked - out of 19 different signal timing splits for different cycle length (60, 75, 80, 90, and 120 seconds). When comparing them all together, the best split showed an 85% match to real life. That signal timing had a cycle length of 90 second and split 50 second to the NB/SB split equal to 25 seconds for the through and 25 seconds for the left-turn movement and 40 seconds for the WB/EB split equal to 20 seconds for the through and 20 seconds for the left-turn (Hummer, 1998; Olarte and Kaisar, 2011) *(see figure 48)*. As for CFI, 4 signal timings were picked out of 6 different signal timing splits and when comparing them all together, the top performing signal split showed a 97% match to real life. The signal timing used for the CFI had a 60 second cycle, split equal to 30 second and 30 second (see figure 49). Phase 1 will clear the through and left-turn traffics at the North and South bound approaches. Phase 2 will clear the other movements at the East and West bound approaches. The cycle lengths for the signal timing and its splits for both CFI and CI designs were shown in Appendix [C]. The top performing signal timing plans for both models were used to perform the driving behavior parameters' sensitivity analysis in next section.

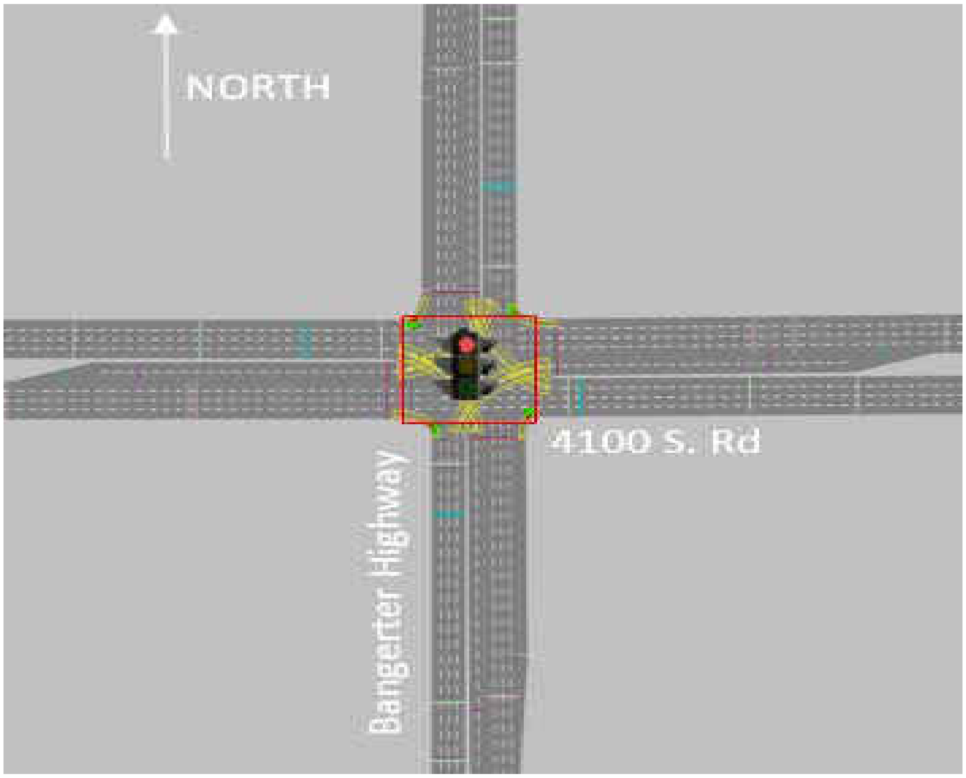


Figure 48 Signal Controller for the Conventional Intersection at Bangerter Hwy and 4100 S. Rd, Utah

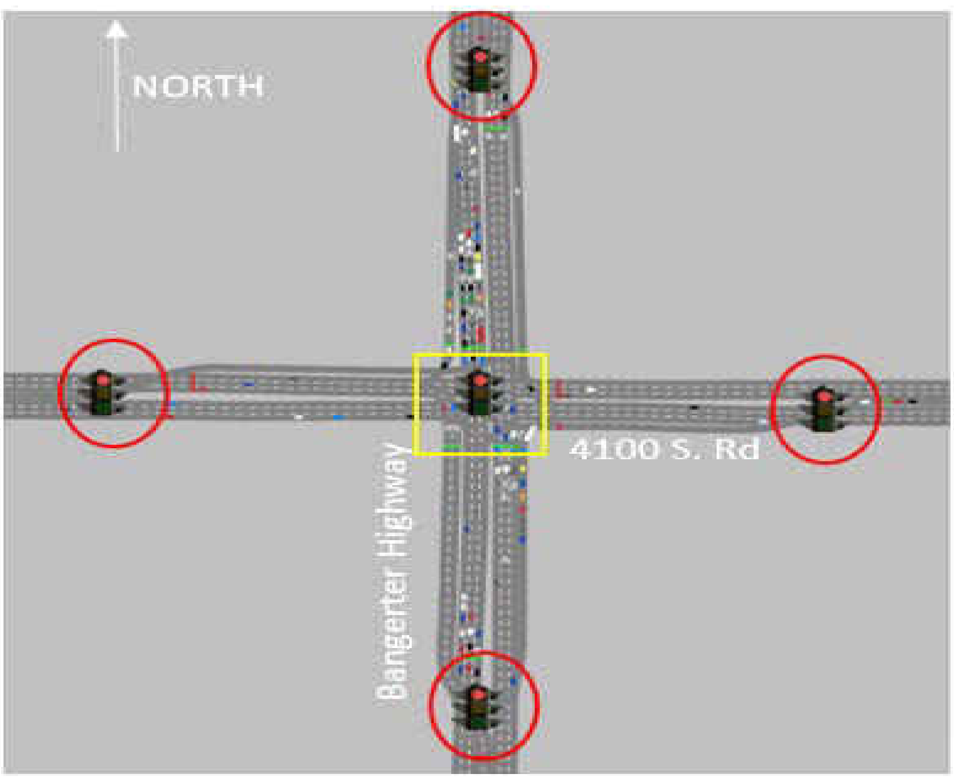


Figure 49 Signal Location for the CFI at Bangerter Hwy and 4100 S. Rd, Utah

5.4.2 Driving Behavior Parameters' Sensitivity Analysis

In VISSIM, there is a need to choose a set out of ten driver behavior parameters and run the model under different parameter values and, then, select the best set of parameters to ensure a 95% or more match between the input demands and model outputs. Those parameters are under the Wiedemann 99 which are CC0: Standstill distance, CC1: Headway Time, CC2: Following Variation, CC3: Threshold for Entering 'Following' State, CC4: Negative 'Following' Threshold, CC5: Positive 'Following Threshold', CC6: Speed Dependency of Oscillation, CC7: Oscillation Acceleration, CC8: Standstill Acceleration, and CC9: Acceleration at 50 mph (Tarko et al., 2008). For the driving behavior parameters, a sensitivity analysis was conducted for different levels using the optimal signal timing for both CFI and CI designs. The driving behaviors that varied were the Wiedemann 99 parameters CC0, CC1, CC2, CC7 and CC8. The CC0, which is responsible for the standstill distance (Manjunatha et al., 2013; Toledo et al., 2003). The CC1, responsible for the headway time and the CC2, which is responsible for the following variation. The CC7 is responsible for the oscillation acceleration. The CC8 is responsible for the standstill acceleration. These 5 parameters have the highest effect on the model's performance (Tarko et al., 2008; Russo, 2008) *(17, 18)*. A simulation run was completed as each parameter has varied while the rest remained in their default values and the change in throughput was recorded for each run. The parameters varied by setting two higher and lower points around the default values, which resulted in 25 different simulation runs (see table 14). Once the optimal driving behavior parameter set was identified, the models were run under this set of parameters and both matched the field data and model outputs within 95 % and higher.

Table 14Wiedemann 99 Parameters Sensitivity Analysis Scenarios

Wiedemann 99 Parameters				
CC0	**CC1**	**CC2**	**CC7**	**CC8**
1.64	0.7	6.56	0.49	4.92
3.28	0.8	9.84	0.66	8.2
4.92	0.9	13.12	0.82	11.48
6.56	1	16.4	0.98	14.76
8.2	1.1	19.69	1.15	18.04

The value for each parameter that had the highest positive impact on the throughput was picked and, then, a final simulation run was completed using all the new values. For the CI, the new Wiedemann 99 parameters values were:

- CC0 = 1.64 ft
- CC1 = 0.7 sec
- CC2 = 6.56 ft
- CC7 = 0.66 ft./s^2
- CC8 = 14.76 ft./s^2

That led to a capacity increase, from 85% to 95% for CI. As for CFI, and variation in the wiedemann 99 parameter, the capacity has either changed negatively or remained the same, leading to the use of default values:

- CC0 = 4.92 ft.
- CC1 = 0.90 sec
- CC2 = 13.12 ft.
- CC7 = 0.82 ft./s^2
- CC8 = 11.48 ft./s^2

5.5 Measures of Effectiveness

In order to search for conditions that make a CFI design better than a CI design, it was deemed necessary to design an experiment that would encompass critical measures of effectiveness. The measures of effectiveness that were used in previous studies have included: vehicle-trips, total delay, moving/total time, delay per vehicle, average speed, storage, phase failure, fuel, HC emissions, NOX emissions, % demand, operational and safety performance, average control delay, number of stops, partial and overall capacity, delay for all movements and CO emissions (Abou-Senna and Radwan, 2016; Hummer and Reid, 2000; Olarte and Kaisar, 2011; Autey et al., 2013). The measures of effectiveness selected for the experiment are the delay time and the intersection's capacity. The average delay per vehicle along with the capacity are two of the most used measures of effectiveness in past studies. Using these two measures to compare the locations before and after conditions would allow a better understanding of the conditions that justify the conversion, from a CI to a CFI.

5.6 Experimental Design

The experiment included a multi-level factorial design that looks at changes of multiple factors and compares the results using measures of effectiveness (Lownes and Machemehl, 2006; Autey et al., 2010). Five main parameters were considered in the experimental design based on literature review that proved their effect on CFI performance. The parameters included: the spacing between the main and secondary intersection, number of lanes for the left and through movements, adjacent intersection distance and volume per hour, per lane. The experiment resulted in 3*2*3*2*5 = 180 scenarios.

The first factor that has varied in the experiment was the spacing distance. In CI case, the spacing distance was defined as the distance that encapsulates the left lane. While in CFI case, it was defined as the distance between the main intersection and the secondary intersection. The spacing distances in the experiment were 500 ft, 700 ft and 900 ft. and were used to identify the effect of spacing distance on CFI and CI designs. The second factor that has varied on the experiment was the number of lanes in the intersection. For each spacing distance used, the number of lanes has changed for different geometric configurations: 1 or 2 left-turn lanes, paired with 2, 3, or 4 through lanes. The NB/SB approaches still had a dedicated right-turn lane, while the WB/EB approaches had one of the through lanes as a shared through and right-turn lane. The third factor that has changed between the scenarios was the distance between the main and adjacent intersections.

For the spacing distance of 500 ft., it was used 1320 ft. and 2640 ft. for each configuration. For the spacing distance of 700 ft., it was used 1535 ft. and 2640 ft. for each configuration. For the spacing distance of 900ft, it was used 1750 ft. and 2640 ft. for each configuration. The distances 1320, 1535, and 1750 were different for each spacing distance because the spacing distance between the main and secondary intersections increased the distance to the adjacent intersection and became insufficient to clear the traffic, resulting in intersection blockage. So, it was necessary to increase the distance between the adjacent intersections. However, the 2640 ft. distance between the adjacent intersections was enough to clear the traffic for all three spacing distances. For each of these scenarios, the theoretical capacity was vehicle per hour, per lane and has varied between 250, 500, 750, 1000 and 1250 vehicles/hour/lane, while allotting 5% of the total volume to the right turners. Each per-lane volume scenario multiplied by the number of lanes, per approach resulted in total volume per approach. The distances on which the delay was measured have varied in relation to the distance between the main intersection and adjacent intersection. For an adjacent intersection at 1320 ft., the left-turn delay was measured based on an 800ft distance. For an adjacent intersection at 1535 ft., it was used 1200 ft. to measure the left-turn delay. For an adjacent distance of 1750 ft., the left-turn delay was measured based on a 1470ft distance. As for all the configurations with 2640 ft. of adjacent distance, the same distances were used for their shorter counterparts were used for them. As the adjacent distance increases, the distance to measure the delay increases, which results in the distance variation between the three scenarios.

During the design of each experiment, a balanced and an unbalanced condition was considered. The unbalanced condition means the volume per lane for the minor road is a percentage (25, 50 and 75%) of the volume, per lane of the major road and the balanced condition means the same volume, per lane used for the four approaches. In order to come up with a conclusive study, the unbalanced condition was first tested, through multiple runs at different volumes. The unbalanced conditions did not show any significant advantage over the balanced condition, as the capacity of each unbalanced condition was close to each other over the varying volume. The experiment proceeded using only the balanced conditions. Table 15 summarizes the design experiment, which was carried out for both CI and CFI. Each scenario on the table was developed by VISSIM and run to detect the tipping point whenever CFI is superior to CI.

Table 15 The Designed Experiment for The Continuous Flow Intersection

Senario Group	Sub Group	Sub Group Iteration	Senario No.	Spacing Distances, (ft)	# of Lanes LT	# of Lanes Thru	Adjacent Intersection Distance (ft)	250	500	750	1000	1250
								Total volume, (Veh/hr)				
1	1	1.1	1	500	1	2	1320	790	1579	2368	3158	3947
		1.2	2	500	1	2	2640					
	2	2.1	3	500	1	3	1320	1053	2105	3158	4211	5263
		2.2	4	500	1	3	2640					
	3	3.1	5	500	1	4	1320	1316	2632	3947	5263	6579
		3.2	6	500	1	4	2640					
	4	4.1	7	500	2	2	1320	1053	2105	3158	4211	5263
		4.2	8	500	2	2	2640					
	5	5.1	9	500	2	3	1320	1316	2632	3947	5263	6579
		5.2	10	500	2	3	2640					
	6	6.1	11	500	2	4	1320	1579	3158	4737	6316	7895
		6.2	12	500	2	4	2640					
2	1	1.1	13	700	1	2	1535	790	1579	2368	3158	3947
		1.2	14	700	1	2	2640					
	2	2.1	15	700	1	3	1535	1053	2105	3158	4211	5263
		2.2	16	700	1	3	2640					
	3	3.1	17	700	1	4	1535	1316	2632	3947	5263	6579
		3.2	18	700	1	4	2640					
	4	4.1	19	700	2	2	1535	1053	2105	3158	4211	5263
		4.2	20	700	2	2	2640					
	5	5.1	21	700	2	3	1535	1316	2632	3947	5263	6579
		5.2	22	700	2	3	2640					
	6	6.1	23	700	2	4	1535	1579	3158	4737	6316	7895
		6.2	24	700	2	4	2640					
3	1	1.1	25	900	1	2	1750	790	1579	2368	3158	3947
		1.2	26	900	1	2	2640					
	2	2.1	27	900	1	3	1750	1053	2105	3158	4211	5263
		2.2	28	900	1	3	2640					
	3	3.1	29	900	1	4	1750	1316	2632	3947	5263	6579
		3.2	30	900	1	4	2640					
	4	4.1	31	900	2	2	1750	1053	2105	3158	4211	5263
		4.2	32	900	2	2	2640					
	5	5.1	33	900	2	3	1750	1316	2632	3947	5263	6579
		5.2	34	900	2	3	2640					
	6	6.1	35	900	2	4	1750	1579	3158	4737	6316	7895
		6.2	36	900	2	4	2640					

5.7 Analyses and Results

The output from the simulation runs were, then, used to evaluate the conditions that warrant a CFI design. The analysis focused on the two measures of effectiveness which were the NB left-turn (LT) delay and NB LT capacity. Table 17 was used as reference for this analysis of group 1. The analyses were divided into three groups based on the spacing distance. Each group was also divided into two sub-groups based on the adjacent intersection distance. All results for each group are documented in Appendix [D].

Looking at the results from Scenario group 1, comparing iterations 1.1, 2.1 and 3.1 regarding NB LT delay and NB LT capacity, CFI outperforms CI (see Figure 50). When comparing the results for 4.1, 5.1 and 6.1 from group 1, CFI outperforms CI regarding delay time in iterations 5.1 and 6.1, 4.1. On the other hand, CFI outperformed CI at the first two volume levels. However, CI and CFI had similar performance regarding NB LT capacity. When comparing the results for 1.2, 2.2 and 3.2 from group 1, CFI outperformed CI with respect to NB LT delay and NB LT capacity. When comparing the results for 4.2, 5.2 and 6.2 from group 1, both CFI and CI designs performed the same with respect to NB LT capacity. As for NB LT delay, in 4.2 and 5.2 scenarios, CI and CFI performed the same at volume level 750 vehicle, per hour, per lane and higher, while CFI outperformed CI in the 6.2 iteration. Results from 1.1, 2.1, 3.1, 4.1, 5.1 and 6.1 were very similar to their counterparts, except for 5.1 and 5.2 regarding NB LT delay (Tarko et al., 2008).

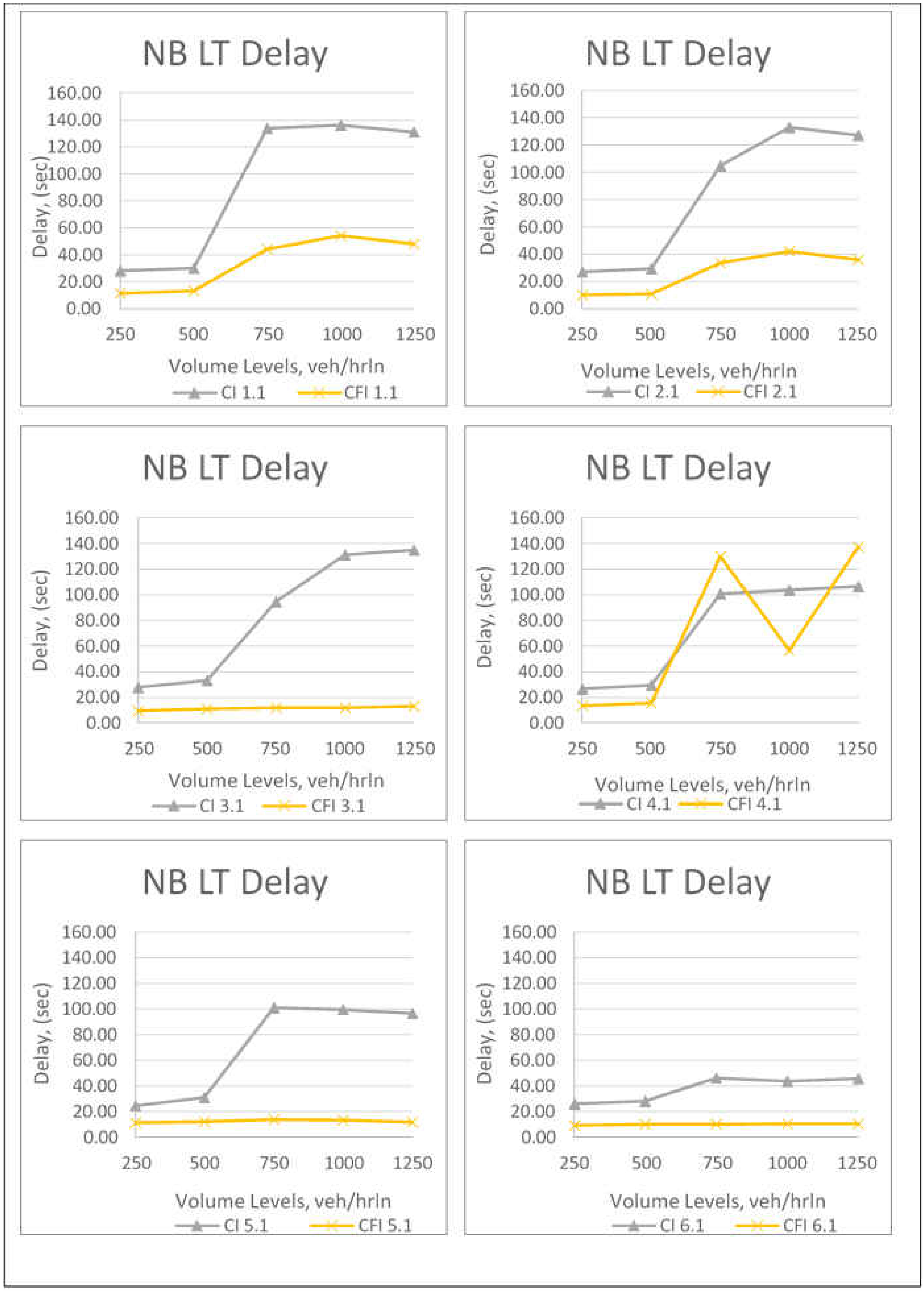


Figure 50 Different Delay Scenarios for Group 1 with 1320 ft Adjacent Distance (CI vs CFI)

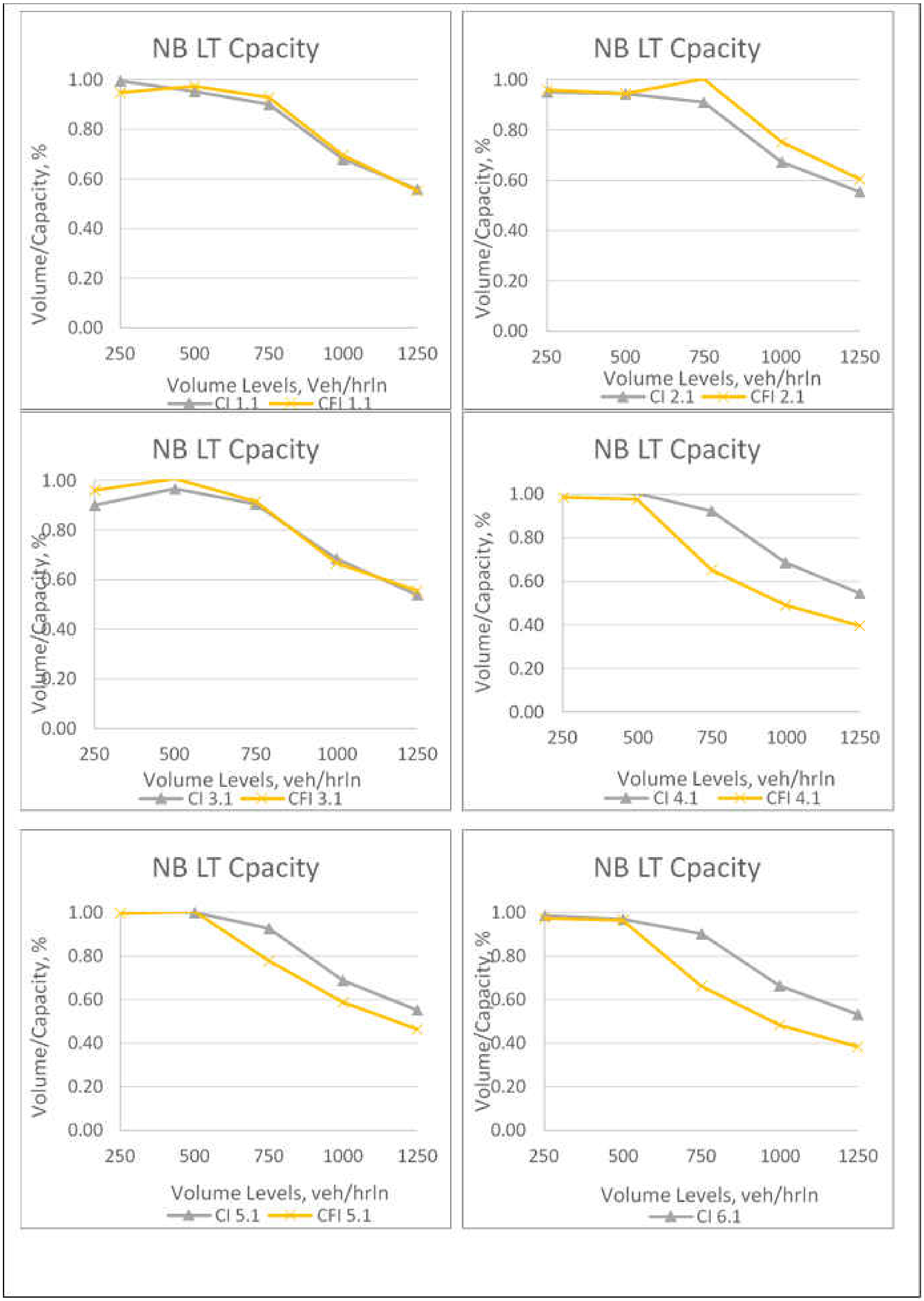


Figure 51 Different Capacity Scenarios for Group 1 with 1320 ft Adjacent Distance (CI vs CFI)

Comparing results for iterations 1.1, 2.1 and 3.1 from group 2, CFI outperformed CI in terms of delay and capacity. When comparing results for 4.1, 5.1 and 6.1 from group 2, with respect to NB LT delay, 5.1 and 6.1 iteration showed that CFI outperformed CI, while in 4.1 there was no significant difference between the two designs. With respect to NB LT capacity, both CFI and CI designs performed the same. When comparing iterations 1.2, 2.2 and 3.2, regarding NB LT delay and NB LT capacity CFI outperformed CI. When comparing the results for iterations 4.2, 5.2 and 6.2, regarding delay, 4.2 and 5.2 show that both designs had the same performance when they reached 750 vehicles, per hour, per lane; however, CFI outperformed CI in 6.2. Regarding NB LT capacity, CI and CFI performed the same in most 4.2, 5.2 and 6.2 scenarios (see figure 52 and 53). When looking at group 2 iterations which is like group 1 results, CFI outperformed CI with single left-turn lane in most scenarios with respect to the delay. CFI outperformed CI in terms of capacity for most the iterations with single left-turn lane; however, there was no significant difference between CI and CFI performances with double left-turn lanes, which could be attributed to the signal optimization and the coordination between the main and secondary intersections. What's more, the balanced approach may have contributed to this insignificance between CI and CFI due to the fact that the intersection is heavily congested at the 750 vphpl volume level, on all four approaches.

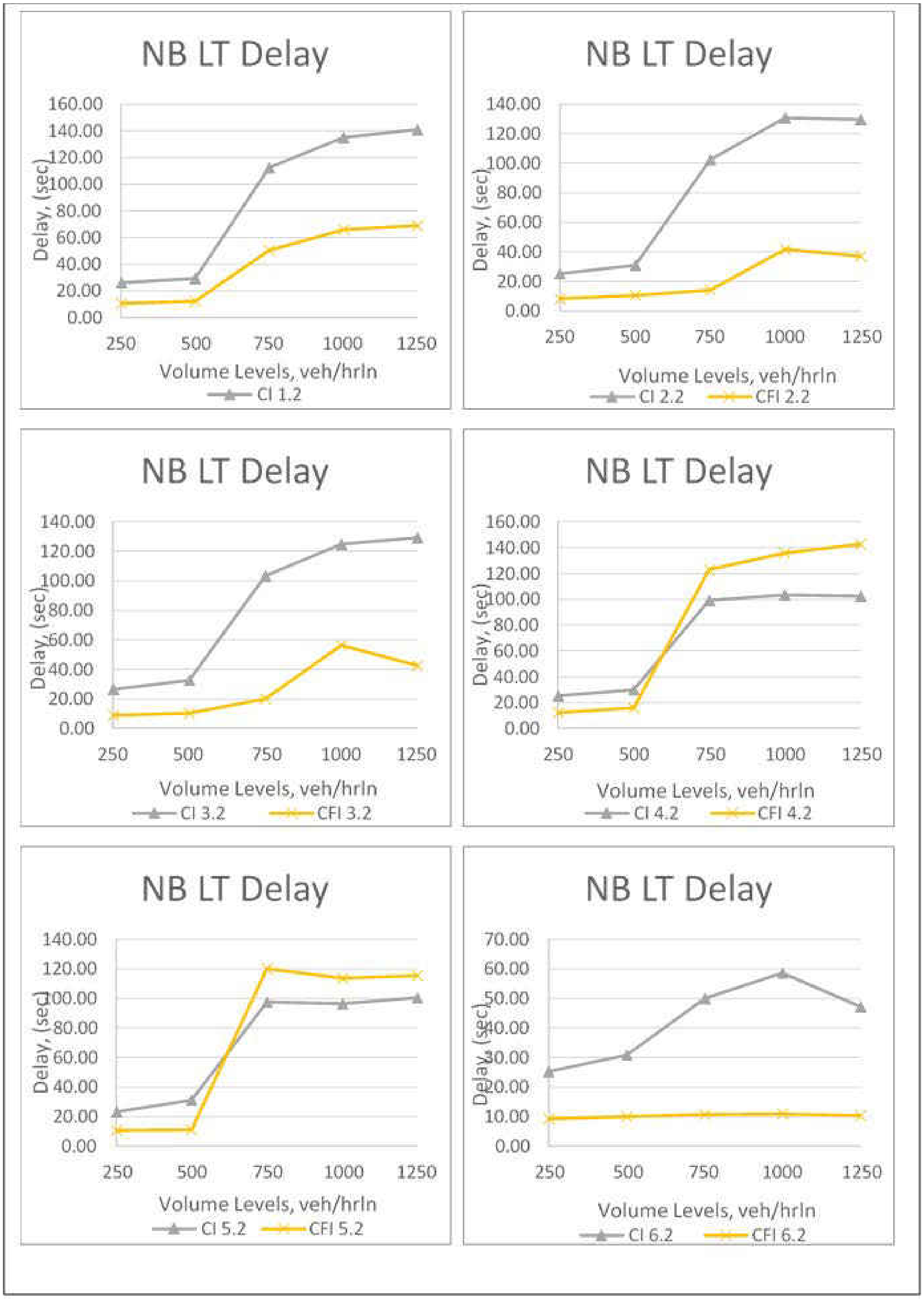


Figure 52 Different Delay Scenarios for Group 2 with 2640 ft Adjacent Distance (CI vs CFI)

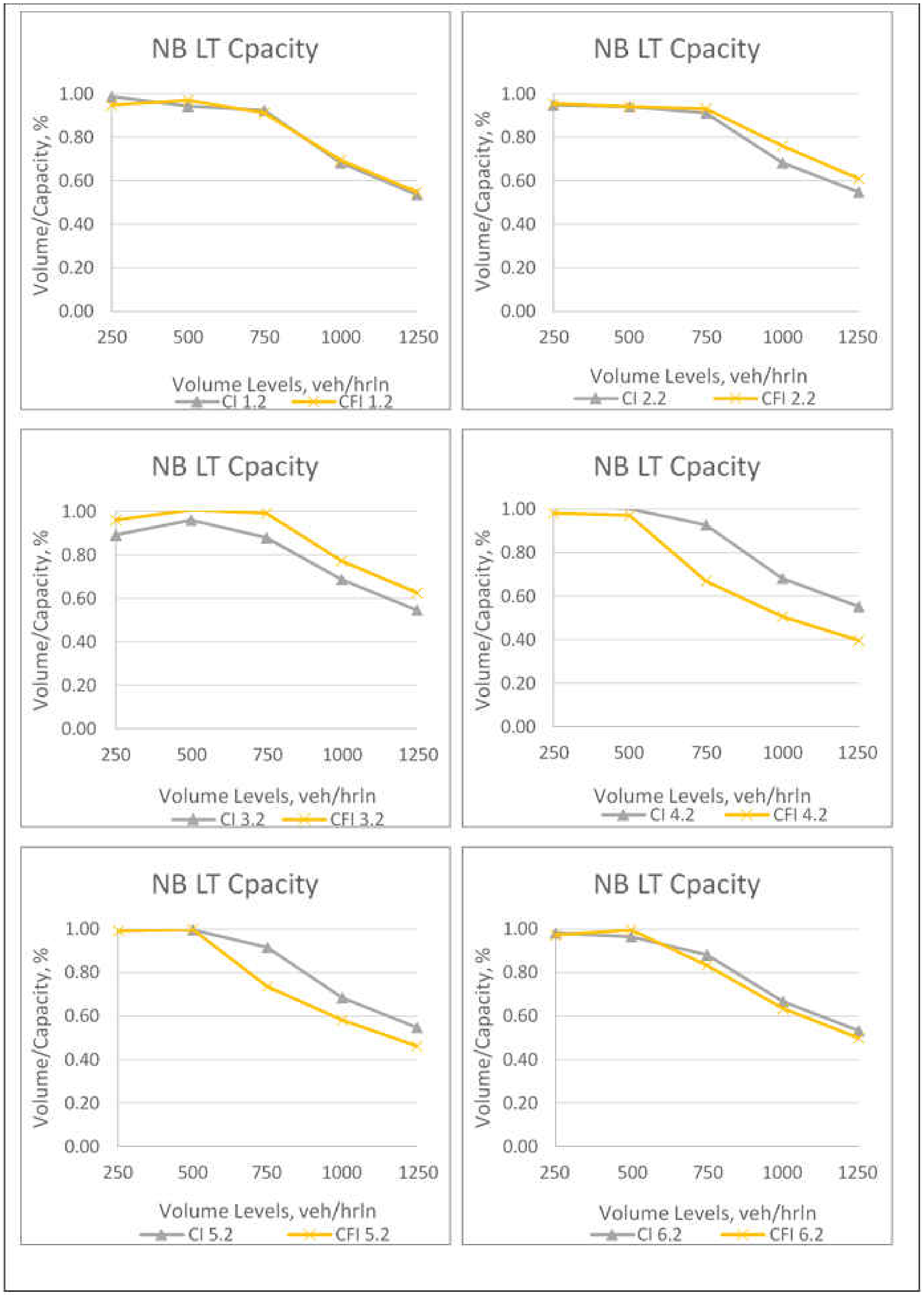


Figure 53 Different Capacity Scenarios for Group 2 with 2640 ft Adjacent Distance (CI vs CFI)

The results for iteration 1.1, 1.2, 2.1, 2.2, 3.1 and 3.2, from group 3 all show that CFI outperformed CI regarding all NB LT delay and NB LT capacity. When looking at iterations 4.1, 5.1 and 6.1, from group 3, regarding NB LT delay in 4.1, there was no significant difference between the two design performances when reached 750 volume, per lane or more, due to the reasons mentioned above, while in 5.1 and 6.1 CFI outperformed CI. Both CFI and CI designs had no significant differences regarding LT capacities, for all three iterations. The results for scenarios 4.2, 5.2 and 6.2, from group 3, regarding NB LT delay, showed that in 4.2 and 5.2, the two designs had the same delay when they reached 750 vehicle per hour, per lane while in 6.2 CFI outperformed CI. In all three iterations the CI performed similarly to CFI with respect to NB LT capacity. The only difference between 1.1, 2.1, 3.1, 4.1, 5.1 and 6.1, if compared to their counterparts was that iterations 5.1 and 5.2 regarding NB LT delay. CFI LT delay and capacity performed better than CI on all single left-turn lane scenarios; however, there was no significant difference between the two designs' performances at the high-volume scenarios, which is attributed to the balanced approach effect and the signal optimization and coordination between the main and secondary intersections as all other parameters were constant. All the analysis charts, for all groups and scenarios, are documented in Appendix [D].

CHAPTER 6: CONCLUSION AND DISCUSSION

The purpose of this dissertation is to better understand the Diverging Diamond Interchange (DDI) and Continuous Flow Intersection (CFI) designs - as well as a range of factors that affect signalized intersections and freeway-interchanges' performance due to increased left-turn traffic volume and assess the need and justification to redesign intersections and interchanges to improve their efficiency and safety. And to that end, an extensive literature review on existing studies was carried out in order to understand the principles of these innovative designs and determine the methodology that was followed in the respective research. Accordingly, several locations with DDI and CFI designs were selected to be candidate-locations and the due data collection was performed. In order to simulate these locations, it was necessary to look at different simulation tools that can imitate the innovative designs' configuration. A micro-simulation tool was selected and used to model the selected location before and after CFI implementation. The simulation was, then, complemented with field data to accurately resemble real life conditions through models' calibration and validation. In addition, two measures of effectiveness (MOEs) were identified and used in this study: average delay and capacity. These MOEs are affected by many factors such as geometric characteristics, traffic volume and signal timing plans, which required an experiment for each innovative design. The experiment was specifically designed to evaluate the innovative designs' performance, under several factors and detect the threshold to switch from the conventional design to the innovative design, using the selected MOEs. There are also more parameters they should be considered when considering the DDI design such right-of-way, benefit-to-cost ratio, accessibility, pedestrian and bicycle interaction.

6.1 Summary of Diverging Diamond Interchange

To acquire a better understanding of the Diverging Diamond Interchange (DDI) and several factors that affect the interchange performance, due to increased left-turn demand, two interchanges were selected to be candidate-locations that already have implemented DDI designs and the required data was collected to calibrate and validate the models. VISSIM (version 8.0) is the micro-simulation software that was selected to perform the analysis at microscopic level. This is mainly due to its ability to replicate the innovative design, ability to simulate signal control plans and/or import signal plans from other tools, and the capability of running the simulation for different replications, random seed and other factors. The calibration and validation of the models were done by using field data, under a set of optimized driver-behavior parameters and signal plans. Several signal plans were optimized for both DDI and CDI designs. The 60-second cycle was found as the optimal cycle length for the DDI design, with two phases and the 90-second cycle, with three phases was found as the optimal cycle length for the CDI design. Five driving behavior parameters have been identified by the literature as they have significant effect on the models (Olarte and Kaisar, 2011). A sensitivity analysis was performed to identify the optimal set of values for these five parameters and three parameters had the most influential effect on the DDI design and the other two had their default values. In addition, two measures of effectiveness (MOE) were identified to be used in this study: capacity and average delay. These MOEs are affected by many factors, which led to the design of an experimental design. The experimental design, including a range of volume conditions, geometric designs and signal plans was set and the MOEs were used to reach the study's goal. The simulation models were, then, run using different volume scenarios. The results were, then, analyzed to compare the DDI performance and the conventional design. Furthermore, the results were used to detect the switching tipping points, from the conventional design to the innovative design.

The left-turn delays in all scenarios did not show any cross-point between CDI and DDI, but they concurred with previous literature that stated DDI's performance is better, if compared to CDI (see table 18) (Lownes and Machemehl, 2006). All the left-turn capacity percentages showed the crossing point between CDI and DDI located between the 500 to 750 vehicles, per hour, per lane. As the number of through lane parameter increases, the left-turn delay increases and the efficiency decreases, for both DDI and CDI designs. But CDI is more affected, while DDI is slightly affected. When the distance between the two crossovers is increased, the delay for DDI increases, but it has no effect on the throughput. The analysis showed that the DDI should not be used with location experiencing low left-turn demand (see table 16).

Table 16 Summary of Diverging Diamond Interchange Results

Crossover Distance FT	Volume Per Lane, veh/hr	NB LT Delay, sec/veh						NB LT Capcity, veh					
		Single LT Lane			Double LT Lane			Single LT Lane			Double LT Lane		
		2-TH	3-TH	4-TH	2-TH	3-TH	4-TH	2-TH	3-TH	4-TH	2-TH	3-TH	4-TH
850	500	Yes	Yes	Yes	Yes	Yes	Yes	NO	NO	NO	NO	NO	NO
	750	Yes	Yes	Yes	Yes	Yes	Yes	E	YES	YES	YES	YES	E
	1000	Yes	Yes	Yes	Yes	Yes	Yes	YES	YES	YES	YES	YES	YES
	1250	Yes	Yes	Yes	Yes	Yes	Yes	YES	YES	YES	YES	YES	YES
	1500	Yes	Yes	Yes	Yes	Yes	Yes	YES	YES	YES	YES	YES	YES
1200	500	Yes	Yes	Yes	Yes	Yes	Yes	NO	E	NO	E	NO	NO
	750	Yes	Yes	Yes	Yes	Yes	Yes	E	YES	YES	Yes	Yes	YES
	1000	Yes	Yes	Yes	Yes	Yes	Yes	YES	YES	YES	YES	YES	YES
	1250	Yes	Yes	Yes	Yes	Yes	Yes	YES	YES	YES	YES	YES	YES
	1500	Yes	Yes	Yes	Yes	Yes	Yes	YES	YES	YES	YES	YES	YES
1550	500	Yes	Yes	Yes	Yes	Yes	Yes	NO	E	YES	E	NO	NO
	750	Yes	Yes	Yes	Yes	Yes	Yes	E	E	YES	YES	YES	YES
	1000	Yes	Yes	Yes	Yes	Yes	Yes	YES	YES	YES	YES	YES	YES
	1250	Yes	Yes	Yes	Yes	Yes	Yes	YES	YES	YES	YES	YES	YES
	1500	Yes	Yes	Yes	Yes	Yes	Yes	YES	YES	YES	YES	YES	YES
YES	DDI outperforms CDI												
NO	DDI does not outperform CDI												
E	DDI and CDI are preforming equally												

The research results and findings are guidelines for decision-makers as to when they should consider switching from the conventional interchange design to the innovative design (DDI). The implementation of innovative designs is increasing in the U.S. and most literature points to innovative designs, which are promising alternatives to enhance the operation and safety performance of such designs (Yang et al., 2013; Esawey and Sayed, 2007). Many of these studies have balanced the innovative design in contrast to its conventional design, under different measures of effectiveness. However, designing a simulation-based experiment to find the threshold to switch from the Conventional Diamond Interchange design to the Diverging Diamond Interchange design would be extremely helpful to professionals and decision makers. The experiment examined potential factors, that is: number of left-turn lane, number of through lane, crossover distance and level of volumes. Left-turn delay and capacity were used as main measures of effectiveness to detect the cross-point between the DDI and CDI designs. The cross-point could not be allocated by using the delay; however, DDI outperformed CDI. The left-turn capacity seemed to be the most reliable measure of effectiveness to identify the cross-point and it falls between 500 to 750 vehicles per hour, per lane. In some scenarios, CDI had better capacity at the low volume level, however, as volume level per lane increased, DDI capacity increased by 20 – 35 % and outperformed CDI. The crossover distance parameter did not show any improvement in terms of capacity as it grew, and it did increase the delay (see Figure 54).

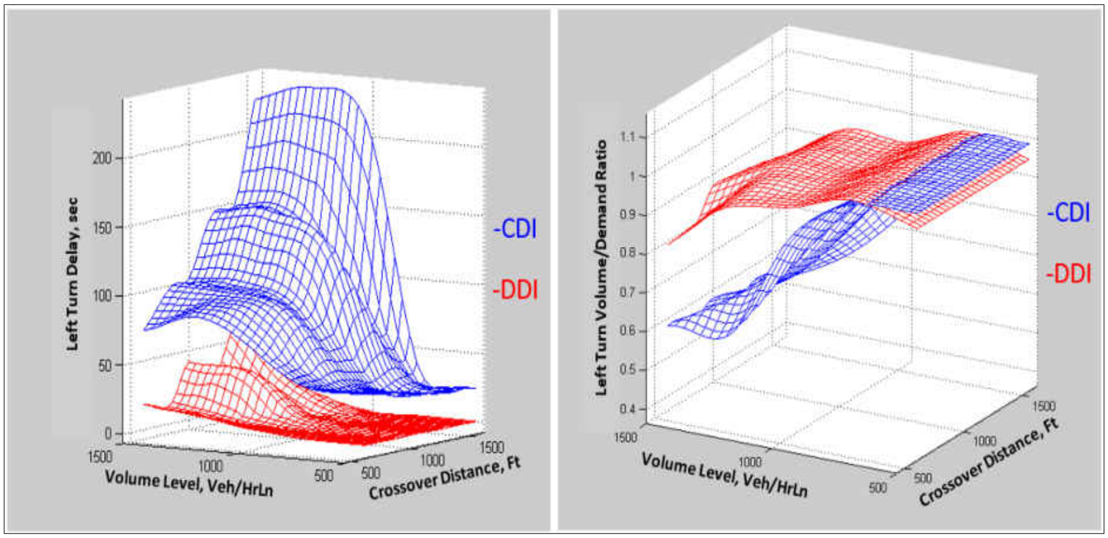


Figure 54Left Turn Delay and Capacity for the CDI vs DDI

6.2 Summary of Continuous Flow Intersection

This study took a closer look at CFI and the numerous factors that affect intersection performance and examined need/justification to redesign the current signalized intersections in order to enhance their operational efficiency. Using these guidelines, traffic engineers would be able to make the optional decision in terms of design (CFI or CI) and meet their operational needs. To build these guidelines, the paper assesses the current strategies for left-turn management, at signalized intersections, and if these strategies meet their intended purpose. It also considers CFI effectiveness regarding operational performance. To do so, locations were carefully selected, and field data was collected. The study also looked at different simulation tools that can imitate the innovative design's configuration. A micro-simulation tool, VISSIM 8, was selected and used to model the selected location, before and after CFI implementation. Using field data, signal optimization and driving-behavior parameter's sensitivity analysis were performed to calibrate the models and replicate real life conditions. In addition to that, an experiment was designed to examine several factors that affect the efficiency of each design. The experiment involved 72 different CFI and CI configurations with 5 different volume levels (180 scenarios for each CFI and CI design) and used two measures of effectiveness, average vehicle delay and capacity to assess the results.

Taking into consideration the results and analyses, the apparent trend seems to be that when comparing a single conventional left-turn lane and a single left-turn CFI, CFI seems to better perform in terms of delay and capacity, if compared to CI. However, there was no significant difference between the double CI left-turn lane and the double CFI left-turn in terms of capacity, for the majority of scenarios. However, when comparing a double CI left-turn and a double CFI left-turn lane, and in terms of delay, CFI seems to outperform CI as the number of through lane increases. The similarity between CI and CFI in some LT capacity results were attributed to the signal optimization and/or coordination between the main and secondary intersections. Also, the balanced approach might cause these fluctuations in the results between CI and CFI capacity results because the same volume, per hour, per lane was assumed for all the four approaches. On top of that, the results show that CFI is improving the delay in most cases, if compared to the other design. The results show that increasing the spacing distance between the main and secondary intersection will increase the delay (see figure 55). The distance between the main intersection and the adjacent intersection seems to have significant effect on the CFI performance. However, when taking queue length into consideration, intersections with longer adjacent distance were able to accommodate the long queue lengths. When looking at iterations 1.1, the trend seems to support past literature that suggests CFI do outperform CI at higher left-turn volumes. The results in this study show that cross-points between CI and CFI capacities happened at a certain volume level range, that is, from 500 to 750 vehicles, per hour, per lane, the range increases as the spacing distance increases with single left-turn scenarios and the difference between CI and CFI delay increases at the same volume range, with the superiority of the CFI design.

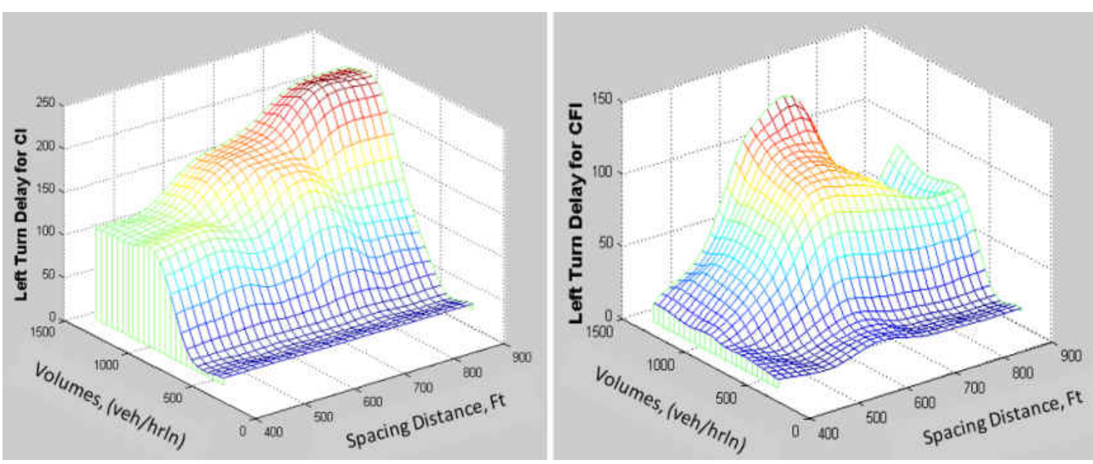


Figure 55 Left-turn Delay for CI and CFI for various Spacing Distance

6.3 Decision Support System

Since the experiments for the continuous flow intersection and diverging diamond interchange (and their conventional counterpart designs) generated 180 and 90 scenarios, respectively, it is necessary to develop an efficient Decision Support System to accelerate the decision, that is, which intersection or interchange design is superior to its conventional counterpart. This system will help decision-makers and professionals to decide if they should switch from the conventional designs to the innovative designs. The system's screen offers input fields that need to be filled with the intersection or interchange design characteristics – which the decision-maker already has and that are based on experiment factors for each design. Figures 56 and 57 show the input fields that need information on CFI and DDI design characteristics. When the characteristics for the intersection or interchange are entered, the results for left-turn delay and capacity will pop up on the screen and the decision will be easier.

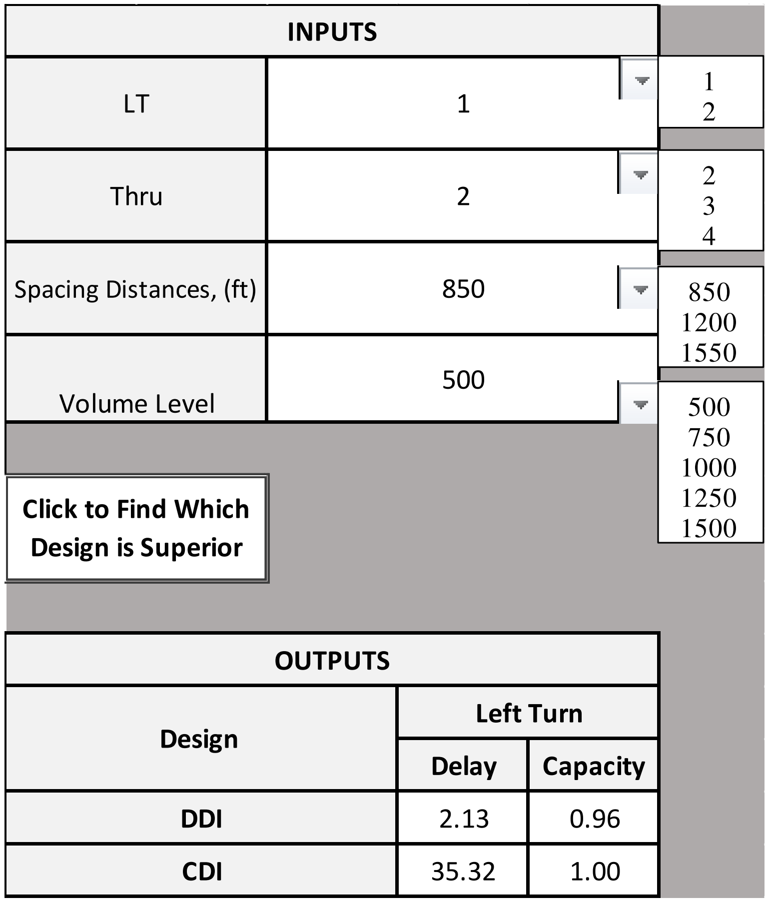


Figure 56 The DDI and CDI Decision Support System Input and Output Screen

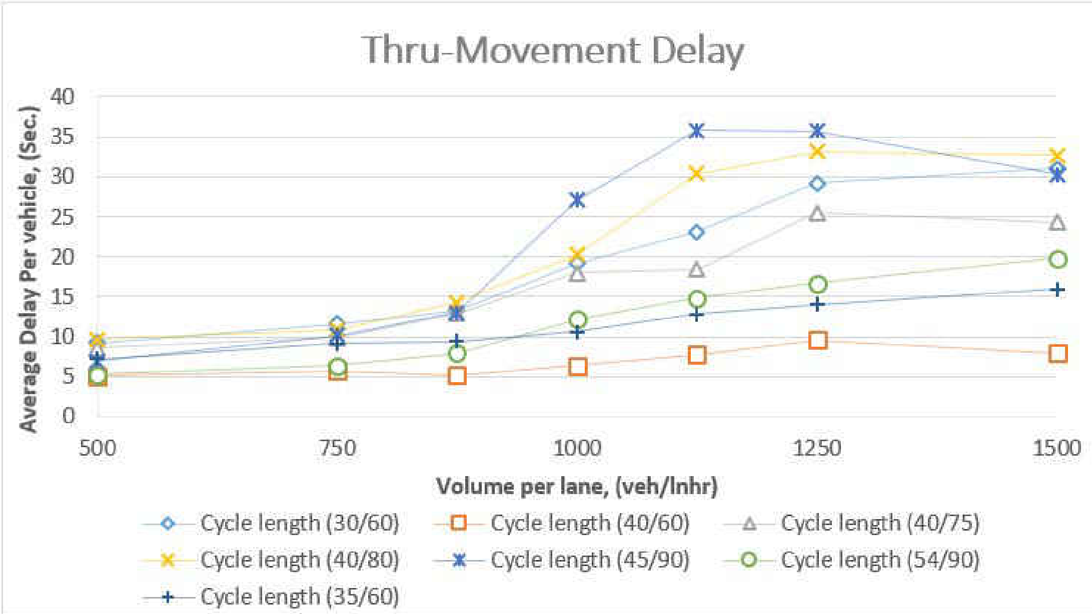
INPUTS		
LT	1	1 2
Thru	2	2 3 4
Adjacent Intersection Distance, (ft.)	1320	1325 1535 1750 2640
Spacing Distances, (ft)	500	500 700 900
Volume Level	250	250 500 750 1000 1250

Click to Find Which Design is Superior

OUTPUTS		
Design	Left Turn	
	Delay	Capacity
CFI	11.48	94.80
CI	28.08	99.60

Figure 57 The CFI and CI Decision Support System Input and Output Screen

APPENDIX A: DIVERGING DIAMOND INTERCHANGE SIGNAL OPTIMIZATION RESULTS



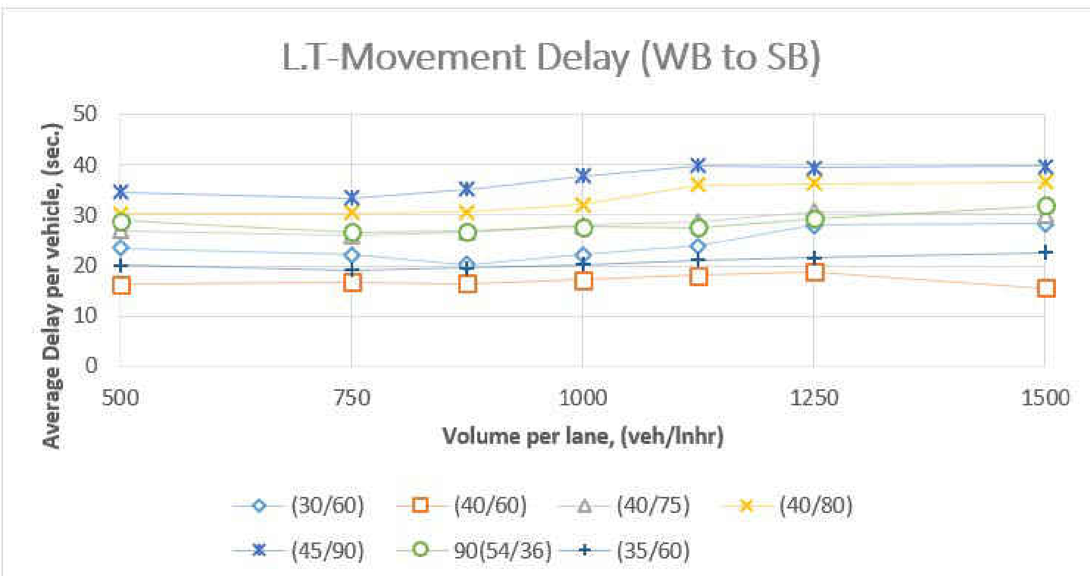
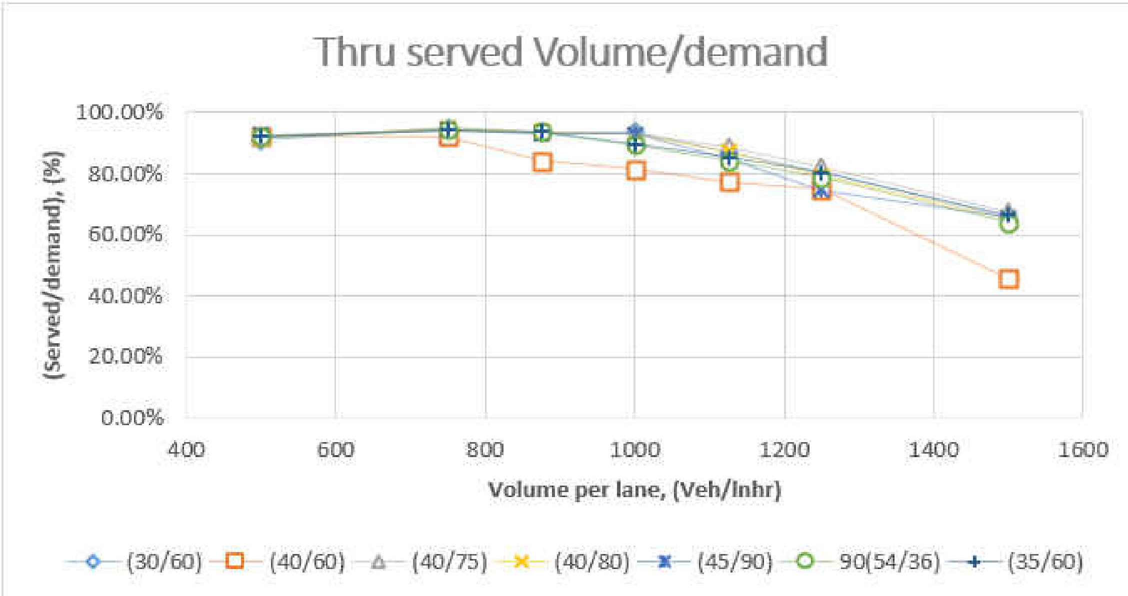


Figure 58 A sample of Diverging Diamond Interchange Signal Optimization Results



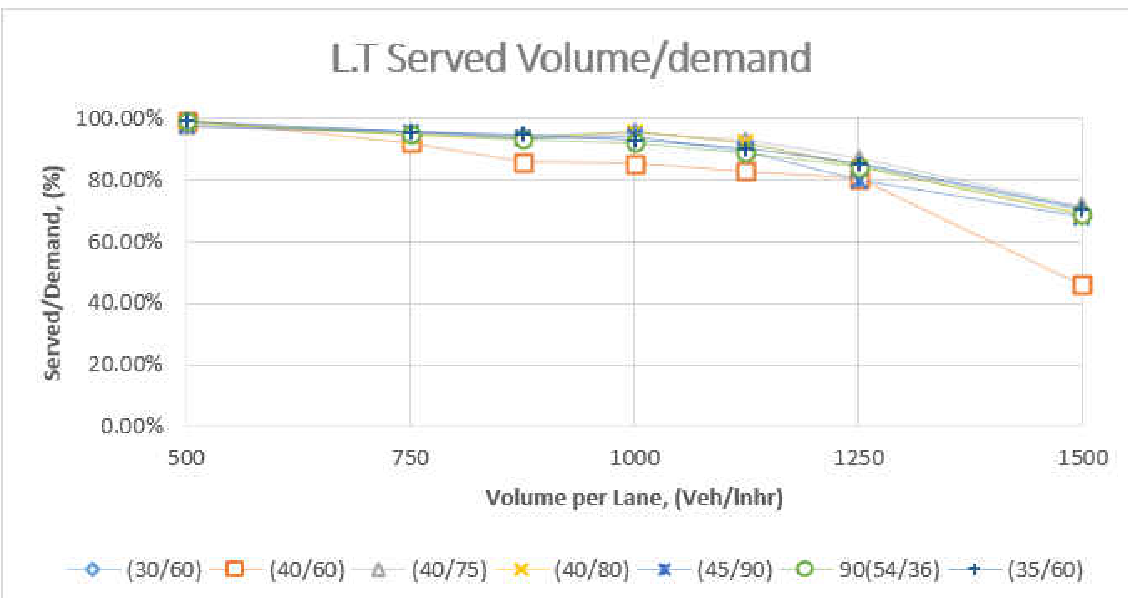


Figure 59 A sample of Diverging Diamond Interchange Signal Optimization Results

APPENDIX B: DIVERGING DIAMOND INTERCHANGE ANALYSIS AND RESULT TABLES AND GRAPHS

Volume/LN	LT Delay/850ft		LT Delay/1200ft		LT Delay/1550ft	
	CDI	DDI	CDI	DDI	CDI	DDI
500	21.32	2.13	29.12	2.02	26.13	1.99
750	37.63	1.95	31.16	2.34	33.86	2.48
1000	89.08	2.39	133.15	2.58	214.15	3.14
1250	97.98	7.6	150.78	4.48	233.85	13.35
1500	99.39	14.76	145.96	25.38	221.17	64.01

Volume/LN	LT Capacity/850ft		LT Capacity/1200ft		LT Capacity/1550ft	
	CDI	DDI	CDI	DDI	CDI	DDI
500	1.00	0.96	1.00	0.96	1.00	0.96
750	0.99	1.00	0.99	1.00	0.98	1.00
1000	0.83	0.95	0.82	0.94	0.76	0.94
1250	0.68	0.94	0.64	0.95	0.61	0.94
1500	0.56	0.80	0.54	0.85	0.52	0.83

Figure 60 A sample of Extracted Data for the DDI and CDI (1LT-2Thru Scenario)

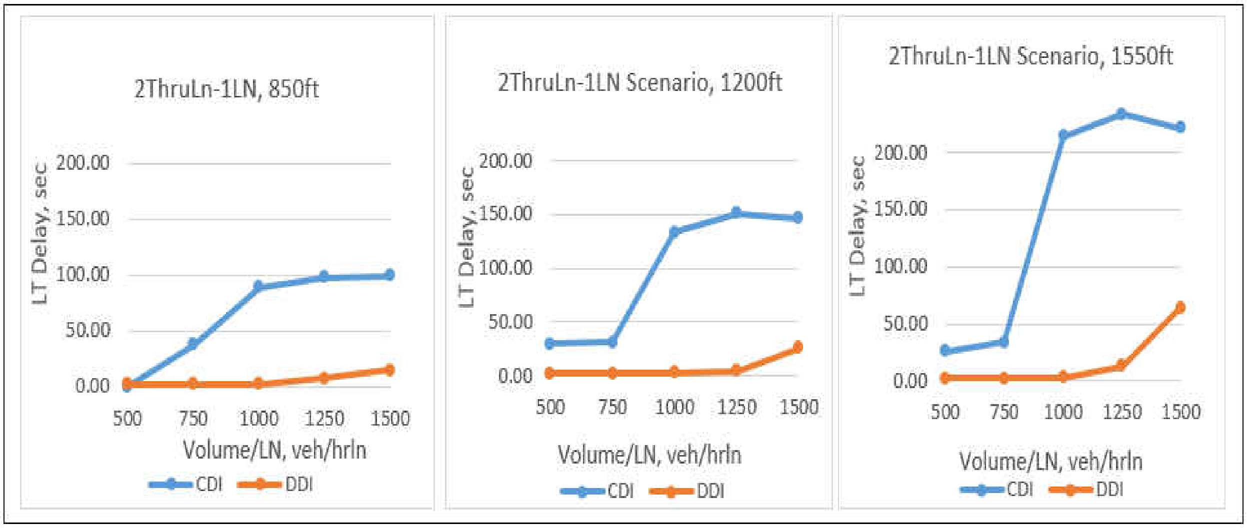


Figure 61A Sample of DDI and CDI LT Delay Graphs at Different Crossover Distance (1LT-2Thru)

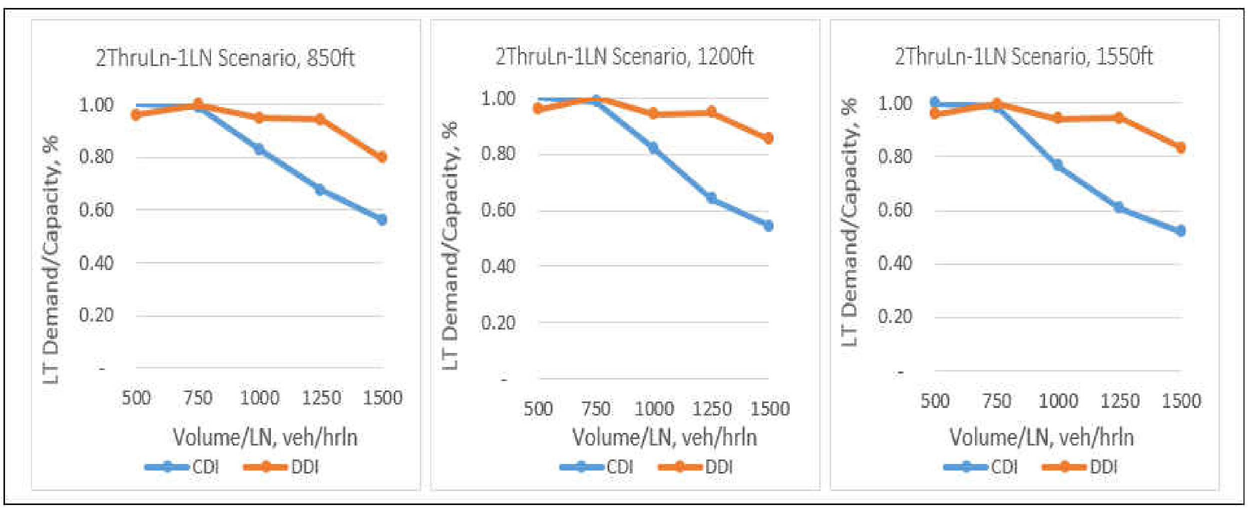
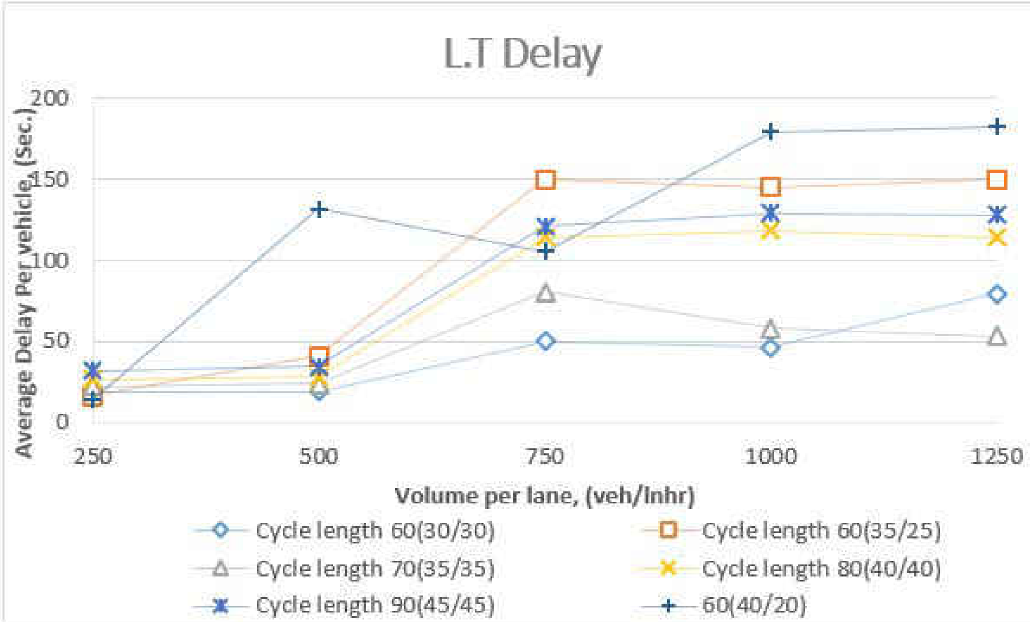


Figure 62 A Sample of DDI and CDI LT Capacity Graphs at Different Crossover Distance (1LT-2Thru)

APPENDIX C: CONTINUOUS FLOW INTERSECTION SIGNAL OPTIMIZATION RESULTS



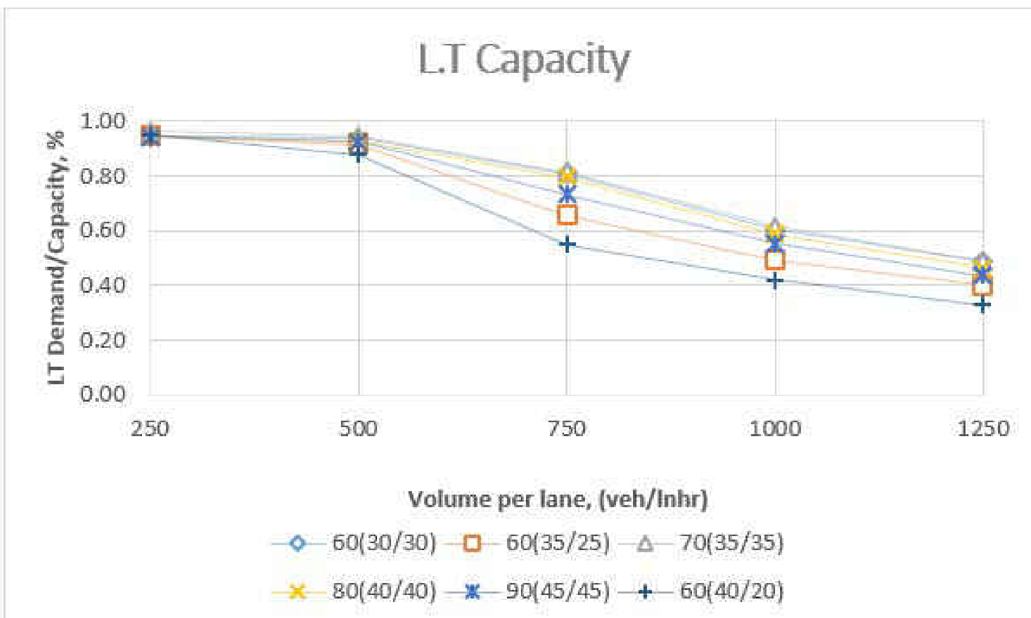


Figure 63 A Sample of the CFI Signal Optimization Graphs

Volume per lane	Cycle length					
	60(30/30)	60(35/25)	60(40/20)	70(35/35)	80(40/40)	90(45/45)
250	18.44	16.62	14.33	22.41	26.61	32.03
500	19.32	40.85	132.07	23.88	28.12	34.52
750	50.31	149.82	105.6	80.78	114.15	121.38
1000	46.64	145.25	179.41	58.22	118.76	129.42
1250	79.3	149.79	182.46	53.15	114.38	127.93

Volume per lane	Cycle length					
	60(30/30)	**60(35/25)**	**60(40/20)**	**70(35/35)**	**80(40/40)**	**90(45/45)**
250	95%	95%	95%	96%	95%	94%
500	94%	92%	88%	94%	93%	93%
750	81%	66%	55%	81%	80%	73%
1000	60%	49%	42%	62%	59%	55%
1250	49%	40%	33%	49%	47%	44%

Figure 64 A sample of CFI Left-turn Delay and Capacity

APPENDIX D: CONTINUOUS FLOW INTERSECTION ANALYSIS AND RESULT TABLES AND GRAPHS

NB LT Delay	CI Scenario											
Volume per lane	1.1	1.2	2.1	2.2	3.1	3.2	4.1	4.2	5.1	5.2	6.1	6.2
250	28.08	25.94	27.13	25.09	27.67	26.46	26.48	25.61	24.67	23.18	26.07	25.26
500	29.89	29.33	29.38	30.92	33.16	32.55	29.67	29.95	30.89	31.08	28.05	30.83
750	133.63	112.29	105.01	102.52	94.63	103.47	100.67	99.08	101.37	97.57	46.08	50.01
1000	136.07	135.11	133.17	130.66	131.24	124.92	103.84	103.21	99.59	96.39	43.65	58.64
1250	131.12	140.84	127.14	129.64	134.86	129.25	106.50	102.44	96.62	100.24	45.83	47.21

NB LT ALL Delay	CFI Scenario											
Volume per lane	1.1	1.2	2.1	2.2	3.1	3.2	4.1	4.2	5.1	5.2	6.1	6.2
250	11.48	10.52	9.94	8.17	9.20	8.94	13.66	12.19	11.50	10.83	9.29	9.26
500	13.25	12.35	10.68	10.37	10.86	10.45	15.66	16.00	12.07	11.09	10.18	10.08
750	43.94	50.20	33.62	14.03	11.91	19.97	130.10	122.92	13.66	120.09	10.28	10.68
1000	54.44	65.68	42.06	41.59	11.75	56.47	56.38	135.70	13.32	113.80	10.37	10.87
1250	47.90	69.12	35.81	36.88	13.08	42.62	137.25	142.69	11.85	115.61	10.37	10.41

Figure 65 A Sample of the CI and CFI Left-turn Delay for 500 ft Spacing Distance

NB LT Cpacity	CI Scenario											
Volume per lane	1.1	1.2	2.1	2.2	3.1	3.2	4.1	4.2	5.1	5.2	6.1	6.2
250	99.60%	98.80%	95.20%	94.80%	90.00%	89.20%	100.00%	100.00%	100.00%	100.00%	98.60%	98.20%
500	95.20%	94.40%	94.40%	94.20%	96.60%	96.00%	100.00%	100.00%	100.00%	99.50%	96.90%	96.50%
750	90.13%	92.27%	91.07%	91.20%	90.53%	88.00%	92.27%	92.67%	92.67%	91.53%	90.27%	88.00%
1000	68.20%	68.20%	67.20%	68.30%	68.40%	68.80%	68.60%	68.25%	68.70%	68.45%	66.25%	66.60%
1250	55.92%	53.68%	55.44%	54.96%	53.84%	54.48%	54.48%	55.16%	55.04%	54.68%	53.24%	53.16%

NB LT Cpacity	CFI Scenario											
Volume per lane	1.1	1.2	2.1	2.2	3.1	3.2	4.1	4.2	5.1	5.2	6.1	6.2
250	94.80%	94.80%	96.00%	95.60%	96.00%	96.00%	98.60%	98.20%	99.60%	99.20%	97.20%	97.20%
500	97.40%	97.00%	94.60%	94.00%	100.80%	100.00%	97.50%	97.10%	100.00%	99.90%	96.30%	99.60%
750	92.93%	91.07%	100.40%	93.07%	91.47%	99.20%	65.27%	67.00%	77.87%	73.47%	66.07%	83.20%
1000	69.50%	69.40%	75.30%	76.00%	66.60%	77.10%	49.05%	50.55%	58.85%	58.10%	48.45%	63.45%
1250	55.20%	54.96%	60.56%	61.04%	55.60%	62.64%	39.56%	39.68%	46.32%	46.16%	38.28%	49.92%

Figure 66 A Sample of the CI and CFI Left-turn Capacity for 500 ft Spacing Distance

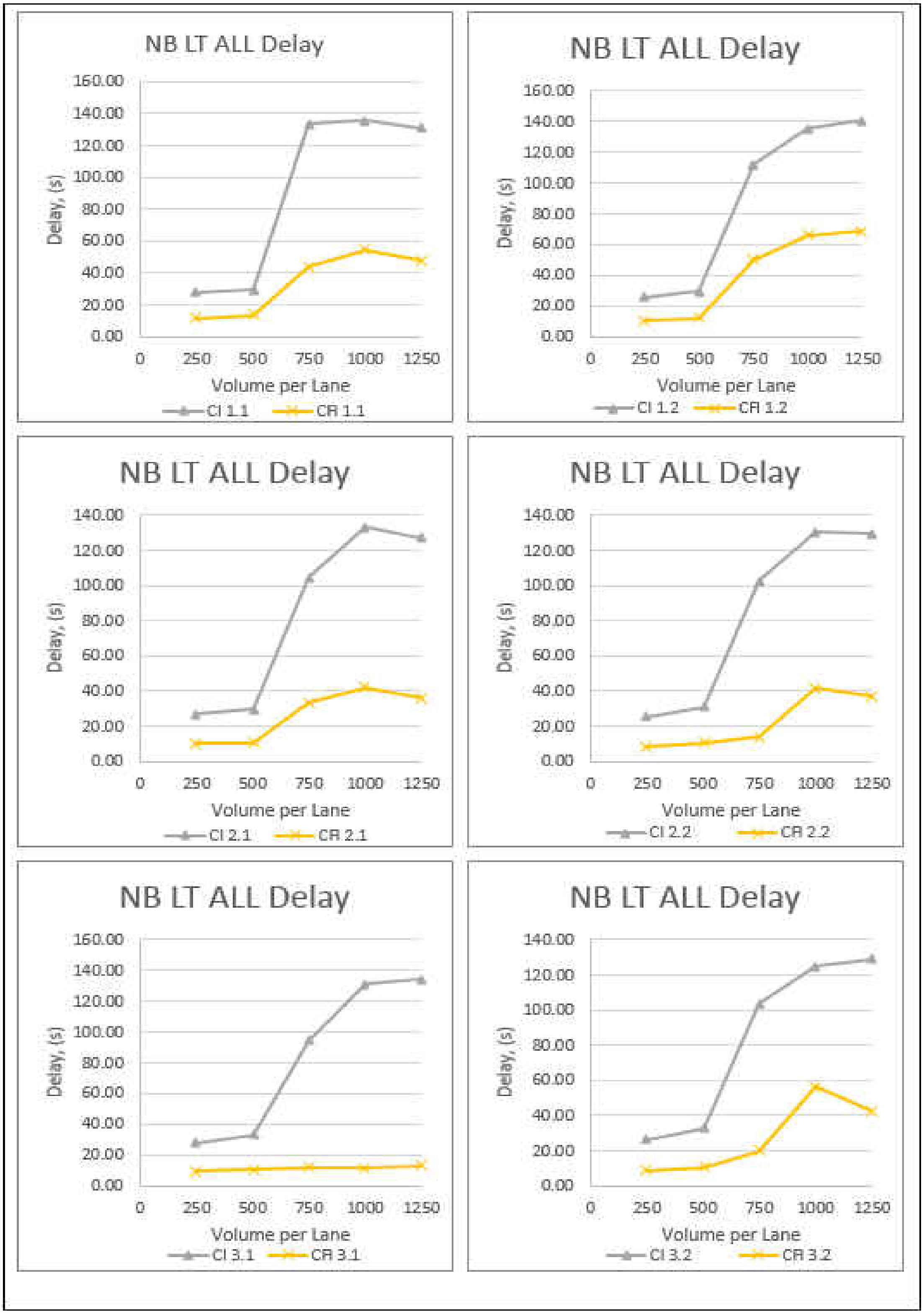


Figure 67 A Sample of the CFI and CI Graphs for 500ft Spacing Distance

APPENDIX E: PRESENTATION AND PUBLICATION

Almoshaogeh, M., Abou-Senna, H., & Radwan, E. (2018), Developing Warrants For Designing Diverging Diamond Interchange. Accepted for presentation at the TRB Annual Meeting, 2018.

REFERENCES

1. Abou-Senna, H., Radwan, E., Tabares, S., Wu, J., & Chalise, S. (2015). Evaluating Transportation Systems Management & Operations (TSM&O) Benefits to Alternative Intersection Treatments
2. Abou-Senna, H., and E. Radwan. "Operational evaluation of partial crossover displaced left-turn (XDL) versus full XDL intersections." Journal of Advances in Transportation Studies Volume 2, PP 27-40 (2016)
3. Autey, Jarvis, Tarek Sayed, and Mohamed El Esawey. "Guidelines for the use of some unconventional intersection designs." 4th International Symposium on Highway Geometric Design, Valencia, Spain. 2010
4. Autey, Jarvis, Tarek Sayed, and Mohamed El Esawey. "Operational performance comparison of four unconventional intersection designs using micro-simulation." Journal of Advanced Transportation 47.5 (2013): 536-552
5. Al-Kaisy, A. F., and J. A. Stewart. "New approach for developing warrants of protected left-turn phase at signalized intersections." Transportation Research Part A: Policy and Practice 35.6 (2001): 561-574
6. Bared, Joe, Praveen Edara, and Ramanujan Jagannathan. "Design and operational performance of double crossover intersection and diverging diamond interchange." Transportation Research Record: Journal of the Transportation Research Board 1912 (2005): 31-38
7. Bared, Joe. Double Crossover Diamond Interchange. No. FHWA-HRT-09-054. 2009.
8. Bared, J. Displaced Left-Turn Intersection. Publication FHWA-HRT-09-055. FHWA, 492 US Department of Transportation, 2009 493, 2009

9. Burbidge, Shaunna K. Identifying Characteristics of High-Risk Intersections for Pedestrians and Cyclists: A Case Study from Salt Lake County. No. UT-12.13. 2012
10. Cheong, Seonyeong, Saed Rahwanji, and Gang-Len Chang. "Comparison of three unconventional arterial intersection designs: continuous flow intersection, parallel flow intersection, and upstream signalized crossover." 11th Int. IEEE Conf. 2008
11. Chilukuri, Venkat, et al. Diverging Diamond Interchange: Performance Evaluation (I-44 and Route 13). No. OR11-012. 2011
12. Chlewicki, Gilbert. "New interchange and intersection designs: the synchronized split-phasing intersection and the diverging diamond interchange." 2nd Urban Street Symposium: Uptown, Downtown, or Small Town: Designing Urban Streets That Work, Anaheim, CA. 2003
13. Chu, Lianyu, et al. "A calibration procedure for microscopic traffic simulation." Intelligent Transportation Systems, 2003. Proceedings. 2003 IEEE. Vol. 2. IEEE, 2003
14. Day, Christopher M., and Darcy M. Bullock. "Cycle-Length Strategies for a Diverging Diamond Interchange in a Coordinated Arterial." Journal of Transportation Engineering 142.12 (2016): 04016067
15. Dhatrak, Amit, Praveen Edara, and Joe Bared. "Performance analysis of parallel flow intersection and displaced left-turn intersection designs." Transportation Research Record: Journal of the Transportation Research Board 2171 (2010): 33-43
16. Dorothy, Paul, Thomas Maleck, and Sarah Nolf. "Operational aspects of Michigan design for divided highways." Transportation Research Record: Journal of the Transportation Research Board 1579 (1997): 18-26

17. Edara, Praveen K., Joe G. Bared, and Ramanujan Jagannathan. "Diverging Diamond Interchange and Double Crossover Intersection-Vehicle and Pedestrian Performance." 3rd International Symposium on Highway Geometric Design, Chicago, Illinois. 2005

18. El Esawey, Mohamed, and Tarek Sayed. "Analysis of unconventional arterial intersection designs (UAIDs): state-of-the-art methodologies and future research directions." Transportmetrica A: Transport Science 9.10 (2013): 860-895

19. Esawey, Mohamed El, and Tarek Sayed. "Unconventional USC intersection corridors: evaluation of potential implementation in Doha, Qatar." Journal of Advanced Transportation 45.1 (2011): 38-53

20. Esawey, Mohamed, and Tarek Sayed. "Comparison of two unconventional intersection schemes: crossover displaced left-turn and upstream signalized crossover intersections." *Transportation Research Record: Journal of the Transportation Research Board* 2023 (2007): 10-19

21. Goldblatt, Reuben, Francisco Mier, and Joel Friedman. "Continuous Flow Intersections." (1994)

22. Haley, Rebecca, et al. "Operational effects of signalized superstreets in North Carolina." Transportation Research Record: Journal of the Transportation Research Board 2223 (2011): 72-79

23. Hildebrand, Thomas E. "Unconventional intersection designs for improving through traffic along the arterial road." (2007)

24. Hu, Peifeng, et al. "An Advanced Signal Phasing Scheme for Diverging Diamond Interchanges." 93rd Annual Meeting of the Transportation Research Board, Washington, DC. 2014

25. Hughes et al., 2010 W. Hughes, R. Jagannathan, D. Sengupta, J. Hummer Alternative Intersections/Interchanges: Informational Report (AIIR) Publication No. FHWAHRT-09-060. Turner Fairbank Highway Research Center, Federal Highway Administration, U.S. Department of Transportation, McLean, VA, U.S.A (2010)

26. Hummer, Joseph E. "Unconventional left-turn alternatives for urban and suburban arterials--Part one." Institute of Transportation Engineers. ITE Journal 68.9 (1998): 26

27. Hummer, John, and J. Reid. "Unconventional left-turn alternatives for urban and suburban arterials: an update." Transportation research circular 501 (2000): 17-p

28. Inman, Vaughan. "Evaluation of signs and markings for partial continuous flow intersection." Transportation Research Record: Journal of the Transportation Research Board 2138 (2009): 66-74

29. - Jagannathan, Ramanujan, and Joe Bared. "Design and operational performance of crossover displaced left-turn intersections." Transportation Research Record: Journal of the Transportation Research Board 1881 (2004): 1-10

30. Jagannathan, Ramanujan, and Joe Bared. "Design and performance analysis of pedestrian crossing facilities for continuous flow intersections." Transportation Research Record: Journal of the Transportation Research Board 1939 (2005): 133-144

31. Kaisar, Evangelos I., et al. "A Comparison of Non-Traditional Intersection Designs using Microscopic Simulation." Transportation Research Board 90th Annual Meeting. No. 11-3001. 2011

32. Kim, Minseok, Gang-Len Chang, and Saed Rahwanji. "Unconventional arterial intersection designs initiatives." IEEE Conference on Intelligent Transportation Systems, Seattle. 2007

33. Lownes, Nicholas E., and Randy B. Machemehl. "VISSIM: a multi-parameter sensitivity analysis." Proceedings of the 38th conference on Winter simulation. Winter Simulation Conference, 2006

34. Manual, Highway Capacity. "Transportation research board." National Research Council, Washington, DC 113 (2000)

35. Manjunatha, Pruthvi, Peter Vortisch, and Tom V. Mathew. "Methodology for the Calibration of VISSIM in Mixed Traffic." 92nd Annual Meeting of the Transportation Research Board. Washington, DC. 2013

36. Olarte, Claudia L., and Evangelos I. Kaisar. "Operational Performance Comparison between Three Unconventional Intersection Designs: Left-turn Bypass, Diverging Flow and Displaced Left-turn.", 2011

37. Reid, Jonathan, and Joseph Hummer. "Analyzing system travel time in arterial corridors with unconventional designs using microscopic simulation." Transportation Research Record: Journal of the Transportation Research Board 1678 (1999): 208-215

38. Reid, Jonathan D. "Using quadrant roadways to improve arterial intersection operations. "Institute of Transportation Engineers. ITE Journal 70.6 (2000): 34

39. Rodegerdts, Lee A., et al. Signalized intersections: informational guide. No. FHWA-HRT-04-091. 2004

40. Russo, Christopher, "The Calibration and Verification of Simulation Models for Toll Plazas" (2008).Electronic Theses and Dissertations.Paper 3480

41. Sayed, Tarek, Paul Storer, and Godwin Wong. "Upstream signalized crossover intersection: Optimization and performance issues." Transportation Research Record: Journal of the Transportation Research Board 1961 (2006): 44-54

42. Schroeder, Bastian, Katayoun Salamati, and Joseph Hummer. "Calibration and field validation of four double-crossover diamond interchanges in VISSIM microsimulation." Transportation Research Record: Journal of the Transportation Research Board 2404 (2014): 49-58

43. Schroeder, Bastian, et al. Diverging Diamond Interchange: Informational Guide. No. FHWA-SA-14-067. 2014

44. Sharma, Siddharth, and Indrajit Chatterjee. "Performance evaluation of the diverging diamond interchange in comparison with the conventional diamond interchange." Transportation Scholars Conference, Iowa State University, Ames. 2007

45. Siromaskul, Smith, and Steven B. Speth. "A comparative analysis of diverging diamond interchange operations." ITE 2008 Annual Meeting and ExhibitInstitute of Transportation Engineers. 2008

46. Steyn, Hermanus, et al. Displaced Left Turn Informational Guide. No. FHWA-SA-14-068. 2014

47. Tarko, Andrew P., Mike Inerowicz, and Brandon Lang. "Safety and Operational Impacts of Alternative Intersections." (2008)

48. Toms, Judith D., and Marc-André Villard. "Threshold detection: matching statistical methodology to ecological questions and conservation planning objectives." Avian Conservation and Ecology 10.1 (2015)

49. Toledo, Tomer, et al. "Calibration and validation of microscopic traffic simulation tools: Stockholm case study." Transportation Research Record: Journal of the Transportation Research Board 1831 (2003): 65-75

50. Utah Department of Transportation. “A UDOT Guide to Continuous Flow Intersections” Report. https://www.udot.utah.gov/main/uconowner.gf?n=10114119157568379. Site last accessed May 16, 2016

51. Utah Department of Transportation. “A UDOT Guide to Diverging Diamond Interchanges” Report. https://www.udot.utah.gov/main/uconowner.gf?n=14769524027177477. Site last accessed April 26, 2016

52. Yang, Xianfeng, et al. "Development of planning-stage models for analyzing continuous flow intersections." Journal of Transportation Engineering 139.11 (2013): 1124-1132

53. Yang, Xianfeng, Gang-Len Chang, and Saed Rahwanji. "Development of a signal optimization model for diverging diamond interchange." Journal of Transportation Engineering 140.5 (2014): 04014010