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Evaluation Of Milwaukee B And Synchronized As New Service Interchange Designs

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EVALUATION OF MILWAUKEE B AND SYNCHRONIZED AS NEW SERVICE INTERCHANGE DESIGNS

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DEDICATION

"God writes spiritual mysteries on our heart, where they wait silently for discovery"

Rumi

I dedicate this work to my lovely parents who are truly the light of God in my life. They have always been the best teachers who teach me the beauty of life. I kindly appreciate the great support of my brothers (Amir and Mehran), sisters-in-law (Sepideh and Niloofar), friends, and the teachers who helped me in all steps of my life.

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LIST OF ACRONYMS AND ABBREVIATIONS

AADT: Annual Average Daily Traffic

AASHTO: American Association of State Highway and Transportation Officials

ADT: Average Daily Traffic

ANOVA: Analysis of Variances

CL: Cycle length

CLV: Critical Lane Volume

DDI: Diverging Diamond Interchange

DOT: Department of Transportation

EB: Eastbound

FHWA: Federal Highway Administration

GPS: Global positioning system

HCM: Highway Capacity Manual

LOS: Level of service

MDOT: Michigan Department of Transportation

MOE: Measure of Effectiveness

NB: Northbound

ROW: Right-of-way

RTOR: Right turn on red

SB: Southbound

SPI: Single-point Interchange

SSAM: Surrogate Safety Assessment Model

TTC: Time to collision

VOT: Value of Time

WB: Westbound

CHAPTER 1 INTRODUCTION

These days, alternative interchanges are attracting the attention of transportation agencies and designers more than ever. Most of the existing interchanges in the U.S were built in the 1950s and 1960s when traffic volume was much lower, and the type of vehicles and driving habits were completely different. Note that the number of vehicles increased by about an average of 3.6 million each year since 1960 in the U.S (FHWA 2017). Moreover, the knowledge of highway design and safety is more developed now, and this provides an appropriate situation to increase the efficiency of interchanges regarding traffic operation and safety using alternative interchanges. The diverging diamond interchange (DDI) is a clear example of searching for new designs to solve problems related to existing (conventional) interchanges. The first study of DDI in the U.S was conducted in 2003 (Chlewicki 2003), while there are more than 80 DDIs now and many more DDIs are in the planning stages (DDI Website 2017). Therefore, the need to replace our conventional interchanges with new ones might be one of the most important and compelling topics in highway design these days.

1.1. Objectives

This research aimed to evaluate the performance of synchronized and Milwaukee B designs as possible substitutes for conventional service interchanges (an interchange is called service when freeways meet arterials or collectors). The performance of new designs was compared with four of the popular existing service interchanges in different conditions of traffic volume, traffic distribution, left/right turning percentages, and heavy vehicle ratio. A two-way interaction analysis was conducted to investigate the effect of the various parameters on the travel time of each design.

Fig. 1 shows diagrams of the Milwaukee B and synchronized interchanges. Note that northbound (NB) and southbound (SB) are assumed as freeways while east and westbound (EB and WB) perform as an arterial in Fig. 1. In the following paragraphs, a description of the characteristics and geometry of each new design is provided.

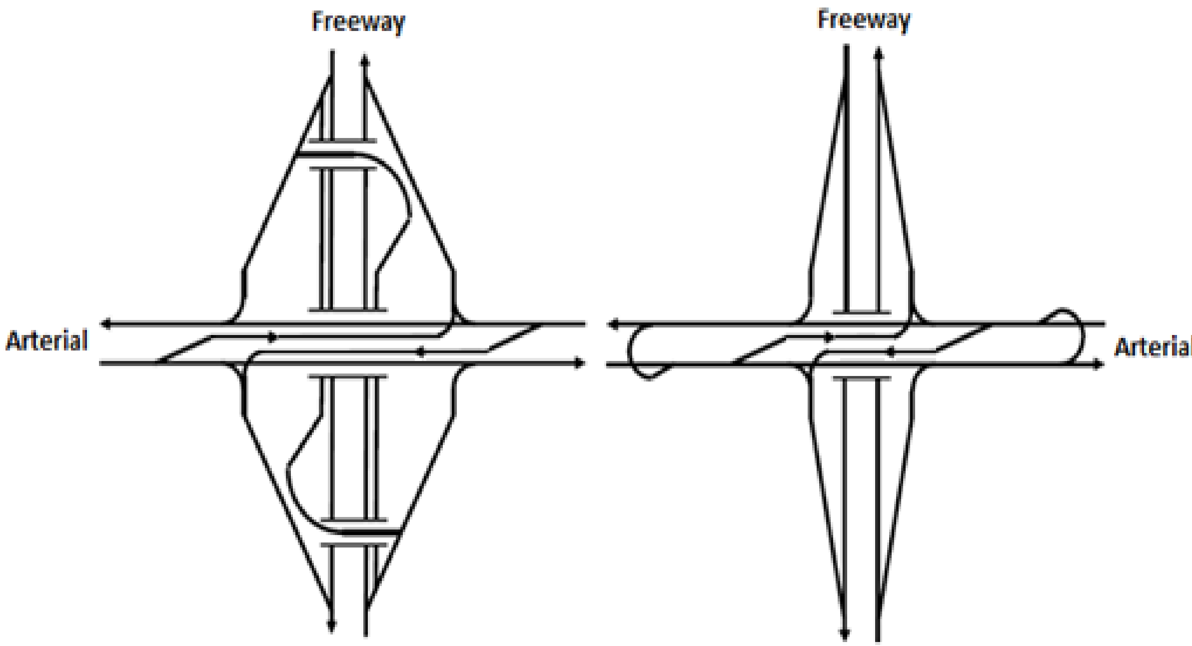


Figure 1- Milwaukee B (left side) and synchronized (right side) diagrams

In light of the primary purpose, specific objectives for this research can be mentioned as:

1. Determine the safety performance of the proposed interchanges in comparison to conventional and other alternative interchanges (such as the DDI), including the impacts on crash frequency and crash severity, the number of unusual maneuvers, the potential for wrong-way movements, and the number of conflict points;
2. Determine the traffic operation performance of the proposed interchanges in comparison to conventional and other alternative interchanges, including the

effects on capacity, the level of service, speeds, distance traveled, travel time, and queues. Also, traffic signals will be discussed to recommend appropriate ones for those interchanges with traffic signals, including the number of signals (or nodes), phasing, progression, and timings;

3. Determine the costs of the proposed interchanges in comparison to conventional and other alternative interchanges, including the construction costs, required right of way (ROW), and bridge size; and
4. Determine the performance of the proposed interchanges in terms of pedestrians in comparison to conventional and other alternative interchanges, including the ease and safety of pedestrian paths, distances, and locations of sidewalks.

According to these specific objectives, safety, traffic operation, costs, and pedestrians were introduced as the main measure of evaluations (MOEs) in this research. These are more important in comparison to other MOEs, and usually transportation agencies consider them as the priority in choosing service interchanges.

1.2. Synchronized

The synchronized interchange, which has not previously appeared in the peer-reviewed literature to the best of the authors' knowledge, has a pattern similar to the superstreet intersection (it is also called a synchronized, RCUT, j-turn, or reduced conflict intersection). Fig. 2 shows a superstreet intersection in Michigan. Based on previous studies (Hummer et al. 2010, Inman and Haas 2012, and Edara et al. 2013), superstreet intersections show great performance from the viewpoint of traffic operation as well as safety when there is a low through and left turn traffic on the minor road (Lakeview Dr in Fig. 2). Note that the through traffic on the freeway is not an important factor in the performance of service interchanges since there is no conflicting point between them and the traffic on arterial. Therefore, it is likely that we would observe great performance on synchronized interchanges when there is not considerable left turn traffic from the freeway to arterial. Also, a synchronized interchange provides contraflow left turn lanes for the left turn traffic from arterial. Contraflow left turn lanes improve the capacity and increase the storage length to reduce the impact of spillback.

Figure 2- Existing Superstreet Intersection in Troy, MI (Red and blue lines show the routes of EB, and NB, respectively. WB, and SB follow the same pattern. Source of aerial: Imagery ©2017 Google, Map data ©2017 Google)

1.3. Milwaukee B

Fig. 3 illustrates an interchange which was built recently on I-894 at 27th Street in Milwaukee, Wisconsin. To provide an easy way to refer to this interchange in the manuscript, the authors called it a Milwaukee A interchange, and a Milwaukee B interchange (the new interchange) was introduced as an improved version of the

Milwaukee A (the existing interchange). In the Milwaukee A design loops facilitate the operation of left turn traffic from the arterial to the freeway, and the left turn traffic from the freeway to the arterial makes a direct left turn. On the other hand, a Milwaukee B makes left turns from the freeway use loops, while a contraflow lane (as in the synchronized interchange) is provided for the left turns from arterial. These changes from the Milwaukee A mean that the Milwaukee B has partial (half) signals instead of full signals on the arterial. A traffic signal is called a half signal when there are only two directions (usually one through and one left/right turn direction) at the node, which allows good progression in the higher volume direction. On the other hand, the Milwaukee A and most conventional intersections and interchanges, such as the diamond interchange, use full signals which stop both directions of the arterial.

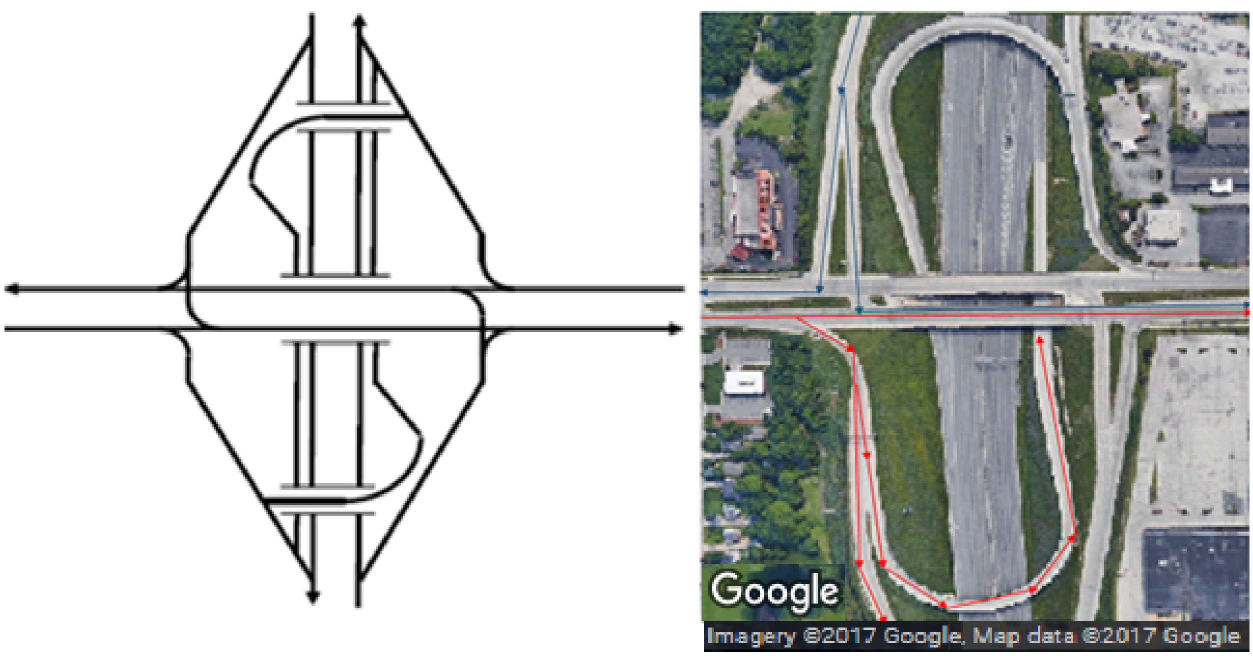


Figure 3- A diagram and a view of the interchange on I-894 at 27th Street in Milwaukee, WI (Red and blue lines show the routes of EB, and SB, respectively. WB, and NB follow the same pattern. Source of aerial: Imagery ©2017 Google, Map data ©2017 Google)

The author believes that the Milwaukee B design was first introduced and published by Eyler (2005). He referred to the design as a “parclo B with inverted loops.”

1.4. Other Involved Interchanges in the Research

The author hoped to compare the new designs to the most popular, the most efficient, the most topical, and the most similar designs, so we chose the conventional diamond, parclo B, diverging diamond, and Milwaukee A interchanges for comparison.

Fig. 4 indicates the design and direction of movements of a typical diamond interchange. A conventional diamond interchange was selected since it has the highest frequency among all the interchanges in the US. The reason of popularity of diamond interchange is due to its simple design and low cost of construction; however, it has a relatively poor performance regarding capacity, especially with high left turn volumes. According to Hummer (2014), the standard diamond interchange has one of the poorest capacity among all the interchanges, and it is not possible to provide a good two-way progression on the arterial through a standard diamond interchange.

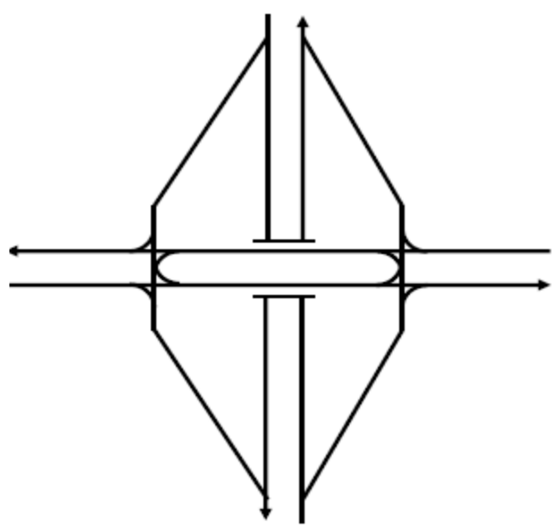


Figure 4- Typical Design of Conventional Diamond Interchange

The parclo B is one the most popular type of partial cloverleaf designs because of the good capacity in comparison to other types of cloverleaf such as parclo A or parclo AB. Loops are located after the bridge for traffic exiting the freeway in the parclo B, while the opposite of this case exists on parclo A (loops are after the bridge from left turns from the arterial). A parclo AB is when a combination of parclo B and A happens (one loop after the bridge and another one behind that). As it is shown in Fig. 5, in addition to the available ramps of diamond interchange, parclo B also provides two loops for left turns that cause fewer conflicts between left turns and through traffic. This fact facilitates the flow by increasing the capacity as well as, likely, safety (due to the removal of crossing conflict point between left turn and through traffic). Besides, there is no need to use "full" traffic signals at a parclo B and the quality of progression can be at the highest level.

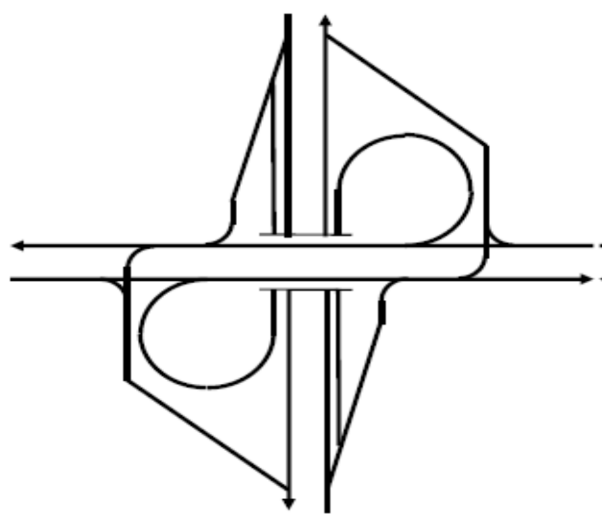


Figure 5- Typical Design of Parclo B Interchange

The main reason for better capacity and progression at the parclo B relative to the diamond is that there is one less crossing conflict point at nodes. These advantages mean that the parclo B is one of the best conventional interchanges available today and is a stern test of the new interchanges. However, despite the advantages of parclo B, it also has a few significant disadvantages such as relatively large right of way (ROW) and low speed (usually between 25-35 mph) and longer travel distances for vehicles on the loops.

As was already discussed in the beginning, the diverging diamond interchange (DDI) has attracted the attention of designers in the last decade and continued to gain popularity. Fig. 6 shows the geometry of the first DDI built in 2009. At a DDI, the direction of approaches changes from the right side to the left side as traffic heads over the bridge. With this pattern, left turns have fewer conflicts with through traffic, and that facilitates the flow. DDI became widespread because of many reasons such as:

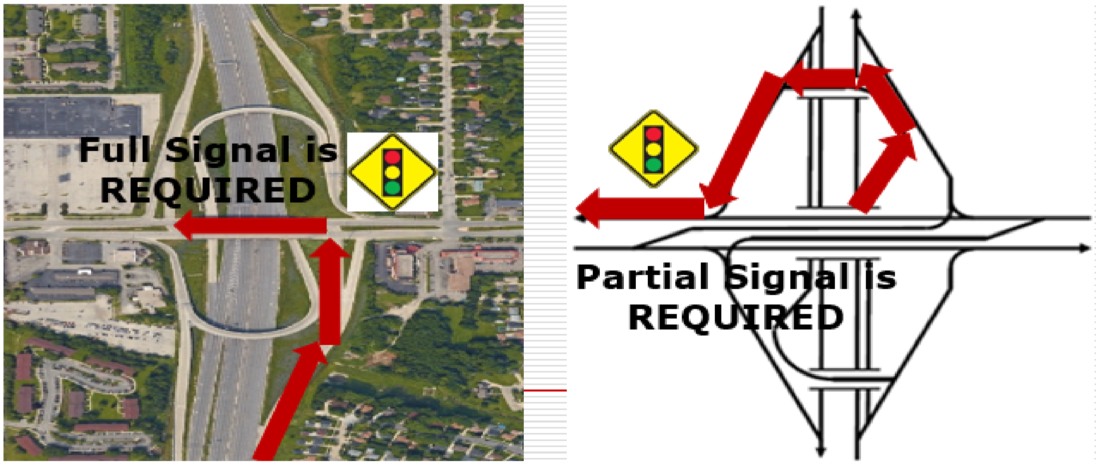
- High capacity for left turns
- Narrow and cheap bridge
- Easy to construct on conventional diamond interchanges

Figure 6- Diverging Diamond Interchange Design (MoDOT 2016)

Finally, the Milwaukee A was selected for this research due to its similarity to Milwaukee B design. Also, no published study has yet evaluated the performance of the Milwaukee A. The Milwaukee A design only has 12 conflict points, which is the lowest number of all known service interchanges. There are two traffic signals with two phases for each node on arterial and less right of way is required at a Milwaukee A in comparison to parclo B (ROW is almost half of parclo B). It is obvious that the main reason for choosing this interchange in this research is that the proposed Milwaukee B interchange has the same shape and similar pattern as Milwaukee A, while it is estimated that Milwaukee B performs better due to the half traffic signals instead of the full traffic signals of the Milwaukee A. Fig. 7 elaborates on the differences more significantly by showing a few examples of a left turn and through movements on Milwaukee B in comparison to Milwaukee B.

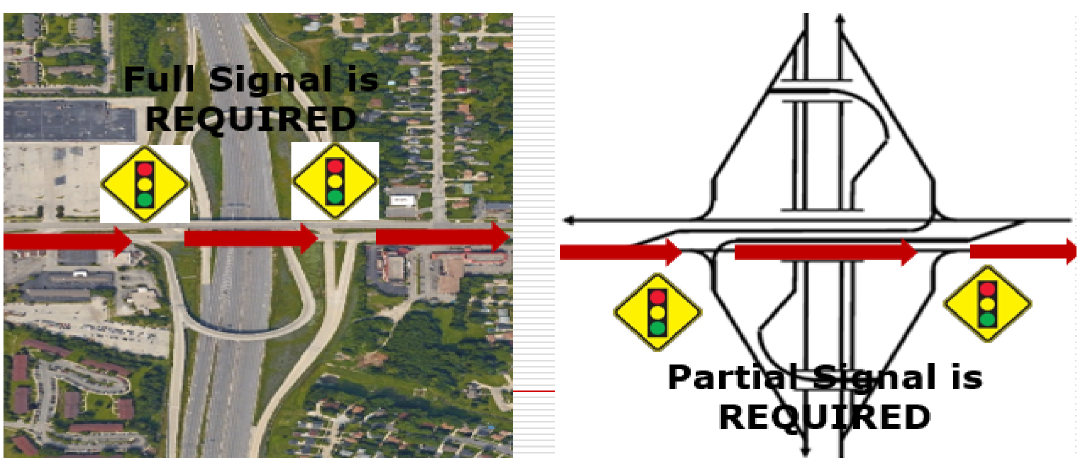
1.5. Scope of the Study

Interchanges can be divided into two different groups as “service interchanges” (when freeways meet arterials or collectors), and “system interchanges” (when freeways meet freeways). This research only concentrates on service interchanges. The main reason for this limitation is that system interchanges have a different set of issues and surely requires another comprehensive research project to study. Moreover, service interchanges seem to be more critical than system interchanges since the vehicle’s speed should be changed more significantly on any entrance and exit ramps. In the United States, it is common to use speed limits of 70 mph and 35 mph for freeways and arterials, respectively. Therefore, the design must be able to provide a safe situation for



a. Left-turn from freeway to arterial

b. Left-turn from arterial to freeway



c. Through movements on Arterial

Figure 7- A Comparison between Milwaukee A (on the left) and Milwaukee B (on the right)

this change (from 35 mph to 70 mph and vice versa). In addition, this research only includes urban and suburban areas. The obvious reason is that the main problems of conventional interchanges are related to capacity and safety. Therefore, urban and suburban areas would face these problems more than rural areas because of the higher volumes of traffic. In addition, it should be mentioned that this study will focus on arterials with two lanes in each direction of travel. Generally, four-lane arterials are more common than other sizes; however, conclusions (especially, capacity) might be different by considering three or more lanes in each direction on the arterial.

The research did not include the effects of drainage, bicycles, and adjacent land uses in the analysis. The location of adjacent land uses, and their impact on the traffic operation, varies case by case in different situations. Since the research did not have a target area of study and the aim was defined to present inclusive results which would be practical for different cases, no adjacent land use was assumed in the study. The drainage had almost the same condition for the consideration in the analysis, and it was excluded since no inclusive condition could be applied to the analysis. Bicycles might be the option which could have more chance to be considered in the research; however, the author decided to put the main focus on the other important MOEs (as selected) and consider the bicycle analysis as a further study. It can also be claimed the effect of bicycles have been included in the pedestrian analysis since the both follow the same features and method of analysis based on the Highway Capacity Manual (HCM 2010). In fact, the speeds would be the most significant difference in the analysis of bicycles in comparison to pedestrians.

The process of this research will include the following steps. The first step of the research will be the geometric design of proposed interchanges. Estimating the appropriate dimensions of horizontal and vertical curves, lengths of grades, the number of lanes and rights of way are necessary for other work. Note that all the geometric design calculations will be based on Green Book (AASHTO 2011). Simulating and traffic analysis will be the second step of the research. In this step, all the results related to traffic operation will be collected from VISSIM and Synchro. VISSIM, which was made in Germany in the 1970s, is microscopic simulation software to model different traffic patterns with detailed geometric configurations and drivers' behavioral characteristics encountered in the transportation system (Liu et al. 2012). It should be mentioned that a part of the pedestrian analysis will be done by VISSIM software as well. On the other hand, Synchro is macroscopic software to model the performance of signalized intersections and roundabouts based in part on Highway Capacity Manual (2010). Safety analysis is another substantial step of this research. SSAM (Surrogate Safety Assessment Model), which analyzes the frequency and character of narrowly averted vehicle-to-vehicle collisions in traffic (FWHA 2008), will be used to get a good view of the performance of the interchanges in terms of safety. Then, an estimation of costs will be done to complete the set of results on the proposed interchanges.

As the last step of this research, evaluation of estimated improvements of proposed interchanges in comparison to conventional or other alternative interchanges will be helpful to present advantages and disadvantages of proposed interchanges. This evaluation might be the most important part of the research since it is going to introduce the most promising interchange for different situations of traffic flow. Of course, there is no perfect design, but it is possible to increase the potential of safety, traffic operation, and other important MOEs by choosing the best design for a particular place based on proven results. For example, a question such as, “which interchange shows the best performance in a case with a large volume of the left turn from the freeway to arterial” will be answered in this part to help transportation agencies in choosing the proper interchange design in particular projects. It should be mentioned that the comparable interchanges and the criteria of selection of them for this evaluation will be discussed later in the Methodology chapter.

CHAPTER 2 LITERATURE REVIEW

2.1. Alternative Interchanges

The first traces of alternative designs for interchanges and intersections might go back to 1950s when jughandles were built in New Jersey. Median U-turn intersections were introduced in Michigan in the 1960s, and there are many of them in Michigan that still perform well. Dorothy et al. (1998) conducted a comprehensive study of the median U-turn interchange and found acceptable performance (especially in terms of capacity) in comparison to conventional diamond interchanges.

Alternative interchanges gained attention in the 1990s with growing traffic demand and tight budgets for funding new highways. In that age, roundabout interchanges and the single-point interchange (SPI) were introduced as potential solutions. Despite the good performance of roundabout interchanges in terms of safety and pedestrians, it did not emerge as a universal treatment since its capacity was limited. On the other hand, the SPI became popular, and hundreds of SPIs were built in the U.S. during the 1990s and early 2000s The SPI could perform very well regarding traffic operation due to its single three-phase signal, but the wide bridge made it an expensive choice. Thompson et al. (2003) and Shin et al. (2008) estimated that a typical single-point interchange costs US$2-4 million more than a typical diamond interchange. The single signal allowed the through traffic to clear the interchange faster than a conventional diamond, and the opposing left turns could move at the same time (Messer et al. 1991; Bonneson 1992; Qureishi et al. 2004). Safety at an SPI was questionable because the geometry was complicated for the users who were not familiar. Bared et al. (2005b) compared the observed and expected crashes of SPI and a tight diamond interchange and found no substantial difference between these two types in a total number of crashes. However, SPIs were estimated to be safer than a diamond in urban districts regarding fatality frequencies. Note that the SPI and roundabout interchange is considered as conventional interchanges nowadays.

The W-interchange was introduced in 2003 as a new design to improve the median U-turn interchange by removing the left turn traffic from the main intersections (Thompson et al. 2003). The minimum number of stops (as a factor of safety) was seen in W-interchange compared to diamond, single-point, and median U-turn interchanges; however, it presented higher travel times in comparison to single-point and median U-turn interchanges.

The diverging diamond interchange (DDI), which is also known as double crossover diamond (DCD), was another alternative design which received a substantial notice after its first publication by Chlewicki (2003). The origin of DDI comes from France in the 1960s (Chlewicki 2003); however, the first DDI in the U.S. was opened in 2009 in Missouri. Since the first established DDI in 2009 (DDI Website 2017), hundreds of studies have been done to review different aspects of its performance. Chlewicki (2003), Bared et al. (2005a), and Edara et al. (2005) conducted the first studies of the performance of DDI and achieved similar rosy conclusions. The DDI usually works very well in areas where diamond, partial cloverleaf, and roundabout interchanges have shown a poor performance, especially if there is a heavy left turn demand. Chlewicki (2003) estimated an average delay of 27 seconds per vehicle could be expected in a DDI with merges, while conventional diamond presents about 80 seconds per vehicles in the same situation. As another example, the delay at a DDI was seen as 50% less than conventional diamond interchange in heavy traffic situations (Hughes et al. 2010). The DDI increased the safety in comparison to the conventional diamond interchange because of the fewer conflict points, the lower speed at crossing-path conflict points (due to the curves in crossovers), and the reduction of traffic delay (Chlewicki 2003; Maji et al. 2013; Yeom et al. 2015). The DDI also can provide a low cost of construction due to the small size of the bridge, but its performance regarding capacity (especially, in low left-turning conditions) and crossing pedestrians is not great (Vaughan et al. 2013; Schroeder et al. (2014); Yeom et al. 2015; Edara et al. 2015). Although, these studies all noted that DDIs would provide poor performance when there is high through traffic on arterial. As an example, a DDI was not a proper treatment for improving a conventional diamond interchange for a case in Alabama (Khan and Anderson 2016). Regarding the pedestrian performance, two different paths have been used at a DDI: (1) a center crossing, and (2) an outside crossing. However, both the paths are like the SPI's path having four free-flowing and four controlled crossings and this added another point to the list of DDI disadvantages.

One of the most innovative research efforts in this area was by Eyler (2005). This research proposed an upgraded design for a segment (about 2.2 km) of Hwy 55 in Plymouth, MN. The proposed design converted the four signals on the segment to half-signals. The Milwaukee B design was considered for the corridor. Simulation results showed that the new design significantly decreased the travel time of the segment. The benefits of travel time savings due to the new design were estimated to cover all the construction costs in a three-year period. However, the simulation conducted in the research was limited, and Eyler (2005) recommended that future studies are necessary to examine a wide range of various traffic conditions.

A creative two-level intersection design was innovated by Shin et al. (2008) to separate the left-turn traffic from through traffic flow. The new design experienced the lowest values of delay in comparison to the three other designs (single-point interchange, center-turn overpass, and echelon interchange) except in very unbalanced traffic conditions between the major and minor roads.

Berry and Click (2011) did VISSIM simulation research on three unconventional designs: median U-turn, superstreet, and a design called “FRE” that requires all left-turning vehicles to use U-turn crossovers downstream from the interchange and found that all these designs have great operational potential.

More recent research that has attempted to develop an alternative interchange design include Chlewicki (2010), Gingrich (2012), Hale (2014), Krauuse et al. (2014), and Zhao et al. (2015).

2.2. Safety Studies by SSAM

The research by Gettman and Head (2003) might be the first attempt at developing a surrogate safety measure based on traffic simulation modeling. This effort resulted in the release of the first version of SSAM in 2008 supported by Federal Highway Administration (FHWA) (Gettman and Head 2008). SSAM uses the trajectory files of traffic simulation packages to recognize the type and number of near misses between vehicles during the simulation period.

The majority of the previous studies on SSAM were related to validation and calibration of the software. Fan et al. (2012) compared the number of conflict points observed in a field study with the estimated number by VISSIM and SSAM models and found an acceptable consistency between the results of the comparison. In a study on 300 km2 of a highway network in Netherland, a relationship was also revealed between simulated conflicts and the six years of crash reports (Dijkstra et al. 2010). Despite the satisfying research results on the validation of SSAM, calibration of driver behavior in the simulation procedure is highly recommended before using SSAM by most the previous studies (Fan et al. 2012; Zhou et al. 2010; Huang et al. 2013; Essa and Sayed 2015). A calibrated driver behavior model in VISSIM could diminish the errors as much as 50% (Fan et al. 2012).

One of the innovative studies presented a new model to estimate the crash modification factor (CMF) based on the conflicts derived from SSAM (Shahdah et al. 2014). The CMF model was defined based on the number of conflicts in after the improvement phase to the before period and the value which is related to the crash-conflict expression and showed a high accuracy compared to CMFs based on crash data.

2.3. Pedestrian Studies

Pedestrians, as the most vulnerable users of the highway system, make up about 22% of annual crash fatalities across the world (World Health Organization 2014). Approximately, 11-13% of highway fatalities in the US and Canada are pedestrians but this rate reaches 25%, 30%, and 38% in China, Poland, and Korea, respectively (Moreno et al. 2011; Zhang et al. 2016; Oskarbski et al. 2016; Jung et al. 2016). The statistics become very substantial in less developed countries, where the rate increases to 57% in Mumbai, India (Marisamynathan and Vedagiri 2013). The threat seems to be more critical in intersections and service interchanges (where a freeway intercepts an arterial) due to the interaction of pedestrians and vehicles, especially in urban and suburban areas. In Montreal, almost 60% of pedestrian crashes occurred at intersections (Brosseau et al. 2013), while the rate in some places in the US is as high as 76% (Pulugurtha and Sambhara 2011).

The design and construction of a majority of current service interchanges in the US go back more than 50 years when there was much less notice regarding pedestrians. In fact, pedestrian performance was always taken into account as an afterthought after vehicular consideration (Keegan and O'Mahony 2003). The considerable traffic growth during last two decades caused transportation agencies to improve some old interchanges using alternative designs such as the diverging diamond interchange (DDI). The DDI offers good traffic operations and superior safety (Vaughan et al. 2013), but questions remain on its friendliness to pedestrians. This point might be one of the reasons for special attention to the topic these days, as a new project entitled NCHRP 07-25 has been recently funded regarding the pedestrian performance at alternative intersections and interchanges (TRB 2017).

Many previous studies had been conducted to analyze and recognize the parameters involved in pedestrian crashes at intersections. Long waiting time for the walk (green) interval, short walk interval, and the high turning volume of vehicles at conflict points with pedestrians on permissive green controls were identified as the most important variables for pedestrian crashes at intersections (Oskarbski et al. 2016). Brosseau et al. (2013) studied the effect of pedestrians waiting time on their safety at intersections. The research defined the waiting time as a factor of signal phasing and arrival time and concluded that minimizing waiting time can considerably decrease the dangerous behavior and violations of pedestrians. Clearance time (or the flashing "DO NOT WALK" interval) was introduced as another important factor in pedestrian safety. Pedestrians tend to commit a violation either when the clearance time is longer or shorter than needed.

Regarding pedestrian operation analysis, many important gaps are still observed in the literature in spite of the recent efforts. Milazzo et al. (1998) and Hubbard et al. (2009) claimed that the method in the version of the Highway Capacity Manual (HCM) in force when they did their research was not accurate and the effect of traffic volume was not reflected well. The HCM 2010 method relates LOS for pedestrians to pedestrian space and delay, while no other variables such as the effect of right-turning traffic on pedestrians are considered. Other possible parameters which are not taken into account in the HCM analysis include the direction of pedestrian movement, pedestrian volume, the time of arrival (whether the pedestrian arrives on time or late to the crossing point), and the crosswalk location (Hubbard et al. 2009). Milazzo et al. (1998) examined the capacity of intersections considering the effect of pedestrians. Their results showed that pedestrian volume impacts the vehicular saturation flow, especially when the rate is more than 500 pedestrians per hour in the US. Milazzo et al. (1998) recommended adding new saturation flow adjustment factors to include the effect of pedestrians on the affected lane groups (right turn and left turn). Another research effort (Rouphail et al. 2005) revealed a negative nonlinear relationship between pedestrians delay and vehicle volume.

CHAPTER 3 METHODOLOGY

This section on the research methodology includes four parts: (1) Simulation Models, (2) Simulation Scenarios, (3) Geometric Features of Interchanges, and (4) Users Behavior.

3.1. Simulation Models

Simulation models are in widespread use in various aspects of transportation engineering from the studies on user behavior (Yang et al. 2006; Fitzpatrick et al. 2013) or vehicle dynamics (Stine et al. 2010; Molan and Kordani 2014a; Kordani et al. 2014), to studies related to operation and safety of transportation systems (Chlewicki 2003; Gettman et al. 2008; Olya 2014a; Olya 2014b; Molan and Kordani 2014b; Kordani and Molan 2015). Based on recent and relevant studies (Rouphail et al. 2004; Gao et al. 2012; Ishaque and Noland 2009; Kim et al. 2013; Oskarbski et al. 2016), it can be concluded that VISSIM is one of the best available tools in this field for modeling vehicles and pedestrians, especially in the U.S. PTV VISSIM 7 was selected as the main tool for traffic modeling in this research. VISSIM is a microscopic multi-modal traffic flow simulation software which is in widespread use around the world as one of the best available simulation software for the traffic modeling in different conditions. VISSIM can include a variety of important factors such as different driver behaviors, types of vehicles, and various traffic signals systems. In addition to the high ability and the popularity of VISSIM in the field of transportation, VISSIM is one of the only microscopic simulation models which had good consistency with SSAM.

There is no doubt that signal timing and phasing plays a notable role in the performance of pedestrians at an interchange. To make sure of the accuracy of signal timing in the simulation, all the signals were designed using Synchro 8. Synchro is a macroscopic model that is the most popular current method for analyzing signalized intersections and providing progression (Traffic Ware 2016). Both VISSIM and Synchro have been validated by many studies (Petraglia 1999; Ishaque and Noland 2009; Eustace and Ponnada 2012; Schroeder et al. 2014; Molan and Hummer 2017) and have provided high accuracy in their outcomes. Ishaque and Noland (2016) conducted extensive research on the calibration and validation of VISSIM and found that the VISSIM car-following algorithm is strongly adapted to the reality of pedestrian and vehicle flow even in the most complicated modeling environments. The extracted travel time graphs from VISSIM were very similar to the graphs of field data in their research. After the Synchro and VISSIM modeling, ANOVA was performed on the results using IBM SPSS to compare the mean values of the measure of effectiveness (MOEs) as well as to investigate the effect of various variables on the performance of each interchange. The following sections describe different aspects of the simulation modeling.

One of the difficulties of highway safety engineering is its dependence on crash statistics. In fact, a wide range of crashes must occur over a long period to prepare the situation for a trustworthy safety analysis (Essa and Sayed 2015) which is a sizable burden for any new design. Releasing SSAM an attempt to diminish that dependence. At the moment, there is no better safety analysis for evaluating the safety performance without waiting for crashes to happen (Zhou et al. 2010), especially in the safety evaluations of new designs with no available crash statistics. SSAM was chosen to conduct the main safety analysis in this study. The SSAM model is a combination of microscopic simulation and automated conflict analysis which is able to study the frequency and character of narrowly averted vehicle-to-vehicle collisions in traffic (FHWA 2008). SSAM provides a great opportunity for researchers to get the safety of roadways based on estimated frequency of traffic facilities without waiting a long-time period to observe the crashes and injuries after the construction of roads. Therefore, the method of safety analysis in this study was to send the trajectory files generated by VISSIM simulation models into SSAM to identify the types and number of conflict points during each simulation run. SSAM considers the time to collision (TTC) as the measure of effectiveness (MOE) and introduces a vehicle conflicting interaction (conflict points or near misses) when the TTC becomes less than the considered threshold during the simulation. It is axiomatic that the higher number of conflict points raises the probability of experiencing more crashes as well as longer travel times (Chai and Wong 2014).

Reviewing the previous studies, the 1.5-sec TTC (SSAM's default) was found as the most popular threshold among the researchers. Huang et al. (2013) reviewed the effect of TTC threshold on the accuracy of outcomes and found an optimum TTC value of 1.6 sec for minimizing the error of rear-end conflicts; however, the effect of TTC threshold was not seen to be very important to the accuracy of the other types of conflicts. No reason was provided for reducing the TTC threshold to less than 1.5 sec by Shahdah et al. (2014). A stronger relationship between simulated and real conflicts was illustrated for higher the TTC thresholds by Essa and Sayed (2015), due to the higher dependency on exposure in higher TTC values. Considering this body of previous research, the threshold of TTC was chosen equal to 1.5 sec in the study.

3.2. Simulation Scenarios

The following paragraphs had delineated simulation scenarios of the study. Note that the vehicle travel time, as the most important MOE of the study, had the most comprehensive simulation procedure while the less number of scenarios were considered for the pedestrian and safety evaluations.

3.2.1 Traffic Operation Evaluation

A fair comparison between interchanges would not be possible without including a wide range of traffic scenarios in the simulation. This research proposed 180 traffic scenarios for each interchange considered to cover various states of traffic volume, traffic distribution (on different directions), left/right turning volume ratios, and heavy vehicles' traffic percentage in the traffic operation evaluation. Table 1 illustrates the considered conditions in terms of turning volumes and traffic distribution. In this table, EB and WB represent the arterial while NB and SB represent the freeway ramps.

Table 1- Turning volumes and traffic distribution cases of the traffic operation modeling

Turning Volume Ratios	**Traffic Distribution on EB/WB**	**Traffic Distribution on NB/SB**
Left turn=Through=Right turn (High Left/Right Turn Case)	EB = WB (Equal Volume on EB and WB)	NB = SB (Equal Volume on NB and SB)
Left turn=0.66Through=Right turn (Moderate Left/Right Turn Case)	EB = 0.75 WB (Slightly High Volume on WB)	NB = 0.75 SB (Slightly High Volume on SB)
Left turn=0.25Through=Right turn (Low Left/Right Turn Case)	EB = 0.5 WB (Significantly High Volume on WB)	NB = 0.5 SB (Significantly High SB Volume)
		0.75 NB = SB (Slightly High Volume on NB)
		0.5 NB = SB (Significantly High Volume on NB)

Altogether there were 15 traffic distribution cases simulated, including balanced and unbalanced cases. Heavy vehicle percentages were based on the results from data collection at a sample of 37 random service interchanges in the US. The percentages simulated--4%, and 8%-- represent the moderate and high heavy vehicle volumes from that sample. The simulations included only cars and trucks—no other types of heavy vehicles were included because of the negligible volume of the other types of heavy vehicles (mostly less than 1% of total volume) in the data collected. Traffic volume levels were defined after critical lane volume (CLV) calculations. CLV calculation is an old, quick, software-independent measure of intersection or interchange operations. CLV considers the conflicting movements at nodes, including arterial through movements, crossover movements, merging movements from off-ramp to arterial, and merging movements at the beginning of the on-ramp (Maji et al. 2013). To cover balanced and unbalanced traffic situations, six different traffic distributions were considered in the study. Fig. 8 shows two of these traffic distributions on a conventional diamond.

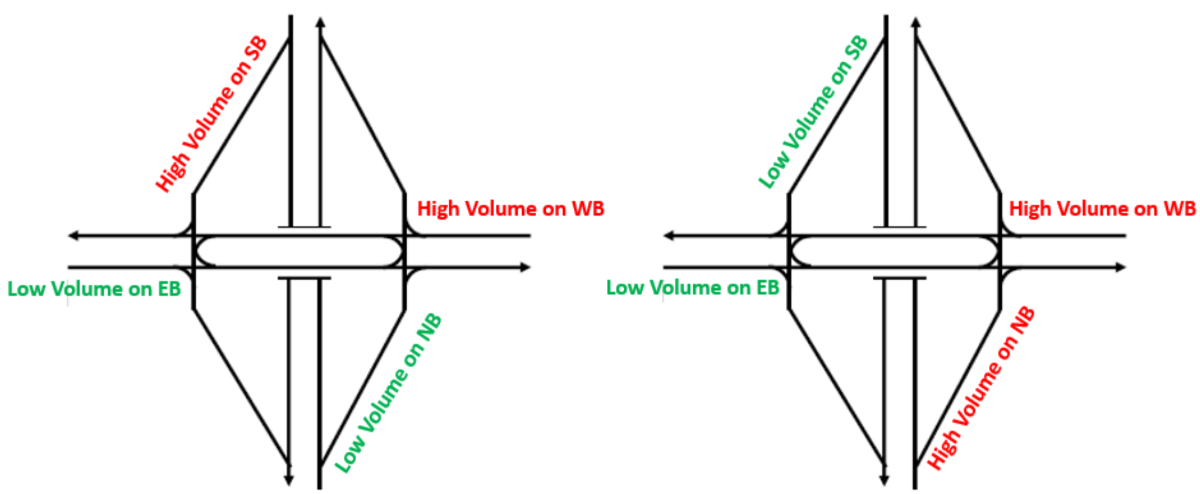


Figure 8- Two examples of considering unbalanced traffic distribution in the simulation

Two volume levels were simulated based on volume over capacity ratios (v/c) from the CLV calculation: (1) when the v/c was equal to one in the diamond interchange, and (2) when the v/c was equal to one in the DDI. Based on these calculations, the simulated traffic demands are shown in Table 2. In sum, a total of 1080 scenarios (6 types of interchanges*3 turning conditions*15 traffic distribution cases*2 heavy vehicle percentages*2 V/C ratios = 1080) was modeled in this part of the research. Each scenario was repeated two times in VISSIM with different random number seeds. Synchro was also used for each of the scenarios to provide the optimum signal timings. Appendix B has illustrated more information regarding the simulation scenarios.

Table 2- Defined traffic volumes based on CLV calculation

Turning Case / V/C Value	**Equal to 1 in Diamond Interchange**	**Equal to 1 in DDI**
High Turning	5140 vph	6400 vph
Moderate Turning	5250 vph	5600 vph
Low Turning	5330 vph	5120 vph

3.2.2 Pedestrian Evaluation

Since the target area of this research is service interchanges in urban and suburban areas, pedestrians should be included in the analysis.

The network was analyzed for two cases of pedestrian volume: (1) when the pedestrian volume was 360 per hour total on all sidewalks, and (2) when the network is located in a place with no effective presence of pedestrians. It is clear that the pedestrian volume can be completely different in particular cases based on the type of the location or even the adjacent land uses. One of the reasons for considering high rates of vehicle and pedestrian volume in the research was to receive more trustworthy and valid outcomes. According to the crash models developed in Pulugurtha and Sambhara (2011), high pedestrian volume models have higher accuracy than the models with lower pedestrian volumes. Fig. 9 indicates the pedestrian path of each design. As shown in Fig. 9, there were four origins and destinations (southwest, southeast, northwest, and northwest) for the pedestrians, so, each route handled 90 pedestrians per hour in the model runs with pedestrians. No pedestrians crossed the arterial; all pedestrians crossed the bridge.

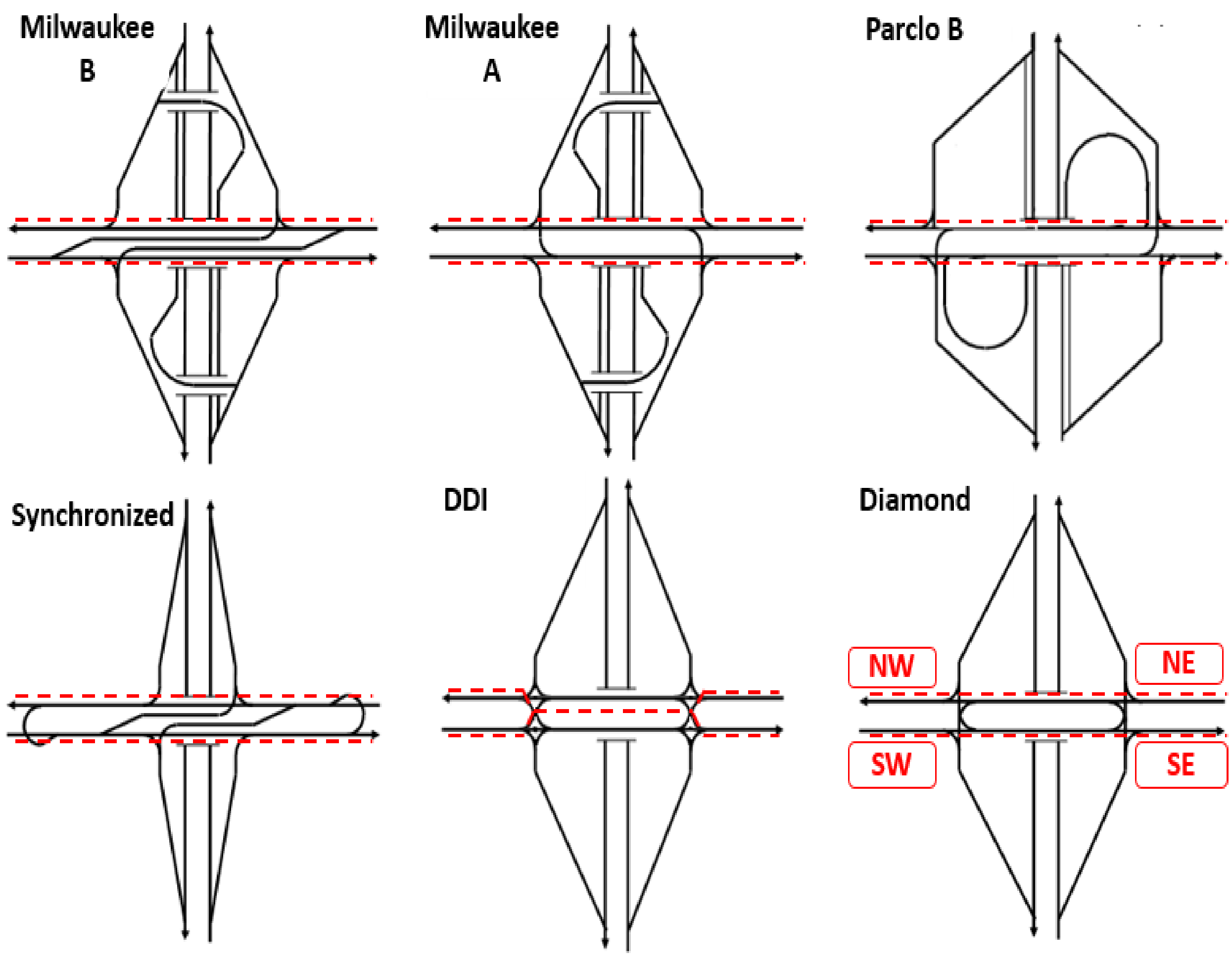


Figure 9- The geometry and the pedestrian path (the dashed red line) in the interchange tested (not to scale)

Table 3 presents a summary of scenarios in the research. A total of 432 VISSIM simulation scenarios were tested.

Table 3- Division of traffic conditions in the simulation modeling of the pedestrian evaluation [a]

Pedestrians Volume	**Truck Percentage**	**Turning Volume Ratios**	**Traffic Distribution**	
			EB/WB	**NB/SB**
360 Pedestrians per Hour	4 % (Moderate Truck Traffic)	Left turn=Through=Right turn (High Turning Condition)	EB = WB (Equal traffic on EB and WB)	NB = SB (Equal traffic on NB and SB)
No Pedestrian	8 % (High Truck Traffic)	Left turn=0.66Through=Right turn (Moderate Turning Condition)	EB = 0.5 WB (high traffic on WB)	NB = 0.5 SB (high traffic on SB)
		Left turn=0.25Through=Right turn (Low Turning Condition)		0.5 NB = SB (high traffic on NB)

[a] the SSAM modeling also had the same scenarios but without considering any pedestrian volume

3.2.3 Safety Evaluation

Safety is other important MOE of this research. Unfortunately, crash statistics in the U.S show over 30,000 fatalities per year. In 2012, almost 30,800 people were killed in the U.S highways (NHTSA 2014). Therefore, this research focuses on safety and tries to consider the different aspects of safety. The number and type of conflict points, wrong-way movements, and unusual maneuvers might be the most available parameters on safety at interchanges (Hummer 2015). The wrong-way movement and unusual maneuvers are the parameters related to the geometry of interchanges and their effects on the safety were slightly considered in the research; however, another research is required to study the effects comprehensively modeling the drivers' behavior by simulation laboratory. Therefore, the main focus of safety analysis in this research was on the conflict points of interchanges. Conflict points might be named as one of the most important variables in any safety study. Overall, there are three types of conflict points: crossing, merging, and diverging. Figure 10 indicates the location of conflict points in a 4-leg conventional intersection to gives a view of their differences (FHWA 2014). The most dangerous conflict point is a crossing point which usually makes the critical safety problems. A crossing conflict happens when two different movements should pass (cross) each other. For example, a left turn from NB makes a crossing conflict with the SB through traffic in a conventional intersection. It is a clear fact that a highway will experience safer if its design provides the minimum number of conflict points. Besides, crossing conflict point also affects the traffic operations, and they are a factor in determining the number of phases in traffic signals.

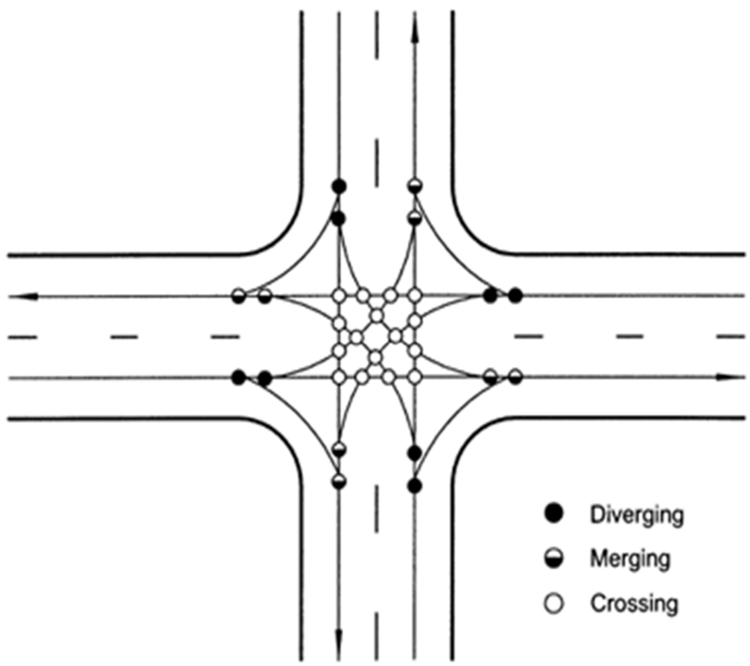


Figure 10- Different types of conflict points on a 4-leg conventional intersection (FHWA 2004)

Merging conflict points are usually less important than crossing points. Merging in service interchanges usually occurs between the right-turn volume on ramps and through the traffic of arterial. These cases will be critical if the enough merging distance (accelerating/decelerating lengths) is not provided in the design or when there is a short weaving area. Diverging, the other type of conflict point, happens when a driver wants to make a left or right turn from the mainline road. Diverging conflicts rarely makes a safety problem.

This research considered 36 scenarios for each of six designs to do the safety analysis by SSAM. The division of scenarios were the same as Table 3 (the scenarios of pedestrian evaluation), but no pedestrians were considered during this effort due to the limitation of SSAM regarding modeling the pedestrians. Therefore, a total of 216 trajectory files (generated by VISSIM) were tested by SSAM. Note that each of the scenarios was run two times (two hours in total) again to include different seeds in VISSIM simulation and then the average of the two SSAM outcomes was used in the safety analysis.

3.2.4 Cost Estimation

As the last MOE considered in this research, an economic analysis was conducted to estimate the costs and benefits of each interchange. Of course, cost estimation is one of the most difficult and critical parts of any project since DOT budgets are so tight and needs for new or upgraded facilities are so great. An initial comparison of the costs of alternatives would be helpful to see if there are large differences and to see the relative ranking of the alternatives. Costs of interchanges can be divided into two various categories: (1) construction costs, (2) and user costs and benefits. The construction costs were examined using unit costs of bridge and pavement structures in the cost estimate worksheet of Michigan Department of Transportation (MDOT 2017), and a published report regarding the land value in Michigan (MSN 2015).

As the user's cost and benefits, the value of travel time was extracted from the previous studies to estimate the benefits of saving travel times of the new interchanges. The other MOEs such as fuel consumption, or the costs of reducing crashes were not considered in this section because the estimation of these MOEs would not be accurate unless conducting a field study. Of course, since the new designs have not built yet, this field study could not be practical at this moment. For example, the fuel consumption is a factor of the vehicle dynamic (acceleration/deceleration) behavior which could not be estimated precisely by simulation modeling. Both the delay and safety costs are very important from the viewpoint of user costs. According to the recent statistics, the average cost of each type-K (fatality) crash is about $10 million (National Safety Council 2014). In fact, a safe interchange that reduces the number of crashes can easily save millions of dollars annually. Table 4 shows the estimated cost of crashes based on the National Safety Council averages. Note that the costs include wage and productivity losses, medical expenses, administrative expenses, motor vehicle damage, employers' uninsured costs and a measure of the value of lost quality of life. The value of lost quality was measured by conducting empirical studies of what people pay to reduce their safety and health risks.

Table 4. Average costs of crashes based on injury severity (National Safety Council 2014)

Death	$9,887,000
Disabling	$1,082,000
Evident	$298,000
Possible injury	$138,100
No injury observed	$45,700

Delay is another variable that causes considerable user costs. In the Netherlands, a rate of 10.4-13.6 billion Euro per year is estimated for traffic accidents, and delays and incidents' delays include almost 12% of this estimation which is about 336- 432 million Euro per year (Steenbruggen et al., 2012). A combination of the crash and its delay can sometimes generate a huge cost. According to UK Department for Transport (Steenbruggen et al., 2012), it is estimated to observe one additional secondary collision from a vehicle running into the back of another vehicle in the queue for every 30 hours of queuing. Also, a 2-hour incident maybe causes up to 600,000 Euro of costs on a blocked 3-lane highway.

3.3. Geometric Features of Interchanges

Radii of curves, ramp lengths, and ROW are some of the most important geometric variables in interchange design. To increase the confidence in the simulated designs in this research, these variables were collected using Google Earth and AutoCAD from 30 existing service interchanges in the US. Table 5 presents the values of collected and selected geometric parameters. The comprehensive detail of data collected is provided by Appendix C as well.

As it is clear from Table 5, the diamond interchange has a large distribution of the distance between ramp terminals. In fact, there are three different categories for diamond interchanges based on the distance between ramps: spread diamond (about 1200 ft), standard diamond (about 600 ft), and tight diamond (about 200 ft). This research considers a standard diamond interchange since a 600-ft distance between ramps is more suitable for most of the interchanges being simulated (all except the parclo B which needs a distance of 1200 ft).

Table 5- Collected geometric data of existing service interchanges

Parameter	Right Turn radius of on-ramp (ft)	Right Turn radius of off-ramp (ft)	Length of on-ramps (ft)	Length of off-ramps (ft)	Loop Radius (ft)	Distance between Ramp Terminals, (ft)		
						Parclo	Diamond	DDI
Average	43	41	1680	1850	250	1320	906	750
Median	48	48	1800	2000	260	1300	850	700
Minimum	20	20	800	900	200	1000	250	600
Maximum	70	70	2600	2800	280	1600	1650	1300
Selected Value	40	40	2000	2000	250	1200	600	600

The synchronized interchange, due to its U-turn crossovers, provided more design challenges. Therefore, the authors collected information from 14 existing superstreet intersections to increase confidence in the design. Table 6 presents a summary of the collected data regarding superstreet intersections.

Table 6- Collected geometric data of existing superstreet intersections

Parameters	Radius of U-turn, (ft)	Radius of Loon, (ft)	Median Width, (ft)	Distance from U-turns to the Center, (ft)
Average	29	47	31	1030
Median	28	45	22	900
Minimum	15	40	8	500
Maximum	40	60	130	2400
Selected Value	30	45	24	800

Note that the dimension of vehicles is the main factor in selecting median width. The median width was chosen as 24 ft since it would be consistent with the dimensions of the design vehicle (large truck) in this research. The maximum longitudinal grades on ramps and loops were chosen as 2% and 3.5%, respectively. Based on Table 5, the radius for all the loop ramps in the parclo B, Milwaukee A, and Milwaukee B was 250 ft which are proper for a speed of 30 mph and the 6% superelevation rate. Based on Yang et al. (2015), trucks and passenger cars have almost the same acceleration-versus-distance profile; however, the acceleration capability of trucks is lower than passenger cars. Therefore, the same acceleration-versus-distance profile was defined on ramps for both the passenger cars and trucks in the research. The elevation of the arterial was designed as 23 ft higher than the freeway, which provides a safe clearance of 16 ft under the bridge. It is more common to locate the arterial on the top of the freeway in the US, so that is what we simulated.

The research assumed two through lanes in each direction of the arterial, one exclusive left turn lane, and one exclusive right turn lane. Of course, each design has its unique features and formation, but the same number of lanes was considered for all the interchanges to provide a fair comparison. Also, all the left and right auxiliary lanes begin at the same location. Right turn lanes have a storage length equal to 400 ft with a 100-foot taper. Fig. 11 shows all the simulated interchanges. A bigger scale drawing of each interchange design is presented in Appendix A.

Synchro was used to provide the optimum values of signal timing and cycle length in each test. Building precise simulation networks is one of the important steps on the way to a useful SSAM result. Based on previous experience (Gettman and Head 2008; Huang et al. 2013), some conflicts might be observed with the TTC equal to 0 (TTC = 0 means a crash) when the link and connectors are not drawn well or if there are overlapped links in the model. To minimize this sort of error, all the designs tested in this research were drawn first in AutoCAD and modeled precisely in VISSIM.

The lengths of the network on each leg were 5280 ft (1 mile) and 1600 ft for vehicles and pedestrians, respectively. Regarding the pedestrian crossing pattern, as indicated by Fig. 9, the outside crossing was chosen for all the interchanges except the DDI, where a center crossing was simulated due to its popularity in comparison to the outside crossing.

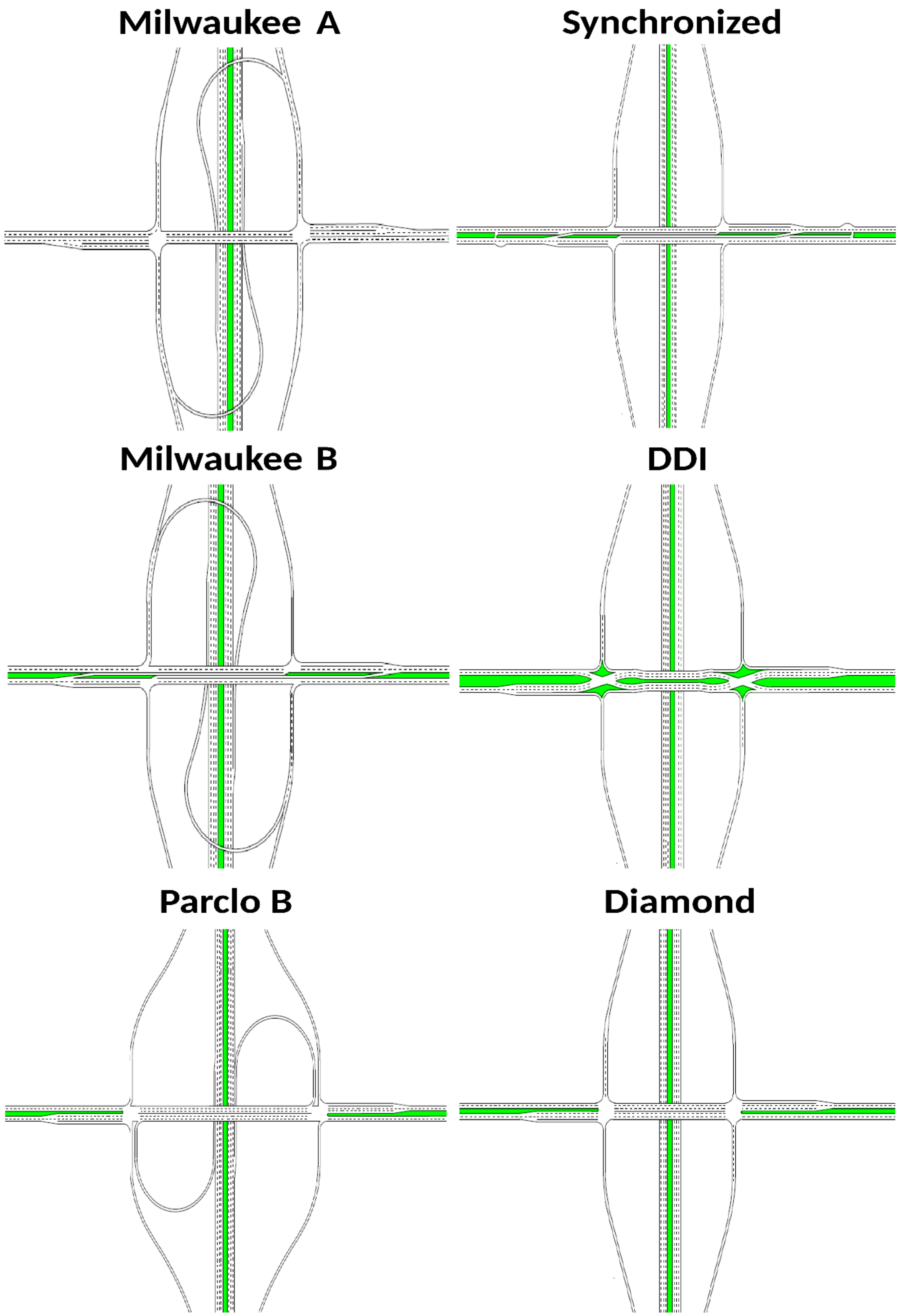


Figure 11- Geometry of the interchanges considered (not to scale)

3.4. Users Behavior

The speed of cars was 70 mph on the freeway while it was defined as 60 mph for trucks. Both cars and trucks had a speed of 35 mph on arterial. The selected speeds are typical in the US for these classes of roads. Fitzpatrick et al. (2006) estimated the 85th percentile free-flow speed of vehicles between 13 to 21 mph and 17 to 29 mph in the center and approach of exclusive right-turn lanes, respectively. Fitzpatrick et al. (2006) also conducted a series of statistical analysis (ANACOVA, ANOVA, and regression) to present new equations of vehicle's speed at the beginning and center of the right-turn lane. Based on their results, there is a significant relationship between radius and vehicle' speed at the beginning of right turn while right-turn lane length can be introduced as another significant independent variable (in addition to radius) on vehicle's speed at the center of turn in an alpha of 0.05. Their proposed equation was:

$$V85BT = 28.16 - 1.62Chan + 0.51CR - 0.03Len + 0.67Wid \qquad \text{eq. (1)}$$

Where

V85BT: 85^{th} percentile free-flow speed near the beginning of the right turn (km/h);

Chan: Channelization present at site (Chan=0 for raised island and 1 for lane line);

CR: Corner radius (m);

Len: Length of right-turn lane (m); and

Wid: Width of the right-turn lane at the start of right turn (m)

Wallwork (2004) revealed that the turning speed of vehicles could be placed in a group of 14-18 mph on the 90-degree angle intersections. Because all the crossovers except in the DDI were at a 90-degree angle based on the recommendation of the

Green Book (AASHTO 2011), the turning speeds of vehicles were set at 20 mph on approaches and 15 mph in the center of turns. Right-turn traffic was allowed to make a right turn when there was a minimum gap time of three seconds or more in traffic on the main route.

The acceleration of vehicles is not constant on ramps and drivers usually tend to have a higher rate when the speed is low at the beginning of on-ramps or when they are approaching a freeway merging area (Yang et al. 2015). For this reason, each ramp contained two curves with radii of 700 ft and 1400 ft to facilitate the speed transition between the arterial and the freeway. These radii provide an appropriate condition for traffic flow with a maximum superelevation rate of 6% (the maximum rate in Michigan) for design speeds of 45 mph and 65 mph. Mean vehicle speeds were set at 70 mph for passenger cars and 60 mph for trucks on the freeway, while both experienced the same speed of 35 mph on arterial. The assumed speed transition of passenger cars and heavy vehicles on directional ramps in this research is illustrated in Figure 12. Note that the ramps' length (horizontal axis) is based on a percentage (%). For instance, passenger cars reduce their speed from 70 mph to 60 mph after passing a quarter of ramp's length.

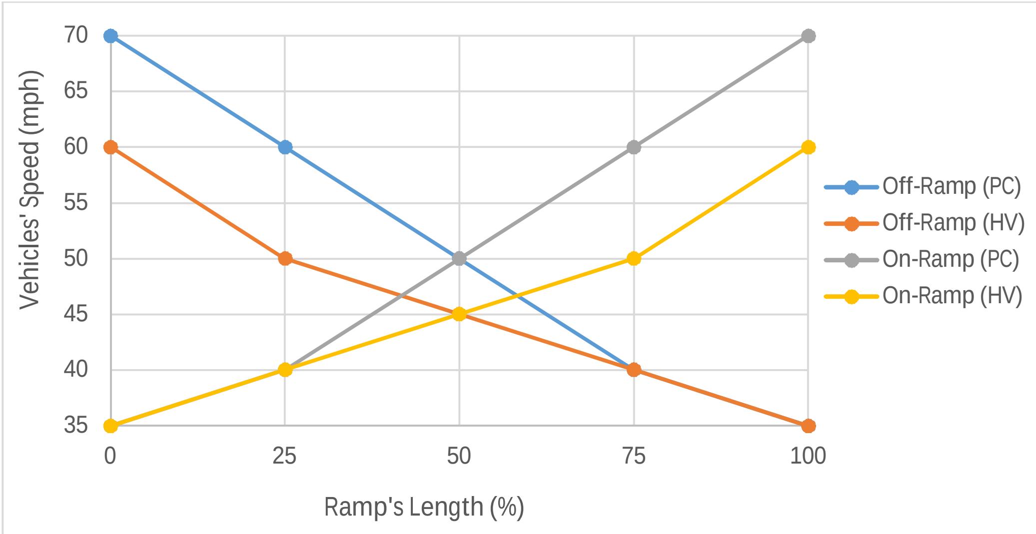


Figure 12- Vehicles' speed transition on the directional ramps

The speed transition on loops is different, and they are not designed to provide high speeds, and drivers should reduce their speed immediately at the beginning of the loop and their speed reaches the low point in the middle of the ramp. Therefore, another speed transition was assumed for loop ramps as Figure 13. All the involved interchanges in this research obey these speed patterns in order to make a fair comparison. Both the Fig. 12 and 13 were defined by the author.

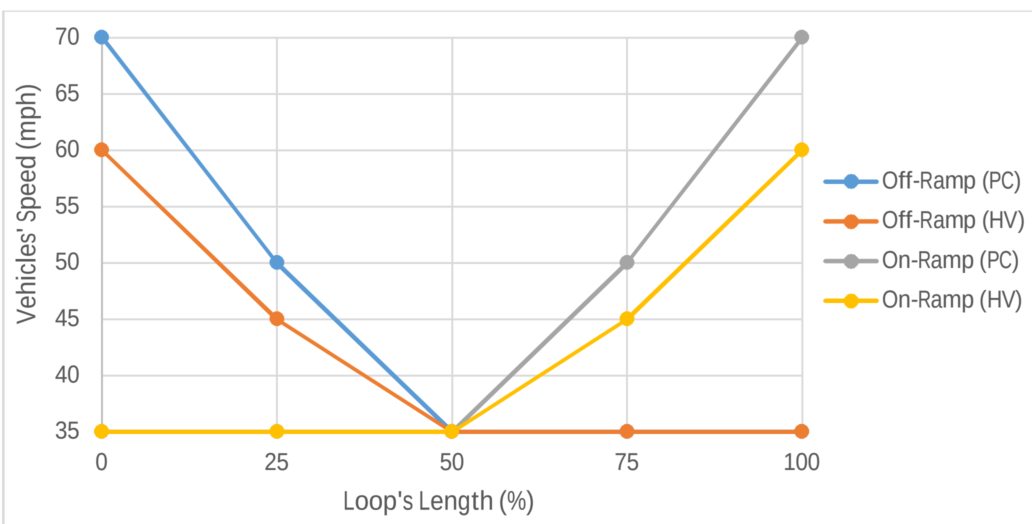


Figure 13- Vehicles' speed transition on loops

Turning speed, lane selection, and priority rule are some of the most important behavioral characteristics of drivers on U-turns. The U-turning speed was measured between 8 mph to 20 mph with a mean of 13.5 mph during a speed study of 422 passenger cars from 13 locations in Florida (Liu et al. 2012). Liu et al. (2012) also found that 80 to 100% of drivers prefer to make a U-turn to the right-most lane (the lane close to the outside shoulder) on four-lane roadways. Therefore, the current research used a U-turning speed of 15 mph and assumed that all the vehicles would turn to the right lane. Regarding the priority rule, like right turns, U-turn traffic could make the turn on red (a legal maneuver in Michigan) with a minimum gap of three seconds or more.

Pedestrian crossing speed depends on many traffic and non-traffic parameters (such as age, gender, type of crossing, conflicting traffic volume, time of day, the day of the week, etc.) and pedestrians usually trend to adjust their speed based on these variables at the particular moment and location. Based information from previous studies, pedestrian speed follows a normal distribution with the majority of speed observations (about 70-80%) near the average. Marisamynathan and Vedagiri (2013) found most pedestrian speeds in their sample between 4 fps to 4.5 fps, while about a range from 4.6 fps to 5.8 fps was noted by Ishaque and Noland (2009). A study in Poland found the range of pedestrian speeds from 3.6 to 4.6 fps (Oskarbski et al. 2016). The current research relied on the field data collected during a previous study on pedestrian performance at superstreet intersections (Hummer et al. 2014), and the same speed graphs were used. In that study, pedestrians were categorized into two groups as “walking pedestrians” with 91% of the observations and the average speed of 5 fps, and “running pedestrians” with 9% of the observations and an average speed of

9.6 fps. Regarding the priority rule of vehicles and pedestrians on the free-flow crossings in DDI and Milwaukee A, vehicle drivers had to stop for the pedestrians when pedestrians could find a minimum gap of 3 sec or longer to initiate a crossing.

Right turn on red (RTOR) was allowed but turning vehicles had to yield to pedestrians in permissive (shared) green intervals in the tests related to pedestrian performance. There was also no jaywalking allowed in the models. Since pedestrian clearance time is an important component of safety, and there are different practices in setting that time around the US, the current research chose clearance time based on field data collection. The authors collected clearance time on all approaches at 25 intersections with pedestrian signals in urban and suburban areas of Detroit, Michigan. Reviewing the data, a wide range of clearance times were observed even in different locations with the same crosswalk length. For example, the range of clearance time in 3-lane crossings varied from 9 to 16 sec. The data confirmed that no single method is dominant and clearance time mostly depends on designer philosophy or factors not related to intersection geometrics. Mean clearance times of 10.3, 13.7, 17.2, and 18.7 sec were found at two-lane, three-lane, four-lane, and five-lane crosswalks, respectively. Therefore, in these simulations, the authors applied a 7-sec clearance time at one-lane crossings and added 3.5 sec for any additional lane.

All the signals had all-red and amber intervals set at two and four seconds, respectively, based on popular and recommended values in Michigan. The maximum of signal cycle length was considered 120 secs in the Synchro. We did not use higher cycle lengths due to increased pedestrian waiting and chance of committing a violation (like jaywalking) for pedestrians as well as raising the threat of spillback on the traffic flow on main roadways. All the minimum green times of pedestrians were considered during the signal design. Pedestrians at the on-ramp crossings at the diamond interchange had an extra protected green interval simultaneously with the green of off-ramps since there was no conflict between vehicles and pedestrians during that phase.

CHAPTER 4 DATA ANALYSIS AND DISCUSSION

This chapter elaborates on the main results of the data analysis on each of the MOEs. The results of each MOE were provided in the following sections. At the end of this chapter, a discussion on model validation has been provided.

4.1. Traffic Operation

This section compares the travel time values of all the involved designs to find the most appropriate choices for different traffic conditions. Then the section provides an elaboration on the performance of each interchange. Based on previous studies (Thompson et al. 2003; Eyler 2005; Olya et al. 2013, Schroeder et al. 2014), the authors chose travel time as the primary criteria to evaluate the competitor interchanges. Travel time is the most suitable measure of effectiveness (MOE) for interchanges because it considers the effects of different travel distances in interchanges based on their unique geometry (Thompson et al. 2003). To obtain the travel time, a square network with legs one mile long was used for all interchanges. After the modeling, outputs were imported into IBM SPSS 24 to conduct two-way Analyses of Variance (ANOVAs).

4.1.1 General Comparison

Since the research considered high values of V/C (V/C = 1 in DDI and V/C = 1 in diamond), some interchanges were not able to complete all the tests due to the lack of capacity. The statistical analysis only included the tests which the interchange could accommodate at least 90% of the entry traffic volume and the tests with less than this rate was removed. Table 7 shows travel times as well as the number of completed tests by each interchange. The mean travel times presented in Table 6 were based on factors weighted by the traffic volume of each movement. The Milwaukee B and parclo B were the only designs which completed all the attempted simulation tests, and the Milwaukee A finished all but two of its tests. The Milwaukee B, Milwaukee A, and parclo B had the best average travel times of 113, 123, and 132 sec/veh, respectively. The synchronized and DDI also had reasonable travel times of 144 and 142 sec/veh, respectively, while completing 83%, and 78% of their tests. The conventional diamond had the worst operation with an average 168 sec/veh travel time while completing only 48% of its simulation runs.

Table 7- Mean travel time values and the percentage of completed tests in each of the designs

Interchange Type	Overall		High Turning		Moderate Turning		Low Turning	
	Travel Time (sec)	Completed Tests (%)	Travel Time (sec)	Completed Tests (%)	Travel Time (sec)	Completed Tests (%)	Travel Time (sec)	Completed Tests (%)
DDI	142	78	138	75	140	93	155	68
Diamond	168	48	169	43	171	36	165	61
Milwaukee A	123	98	124	96	122	100	122	100
Milwaukee B	113	100	112	100	113	100	115	100
Parclo B	132	100	138	100	122	100	127	100
Synchronized	144	83	144	50	149	100	138	100

The performance of interchanges based on the mean travel time for each movement on the arterial and the freeway was also analyzed in Fig. 13. According to Fig. 13, the synchronized interchange was at the same level as the parclo B and Milwaukee A in serving through traffic on the arterial, and it also showed lower travel times than the parclo B and the DDI for left turns from arterial. However, as was expected, the performance of the synchronized interchange on left turns from the freeway was considerably worse than the other designs. The Milwaukee B demonstrated the best performance for all movements except for left turns from the arterial, where the Milwaukee A had slightly lower travel times because those left turns encounter little conflicting traffic.

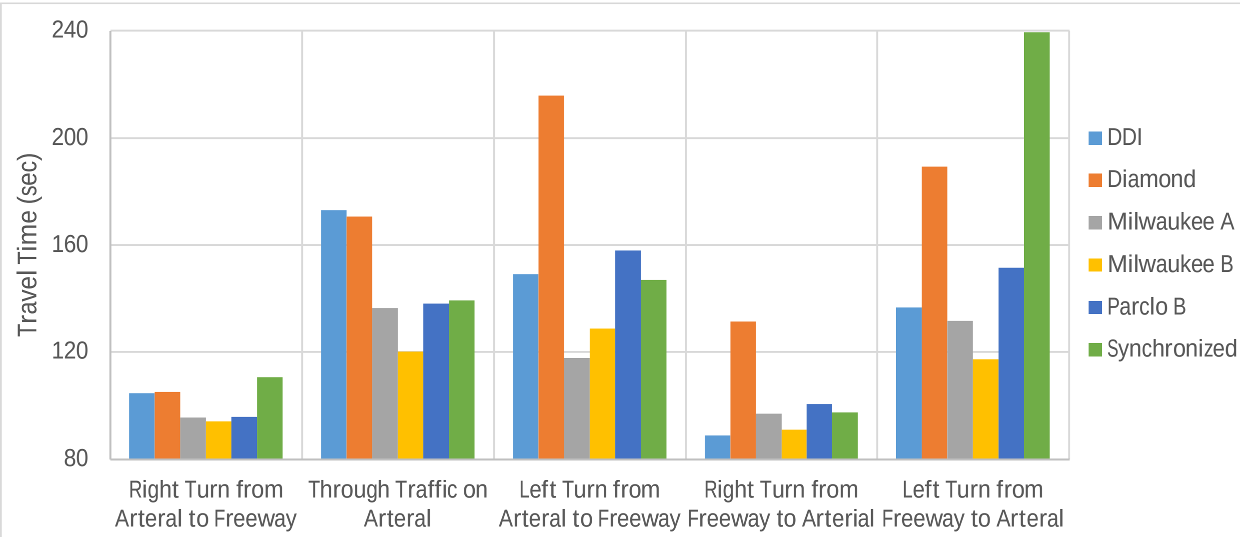


Figure 14- Mean travel time values of interchanges on each direction

4.1.2 The Comparison in Different Conditions of Turning

Table 8 provides the mean differences in travel time between pairs of interchanges and ANOVA statistical significance results. Highlights from Table 8 include:

- In all the three turning cases, the Milwaukee B, Milwaukee A, and parclo B have the top ranks. The mean travel time of Milwaukee B is always significantly different from the other designs.
- The parclo B travel time gets relatively better as the turning percentage decreases. The parclo B average travel time was 14.6 sec/veh worse than the Milwaukee A with a high turning percentage, for example, but the difference was reduced to 4.5 sec/veh with a low turning percentage.

Table 8- ANOVA with post hoc tests for travel time per interchange design

Interchange Type	Compares with…	Mean Difference (sec/veh)			
		Overall	High Turning	Moderate Turning	Low Turning
DDI	Diamond	-25.7	-39.1	-31.0	-8.98
	Milwaukee A	19.0	**8.35**	17.4	32.9
	Milwaukee B	28.9	20.5	27.2	40.7
	Parclo B	10.1	**-6.32**	9.81	28.4
	Synchronized	**-1.74**	-11.7	-10.0	18.0
Diamond	Milwaukee A	44.6	47.4	48.4	41.9
	Milwaukee B	54.6	59.7	58.2	49.7
	Parclo B	35.7	32.8	40.8	37.4
	Synchronized	23.9	27.4	20.9	27.0
Milwaukee A	Milwaukee B	9.99	12.2	9.76	7.87
	Parclo B	-8.89	-14.7	-7.64	**-4.50**
	Synchronized	-20.7	-20.0	-27.5	-14.9
Milwaukee B	Parclo B	-18.9	-26.9	-17.4	-12.4
	Synchronized	-30.7	-32.3	-37.2	-22.7
Parclo B	Synchronized	-11.8	**-5.38**	-19.8	-10.4

Bold represents the insignificant differences at the 0.05 level.

- The diamond is the worst interchange design for all turning percentages.
- Two of the interesting interactions revealed in Table 8 were between the DDI and the synchronized interchange and the DDI and the parclo B. The DDI performed better than both of the others with high turning percentages and worse than both of the others with low turning

percentages. It appears that these interchanges might be complementary to each other and designers can count on the synchronized or parclo B as a substitute to the DDI when there is high through percentage.

- Overall there was a significant difference in the mean value of travel time between all pairs of designs except between the synchronized and DDI.

4.1.3 Performance of Each Interchange

Table 9 shows the relationship between travel time and the various independent variables in the research based on ANOVA. The following paragraphs highlight some of the important points from Table 9.

V/C plays a key role on the travel time of DDI. The F value for V/C was estimated at 239 while the other variables had much lower F values. Among the interactions, the interaction between turning percentage and V/C had the greatest effect on travel time.

Turning percentage did not have a statistically significant effect on travel time for the diamond interchange. The most influential factors on travel time at diamonds were the truck percentage and V/C.

Table 9- Effects of variables on travel time

Type	Variable	F [a]	Sig [b]	Type	Variable	F [a]	Sig [b]
DDI (R^2=0.96)	Intercept	48900	0.000	Diamond (R^2=0.93)	Intercept	14000	0.000
	Turning Case	57.8	0.000		Turning Case	2.89	**0.080**
	Traffic Distribution	23.3	0.000		Traffic Distribution	5.27	0.001
	Truck %	19.1	0.000		Truck %	26.6	0.000
	V/C	239	0.000		V/C	17.8	0.000
	Turning Case-Traffic Distribution	2.60	0.001		Turning Case-Traffic Distribution	1.68	**0.131**
	Turning Case-Truck %	0.48	**0.620**		Turning Case-Truck %	1.93	**0.172**
	Turning Case-V/C	93.0	0.000		Turning Case-V/C	1.69	**1.134**
	Traffic Distribution-Truck %	0.41	**0.965**		Traffic Distribution-Truck %	2.96	0.016
	Traffic Distribution-V/C	9.48	0.000		Traffic Distribution-V/C	3.26	0.013
	Truck %-V/C	0.41	**0.523**		Truck %-V/C	1.58	**0.223**
Milwaukee A (R^2=0.88)	Intercept	89700	0.000	Milwaukee B (R^2=0.85)	Intercept	543000	0.000
	Turning Case	2.61	**0.079**		Turning Case	31.0	0.000
	Traffic Distribution	26.6	0.000		Traffic Distribution	15.2	0.000
	Truck %	2.10	**0.151**		Truck %	16.2	0.000
	V/C	86.1	0.000		V/C	3.42	**0.067**
	Turning Case-Traffic Distribution	0.54	**0.966**		Turning Case-Traffic Distribution	2.08	0.004
	Turning Case-Truck %	0.23	**0.791**		Turning Case-Truck %	1.02	**0.364**
	Turning Case-V/C	89.0	0.000		Turning Case-V/C	74.0	0.000
	Traffic Distribution-Truck %	0.07	**1.000**		Traffic Distribution-Truck %	0.39	**0.975**
	Traffic Distribution-V/C	2.38	0.007		Traffic Distribution-V/C	5.13	0.000
	Truck %-V/C	0.07	**0.785**		Truck %-V/C	1.80	**0.183**
Parclo B (R^2=0.95)	Intercept	365000	0.000	Synchroni-zed (R^2=0.89)	Intercept	94600	0.000
	Turning Case	259	0.000		Turning Case	80.6	0.000
	Traffic Distribution	39.0	0.000		Traffic Distribution	8.31	0.000
	Truck %	19.6	0.000		Truck %	25.8	0.000
	V/C	440	0.000		V/C	52.5	0.000
	Turning Case-Traffic Distribution	1.64	0.039		Turning Case-Traffic Distribution	0.94	**0.552**
	Turning Case-Truck %	3.81	0.025		Turning Case-Truck %	1.30	**0.280**
	Turning Case-V/C	306	0.000		Turning Case-V/C	142	0.000
	Traffic Distribution-Truck %	0.55	**0.895**		Traffic Distribution-Truck %	0.58	**0.872**
	Traffic Distribution-V/C	4.10	0.000		Traffic Distribution-V/C	1.80	**0.055**
	Truck %-V/C	1.62	**0.206**		Truck %-V/C	1.84	**0.179**

Dependent Variable: Travel Time

Bold represents the insignificant variables at the 0.05 level

[a] Variation Between Sample Means / Variation Within the Samples

[b] **Sig** determines whether any of the differences are significant at the level of 0.05.

The interaction of turning percentage and V/C was found to have a large effect on the travel time at a Milwaukee A. V/C, and traffic distribution was also important. However, turning percentage and truck percentage did not seem to affect the travel time significantly.

A surprising result from the Milwaukee B was its independence from V/C. The Milwaukee B has a high capacity, and it can accommodate relatively large volumes of traffic. Like the Milwaukee A, the highest F value for the Milwaukee B was for the interaction between turning percentage and V/C.

All the main effects were statistically significant for the parclo B, as were all but two of the interactions. The F values of V/C, turning percentage, and the interaction of turning percentage and V/C were considerably higher than for the other parameters and interactions.

Synchronized interchange results indicated that the interaction between turning percentage and V/C had the most influence on its travel time, while the other interactions were not very important. Turning percentage, V/C, truck percentage, and traffic distribution were all statistically significant main effects.

4.2. Pedestrians

The primary objective of this part was to study the performance of pedestrians in two new service interchanges in comparison to four existing designs. In addition, the research identified the impact of pedestrians on the vehicular travel time. There were only a few studies to this point (Milazzo et al. 1998; Banerjee et al. 2004; Hubbard et al. 2009) which focused on the impact of pedestrians on vehicle operation at intersections, so this effort hoped to build that knowledge base.

The following paragraphs describe different aspects of the results regarding vehicle and pedestrian operations.

4.2.1 Overall Pedestrian Performance

The mean travel times and the number of stops in each interchange are provided in Table 10 while Table 11 compares the mean values to recognize the significant differences among the designs. Regarding pedestrian travel time, the Milwaukee A was the best interchange which provided faster routes for the pedestrians by a slim margin over the diamond and the parclo B. The reason for the strong travel time performance of the Milwaukee A is the existence of only one signalized crossing for each route in the geometry; its other crossing is a free-flow one with the right-of-way for pedestrians. If pedestrians had to wait for vehicles at the free-flowing crossing, the result would be much different. The Milwaukee A performance was significantly better than the other designs in the high turning conditions, where its margin was wider over the diamond and parclo B.

Table 10- Mean values of pedestrians' travel time and stop in each interchange

Interchange Type	Overall		High Turning		Moderate Turning		Low Turning	
	Travel Time (sec)	Stops (no)	Travel Time (sec)	Stops (no)	Travel Time (sec)	Stops (no)	Travel Time (sec)	Stops (no)
DDI	386	2.05	380	2.13	386	1.96	391	2.07
Diamond	346	0.68	348	0.71	346	0.66	344	0.65
Milwaukee A	342	0.93	343	1.03	342	0.89	342	0.87
Milwaukee B	355	1.25	357	1.29	356	1.32	353	1.13
Parclo B	348	1.34	352	1.20	348	1.10	345	1.09
Synchronized	364	1.34	371	1.27	364	1.15	360	1.02

Table 11- Mean difference of pedestrians MOEs per design by ANOVA with post hoc tests

Interchange Type	Compares with…	Overall		High Turning		Moderate Turning		Low Turning	
		Travel Time (sec)	Stops (no)	Travel Time (sec)	Stops (no)	Travel Time (sec)	Stops (no)	Travel Time (sec)	Stops (no)
DDI	Diamond	39.9	1.37	32.5	1.41	40.0	1.29	47.1	1.420
	Milwaukee A	43.7	1.12	37.0	1.09	44.1	1.06	49.9	1.20
	Milwaukee B	30.3	.800	22.7	.831	29.6	.632	38.5	.936
	Parclo B	37.3	.918	27.9	.921	37.6	.854	46.4	.979
	Synchronized	20.9	.918	9.17	.892	21.6	.811	31.9	1.05
Diamond	Milwaukee A	3.81	-.254	4.58	-.313	4.08	-.231	**2.75**	**-0.21**
	Milwaukee B	-9.58	-.575	-9.75	-.580	-10.4	-.663	-8.58	-.483
	Parclo B	**-2.58**	-.457	-4.58	-.490	**-2.42**	-.441	**-0.75**	-.440
	Synchronized	-19.0	-.457	-23.3	-.520	-18.4	-.484	-15.2	-.369
Milwaukee A	Milwaukee B	-13.3	-.321	-14.3	-.267	-14.5	-.431	-11.3	-.265
	Parclo B	-6.39	-.203	-9.17	-.177	-6.50	-.210	**-3.50**	-.222
	Synchronized	-22.8	-.203	-27.9	-.206	-22.5	-.252	-18.0	**-0.15**
Milwaukee B	Parclo B	7.00	.118	5.17	**0.09**	8.00	.221	7.83	**0.04**
	Synchronized	-9.42	.118	-13.5	**0.06**	-8.00	.179	-6.67	**0.11**
Parclo B	Synchronized	-16.4	**0.01**	-18.7	**-0.02**	-16.0	**-0.04**	-14.5	**0.07**

Bold represents the insignificant differences in the level of 0.05.

The new Milwaukee B and synchronized interchanges had travel times that were 13 to 22 seconds higher than the Milwaukee A on average. The DDI had the worst performance in terms of travel time and the number of stops. The diamond was the best interchange regarding the number of stops, with an average of 0.68 per pedestrian, due to its long green interval in each cycle. After the diamond, the Milwaukee A and Milwaukee B had the lowest number of stops with the values of 0.93, and 1.25, respectively.

On average, the parclo B and synchronized interchanges had the same number of stops per pedestrian, but the parclo B did better with higher turning volumes, and the synchronized did better in low turning conditions.

The number of stops should be one of the most effective variables looking at pedestrian safety. Pedestrians likely commit more violations as the number of stops increase. To elaborate on this issue, Table 12 examined the waiting time of pedestrians multiplying half of the red interval by the number of stops. Note that the number of stops is used as a factor for the probability of facing a red light in this research since pedestrians had the right-of-way for crossing at any other conflict point with vehicles, so all the stops were because of red lights. The reason for using half the red interval was to consider an average stop length for the pedestrians assuming random arrivals.

Table 12- Cycle length, red light interval, and the estimated waiting time of pedestrians (sec)

Parameters	Overall			High Turning			Moderate Turning			Low Turning		
	CL [a]	**R [b]**	**Waiting Time**	**CL**	**R**	**Waiting Time**	**CL**	**R**	**Waiting Time**	**CL**	**R**	**Waiting Time**
DDI	75	43	44	61	36	38	76	42	41	89	49	51
Diamond	120	36	12	120	39	14	120	36	12	120	33	11
Milwaukee A	67	31	14	62	32	16	66	31	14	73	30	13
Milwaukee B	57	28	18	53	30	19	57	28	18	62	27	15
Parclo B	70	32	21	67	34	20	70	32	18	74	31	17
Synchronized	68	28	19	65	30	19	67	27	16	71	26	13

[a] Average cycle length of scenarios, (sec)
[b] Average red interval of pedestrians (clearance time of pedestrians is included), (sec)

Table 12 emphasized the poor performance of the DDI, with 3.6 times higher waiting times than the diamond which was the interchange with the lowest waiting times. There were three main reasons for predicting higher waiting time in the DDI in comparison to the other designs: (1) more number of stops based on Table 10, (2) the higher clearance time due to crossing longer crosswalks, especially in crossing the through traffic in the crossovers, and (3) lower ratios of green/cycle length (G/CL) since the pedestrians had to conflict with the main flow (through traffic) in the signalized crossings in the DDI, while the pedestrians of other designs had to be stopped only for the turning traffic and were receiving green time simultaneously with the through traffic of the arterial. The diamond had the lowest waiting times overall, and the Milwaukee A had the second best waiting times. The new interchange designs were superior to the parclo B for waiting times due to shorter red intervals. The synchronized interchange performed particularly well in low turning scenarios.

Based on the literature review (Oskarbski et al. 2016; Brosseau et al. 2013; Hubbard et al. 2009), the type, frequency, and size (length) of conflict points with vehicles are important parameters for pedestrian safety. The volume of conflicting traffic is also important to pedestrian safety. Table 13 shows the details regarding vehicle-pedestrian conflict points for each interchange considered.

The results that stand out in Table 13 are for the DDI, and the Milwaukee A. All the other designs provided the same performance with four crossings of six total lanes and a total conflicting volume of 2270 vehicles per hour. On the other hand, two of the conflict points in Milwaukee A are free-flowing crossings, and there are eight total lanes to cross. Most notably, because they cross and recross the through arterial lanes, pedestrians in a DDI face more and longer conflicting points with a significantly higher total conflicting volume.

Table 13- Vehicle-pedestrian conflict points for each interchange

Parameters	Free-Flow Crossing			Permissive Crossing			Protected Crossing			Total		
	N[a]	**L**[b]	**V**[c]	**N**	**L**	**V**	**N**	**L**	**V**	**N**	**L**	**V**
DDI	4	4	1514	0	0	0	4	20	2752	8	24	4266
Diamond	0	0	0	2	2	758	2	4	1512	4	6	2270
Milwaukee A	2	4	1516	0	0	0	2	4	1512	4	8	3028
Milwaukee B	0	0	0	2	2	758	2	4	1512	4	6	2270
Parclo B	0	0	0	2	2	758	2	4	1512	4	6	2270
Synchronized	0	0	0	2	2	758	2	4	1512	4	6	2270

a Number of crossings
b Total Length (number of lanes)
c Total Conflicting Volume (veh/hr)

Table 14 summarizes the results from Tables 10 through 13 in terms of how each of the six interchanges tested ranked on each of the four categories. Table 6 shows that the diamond is probably the best overall interchange for pedestrians, the Milwaukee B is next best with no major weaknesses. The Milwaukee A, the parclo B, and the synchronized interchange were at the next level with some good points but some weaknesses. The DDI was clearly the weakest performer of the six interchanges tested.

Table 14- Summarized results of pedestrian performance [a]

Parameters	Waiting Time	Conflicts with Vehicles	Number of Stops	Travel Time
DDI	6	6	6	6
Diamond	1	1	1	2
Milwaukee A	2	5	2	1
Milwaukee B	3	1	3	4
Parclo B	5	1	4	3
Synchronized	4	1	4	5

[a] the ranking is among the interchanges considered in this research

4.2.2 Effective Variables on the Pedestrian Performance

Table 15 presents the effect of traffic variables on pedestrian travel time and stops, including turning volume ratio, traffic distribution, the percentage of truck volume, and the interactions between them. Based on consistently high F-values and consistently low significance levels, the turning condition (high, moderate, or low turning cases) had the most influence on all the parameters on pedestrian travel time in all the interchanges.

The DDI and the synchronized interchange were the designs most sensitive to the turning case. The DDI performed better as the turning ratio raises while the synchronized showed an opposite reaction. The parclo B had the same behavior as the synchronized, with better performance in lower turning ratio cases, while the rest of interchanges did not change significantly with different turning ratios. Traffic distribution was found to be important to travel time for all designs. None of the interchanges showed a significant relationship at the 0.05 level between truck percentage and travel time. Among the two-way interactions, only the interaction between turning case and traffic distribution was statistically significant in most cases examined.

Table 15- Effects of traffic variables on the pedestrians MOEs of the existing interchanges

Type	Variable	Travel Time		Stops	
		F [a]	Sig [b]	F	Sig
DDI	Intercept	4240000	0.000	18600	0.000
	Turning Case	314	0.000	11.0	0.003
	Traffic Distribution	33.9	0.000	17.9	0.000
	Truck %	4.31	**0.064**	5.52	0.041
	Turning Case-Traffic Distribution	15.1	0.000	9.66	0.001
	Turning Case-Truck %	0.81	**0.470**	1.01	**0.396**
	Traffic Distribution-Truck %	1.56	**0.254**	1.38	**0.307**
Diamond	Intercept	6050000	0.000	20400	0.000
	Turning Case	43.2	0.000	16.8	0.001
	Traffic Distribution	16.8	0.000	6.25	0.007
	Truck %	1.91	**0.197**	0.08	**0.775**
	Turning Case-Traffic Distribution	7.09	0.002	6.17	0.004
	Turning Case-Truck %	6.71	0.014	0.01	**0.997**
	Traffic Distribution-Truck %	1.82	**0.196**	1.70	**0.221**
Milwaukee A	Intercept	19400000	0.000	11100	0.000
	Turning Case	27.1	0.000	31.1	0.000
	Traffic Distribution	11.8	0.001	2.61	**0.092**
	Truck %	1.15	**0.308**	0.72	**0.414**
	Turning Case-Traffic Distribution	2.74	**0.064**	1.12	**0.429**
	Turning Case-Truck %	3.46	**0.072**	1.19	**0.343**
	Traffic Distribution-Truck %	0.53	**0.744**	1.09	**0.421**
Milwaukee B	Intercept	102000000	0.000	705000	0.000
	Turning Case	1330	0.000	1560	0.000
	Traffic Distribution	98.5	0.000	0.25	**0.930**
	Truck %	2.50	**0.145**	5.00	0.049
	Turning Case-Traffic Distribution	62.5	0.000	676	0.000
	Turning Case-Truck %	2.50	**0.132**	5.00	0.031
	Traffic Distribution-Truck %	1.00	**0.465**	1.00	**0.465**
Parclo B	Intercept	20100000	0.000	3620000	0.000
	Turning Case	693	0.000	3610	0.000
	Traffic Distribution	478	0.000	4530	0.000
	Truck %	4.61	**0.057**	0.21	**0.651**
	Turning Case-Traffic Distribution	27.6	0.000	1070	0.000
	Turning Case-Truck %	1.15	**0.354**	1.52	**0.265**
	Traffic Distribution-Truck %	0.30	**0.897**	1.26	**0.352**
Synchronized	Intercept	172000000	0.000	621000	0.000
	Turning Case	13900	0.000	1850	0.000
	Traffic Distribution	1550	0.000	7.64	0.003
	Truck %	1.00	**0.341**	0.93	**0.357**
	Turning Case-Traffic Distribution	315	0.000	110	0.000
	Turning Case-Truck %	1.00	**0.402**	1.04	**0.387**
	Traffic Distribution-Truck %	1.00	**0.465**	1.02	**0.454**

Bold represents the insignificant variables in the level of 0.05

4.2.3 Impact of Pedestrians on Traffic Operation

As the last part of the evaluation in this research, the impact of pedestrians on vehicle travel time was analyzed. Table 16 presents the travel times extracted from VISSIM for runs with 360 pedestrians per hour and runs without pedestrians. The interchanges with better vehicle travel time performance showed less vulnerability to the presence of pedestrians. According to an ANOVA conducted on the mean differences, the diamond was the only design with a significant difference at the 0.05 level between its results with and without pedestrians. The synchronized interchange and the DDI had mean differences of 6.6 sec, and 5.4 sec between pedestrian and no pedestrian cases, respectively, which were higher impacts than the Milwaukee A, Milwaukee B, and parclo B.

Table 16- Vehicle travel time in different conditions of pedestrian presence (unit: sec)

Interchange Type	**Overall**			**High Turning**			**Moderate Turning**			**Low Turning**		
	With Ped	**No Ped**	**Mean Diff**	**With Ped**	**No Ped**	**Mean Diff**	**With Ped**	**No Ped**	**Mean Diff**	**With Ped**	**No Ped**	**Mean Diff**
DDI	139	133	5.41	122	121	0.96	131	129	1.78	164	151	12.9
Diamond	190	172	20.3*	188	172	15.6	208	174	34.8*	178	167	10.6
Milwaukee A	122	121	1.14	117	114	2.50	122	120	2.50	129	128	1.58
Milwaukee B	113	112	1.81	112	110	2.08	113	113	0.67	113	112	0.83
Parclo B	131	128	2.11	131	127	4.33	131	129	2.17	130	129	0.17
Synchronized	148	142	6.61	159	146	13.08	144	141	4.33	143	140	2.42

* The mean difference is significant at the level of 0.05

Table 16 also shows the Milwaukee B as the best interchange in terms of vehicle travel time either in the presence or absence of pedestrians. This confirms findings from the previous part related to traffic operation. Table 16 shows that the synchronized interchange can be a good substitute for a DDI in low turning cases while the DDI was more promising during higher turning scenarios.

4.3. Safety

The analysis was done based on three categories of evaluations: (1) the number of conflict points, unusual maneuvers, and the potential of wrong-way movements based on the geometric configuration of interchanges; (2) the frequency and type of simulated conflicts, the maximum speed of conflicting vehicles, and the TTC value obtained by SSAM; and (3) the number of vehicles stops extracted from the VISSIM simulation.

As a general comparison of the safety of interchanges, Table 17 reviewed the total number of conflict points, the number of unusual maneuvers, and wrong-way potential of each interchange geometry. Conflict points are interactions between directions (movements). The wrong way movement potential was examined based on five traits including 1) whether a median opening exists at an off-ramp terminal, 2) whether an off-ramp intersects the arterial at an acute angle, 3) whether a left turn lane is developed early (which might violate the expectancy of drivers), 4) whether the interchange seems unfamiliar for the users, and 5) whether there are two or four off ramps. The unusual maneuvers were also defined based on the number of directions which seems to be not clear for drivers. For example, a movement is labeled unusual when the driver who wants to make a left must turn right. According to Table 2, the Milwaukee B with 12 conflict points had the minimum among all the interchanges studied, while the conventional diamond had the highest with 18 conflict points. On the other hand, the diamond seems fine in terms of unusual maneuvers and wrong-way movements, while the synchronized interchange has more unusual maneuvers and the DDI has the greatest potential for wrong way movements.

Table 17- General safety features based on the geometry of interchanges

Interchange	Number of Conflict Points	Number of Unusual Maneuvers	Wrong way Potential
DDI	14	2	High
Diamond	18	0	Low
Milwaukee A	14	2	Moderate
Milwaukee B	12	2	Low
Parclo B	14	0	Low
Synchronized	14	4	Low

4.3.1 The Comparison of the Conflicting Interactions

Table 18 shows overall results from SSAM, while Table 19 shows the number of conflict results from SSAM broken out by turning scenario. Table 20 presents comparisons between the different interchanges regarding numbers of conflicts, and also shows whether the differences were statistically significant at the 0.05 level based on ANOVA. It should be mentioned that in some of their simulations the conventional diamond and the DDI were not able to process at least 90% of the entry traffic volume due to the lack of capacity.

Table 18- The comparison of frequency and severity of conflicting interactions in designs [a]

Interchange	Frequency				Max	Average
	Total	Crossing	Rear-End	Lane Change	Speed [b]	TTC [c]
DDI	468	1	379	90	5.71	0.72
Diamond	3340	30	2755	555	4.96	0.89
Milwaukee A	1800	564	1057	180	8.35	0.68
Milwaukee B	512	58	361	93	10.78	0.58
Parclo B	1823	543	986	294	9.76	0.75
Synchronized	2172	71	1660	441	4.61	0.93

[a] All the values are showing the average of total scenarios
[b] Maximum speed of conflicting vehicles (mph)
[c] Average TTC recorded (TTC threshold = 1.5 sec)

Table 19- The mean conflicting interactions of interchanges in different traffic turning cases

Interchange	High Turning Case				Moderate Turning Case				Low Turning Case			
	Total	Cros-sing	Rear-End	LC [a]	Total	Cros-sing	Rear-End	LC	Total	Cros-sing	Rear-End	LC
DDI	392	0	324	70	447	1	362	84	586	1	464	121
Diamond	2839	29	2283	526	3632	32	3061	538	3728	29	3079	619
Milwaukee A	1385	589	619	177	1656	577	907	173	2359	525	1646	189
Milwaukee B	508	71	340	97	534	59	382	92	494	42	361	90
Parclo B	1782	548	931	303	1902	561	1044	298	1784	521	983	280
Synchronized	2609	86	1984	540	2006	69	1513	424	1900	57	1484	360

[a] Lane Change

Table 20- ANOVA with post hoc tests for simulated conflicts per interchange design

Interchange Type	Compares with…	Mean Difference			
		Overall	High Turning	Moderate Turning	Low Turning
DDI	Diamond	-2872	-2447	-3185	-3141
	Milwaukee A	-1331	-993	-1209	-1773
	Milwaukee B	**-44**	**-116**	**-87**	**92**
	Parclo B	-1354	-1390	-1455	-1197
	Synchronized	-1703	-2218	-1559	-1313
Diamond	Milwaukee A	1540	1453	1975	1368
	Milwaukee B	2828	2331	3098	3234
	Parclo B	1517	1057	1729	1943
	Synchronized	1168	**230**	1626	1827
Milwaukee A	Milwaukee B	1288	877	1122	2359
	Parclo B	**-22**	**-397**	**-246**	**575**
	Synchronized	**-371**	-1224	**-349**	**459**
Milwaukee B	Parclo B	-1311	-1274	-1341	-1290
	Synchronized	-1660	-2101	-1472	-1406
Parclo B	Synchronized	**-348**	-827	**-103**	**-115**

Bold represents the insignificant differences at the 0.05 level.

The diamond could not meet the 90 percent level in 21 of its 36 tests while there were two failed tests for the DDI. These failed tests were ignored in the analysis, and the tables in this research only represent the completed simulation tests. The results in Table 18 should be interpreted as related to the probability of crashes (the frequency of rear-end and lane change conflicts), and as related to the severity of crashes (the maximum speed of vehicles in the conflict, the TTC at the moment of conflict, and the frequency of crossing conflicts). According to the results, there is no doubt that DDI had the best performance, especially from the viewpoint of the frequency of crossing conflicts, due to the unique geometry of DDI with a sharp angle between conflicting traffic streams. A review of the maps of simulated conflicts for all interchanges showed that the main locations of crossing conflicts were where permissive-controlled traffic (like right turns from the freeway when the light is red) entered a through traffic stream. In fact, this type of conflict almost always occurred at an acute angle in the DDI since its islands deflected the angle of entering and exiting traffic

The Milwaukee B had a very similar performance to the DDI regarding the frequency of simulated conflicts, but its performance was not as good regarding the severity of conflicts since it had the highest conflicting speed as well as the lowest average TTC. The reason for the higher conflict severity in the Milwaukee B is likely because of the higher flow speed and level of service. Based on the traffic operation results, the travel time at a Milwaukee B was observed to be significantly lower than the other types of interchanges with the same demand level, which means a higher speed of travel.

There was no statistically significant difference in the overall conflict frequency between the parclo B, Milwaukee A, and synchronized interchanges based on Table 20. However, the synchronized interchange had an advantage relative to these other interchanges based on substantially lower expected crash severity, with the lowest conflicting speed and the highest TTC among all the designs. Table 19 highlights the relatively poor performance of the synchronized interchange at high levels of turning demand when the synchronized almost reached the same total conflict frequency of the diamond. It was no surprise that the diamond was the weakest interchange overall with the highest numbers of conflicts and nearly the highest average TTC.

4.3.2 The Effects of Traffic Variables on the Conflicting Interactions

The effect of various traffic factors on the number of conflicts, and the effects of two-way interactions were investigated using ANOVA and Table 21 shows the results. Note that the diamond was not included in this analysis since the number of completed tests by diamond was not enough to conduct an ANOVA with post hoc tests.

Surprisingly, the DDI was statistically independent of any traffic variables. It seems that the geometry of DDI plays the main role in its low conflict frequency. For the rest of designs, Table 21 shows that all the traffic variables were significantly related to the number of conflicts while most of the two-way interactions between variables were not.

The turning case was the most important factor in explaining variation in the number of conflicts at a Milwaukee A interchange. Reviewing this point with a look to Table 19 revealed that the low turning cases provided the most dangerous situation for the traffic at a Milwaukee A. In fact, the Milwaukee A was able to boost the flow of left-turn traffic from arterial to the freeway with its loops; however, the loops became superfluous at low levels of turning (from arterial) and the interchange performed similar to a traditional diamond.

The Milwaukee B was the design most vulnerable to a higher truck percentage. The truck percentage had the highest F-values in all the conflict types in comparison to the other traffic variables. The reason was the high capacity of the Milwaukee B which made it able to perform the same in different conditions of traffic. When most other

Table 21- Effects of traffic variables on the conflicting interactions in each interchange [a]

Type	Variable	Total		Crossing		Rear-End		Lane Change	
		F	Sig	F	Sig	F	Sig	F	Sig
DDI	Intercept	407	0.000	18.9	0.001	566	0.000	166	0.000
	Turning Case	1.47	**0.270**	1.48	**0.268**	1.51	**0.262**	1.51	**0.262**
	Traffic Distribution	0.36	**0.861**	0.32	**0.890**	0.35	**0.870**	0.32	**0.885**
	Truck %	0.32	**0.578**	0.58	**0.460**	0.59	**0.455**	0.02	**0.881**
	Turning Case-Traffic Distribution	0.22	**0.983**	1.64	**0.216**	0.18	**0.991**	0.41	**0.901**
	Turning Case-Truck %	0.37	**0.665**	1.88	**0.271**	0.57	**0.411**	0.62	**0.449**
	Traffic Distribution-Truck %	0.07	**0.989**	2.42	**0.111**	0.12	**0.971**	0.13	**0.965**
Milwaukee A	Intercept	14800	0.000	64800	0.000	6450	0.000	15200	0.000
	Turning Case	386	0.000	79.5	0.000	539	0.000	10.5	0.003
	Traffic Distribution	278	0.000	4.11	0.027	331	0.000	34.6	0.000
	Truck %	16.8	0.002	35.5	0.000	8.95	0.014	30.2	0.000
	Turning Case-Traffic Distribution	69.4	0.000	4.71	0.011	83.8	0.000	4.00	0.019
	Turning Case-Truck %	1.43	**0.284**	0.98	**0.408**	1.95	**0.192**	8.10	0.008
	Traffic Distribution-Truck %	2.58	**0.095**	2.32	**0.120**	3.14	**0.058**	2.78	**0.079**
Milwaukee B	Intercept	17200	0.000	625	0.000	16100	0.000	9460	0.000
	Turning Case	8.94	0.006	13.8	0.001	18.2	0.000	3.60	**0.066**
	Traffic Distribution	2.80	**0.078**	8.20	0.003	0.91	**0.508**	3.67	0.038
	Truck %	85.0	0.000	10.0	0.010	59.6	0.000	48.9	0.000
	Turning Case-Traffic Distribution	3.37	0.034	3.55	0.029	0.61	**0.771**	9.75	0.001
	Turning Case-Truck %	2.24	**0.156**	0.54	**0.596**	4.07	**0.051**	9.82	0.004
	Traffic Distribution-Truck %	0.94	**0.493**	6.62	0.006	1.33	**0.325**	3.04	**0.063**
Parclo B	Intercept	8920	0.000	10200	0.000	6630	0.000	4370	0.000
	Turning Case	4.26	0.046	4.74	0.035	7.30	0.011	2.55	**0.127**
	Traffic Distribution	53.0	0.000	30.2	0.000	56.7	0.000	27.1	0.000
	Truck %	10.2	0.009	11.4	0.007	8.84	0.014	3.01	**0.114**
	Turning Case-Traffic Distribution	5.10	0.008	3.82	0.023	6.92	0.003	3.62	0.027
	Turning Case-Truck %	1.77	**0.218**	4.05	**0.051**	1.59	**0.251**	0.24	**0.787**
	Traffic Distribution-Truck %	0.34	**0.873**	0.51	**0.758**	0.34	**0.873**	0.66	**0.659**
Synchroniz-ed	Intercept	2670	0.000	1240	0.000	2450	0.000	3300	0.000
	Turning Case	27.7	0.000	17.1	0.001	23.4	0.000	47.0	0.000
	Traffic Distribution	8.35	0.002	2.05	**0.156**	11.0	0.001	2.45	**0.106**
	Truck %	8.76	0.014	9.91	0.010	7.87	0.019	9.66	0.011
	Turning Case-Traffic Distribution	12.8	0.000	2.03	**0.138**	16.3	0.000	3.64	0.027
	Turning Case-Truck %	0.66	**0.534**	0.91	**0.432**	0.66	**0.535**	0.72	**0.507**
	Traffic Distribution-Truck %	1.13	**0.404**	1.19	**0.377**	1.21	**0.372**	0.67	**0.652**

[a] Post hoc tests were not performed for diamond due to its few number of completed tests
Bold represents the insignificant variables in the level of 0.05

variables were accounted for, the role of trucks stood out more due to their bigger size and lower speed (on the ramps and the freeway) in comparison to passenger cars.

Traffic distribution was the factor tested which had the most effect on conflicts at a parclo B. The authors believe the reason is related to the progression system of the parclo B. Despite the fact that parclo B has one of the best progression system among all the conventional interchanges, its signals are dependent on traffic from four different approaches (through traffic on the arterial and traffic from the freeway off ramps). This means that the signals controlling each direction of the arterial do affect each other to some extent, and that certain traffic distributions either help or hurt performance through those signals.

Like the Milwaukee A, the number of conflicts at a synchronized interchange was influenced the most by the turning case. The effect of turning case on the conflicting interactions of synchronized was clear in Table 18 as well, where the synchronized illustrated great performance in low-turning condition and a poor operation (similar to the conventional diamond) in high-turning scenarios.

4.3.3 The Comparison of the Number of Stops

As the last step of the analysis, the number of vehicles stops was recorded and analyzed from the VISSIM models. The number of stops affects the drivers comfort and may be associated with rear-end crashes (Thompson et al. 2003). Table 22 compared the number of stops at each interchange. There was some correlation between the number of stops results in Table 22 and the frequency of rear end conflict results from SSAM in Table 18. The exception was for the Milwaukee A, which was in the middle of the pack in rear end conflicts but experienced the lowest number of stops as shown in

Table 22, probably due to free-flow turning traffic from the arterial to the freeway. Of course, free-flow traffic onto or off of a ramp can be a serious threat at locations with pedestrian demand, as discussed in previous section. The DDI was second in terms of numbers of stops, with the Milwaukee B close behind. The parclo B and synchronized interchanges performed moderately for the number of stops, but the synchronized interchange again was relatively better with lower turning demands. As for the frequency of rear end conflicts, the diamond had the highest number of vehicle stops.

Table 22- Mean number of vehicles stops (in one hour) in each interchange

Interchange Type	**Overall**	**High Turning**	**Moderate Turning**	**Low Turning**
DDI	0.55	0.49	0.52	0.75
Diamond	0.84	0.86	0.85	0.77
Milwaukee A	0.38	0.37	0.38	0.38
Milwaukee B	0.57	0.58	0.59	0.55
Parclo B	0.62	0.64	0.63	0.60
Synchronized	0.73	0.80	0.71	0.69

4.4. Cost

As the last MOE considered in this research, an economic analysis was conducted to estimate the costs and benefits of each interchange. Of course, cost estimation is one of the most difficult and critical parts of any project since DOT budgets are so tight and needs for new or upgraded facilities are so great. An initial comparison of the costs of alternatives would be helpful to see if there are large differences and to see the relative ranking of the alternatives. This research aimed to compare the primary components of costs for building a new interchange in Michigan. For this purpose, the evaluation was divided into two groups: (1) infrastructure costs, and (2) operational benefits. The first group includes the construction and right of way cost of interchanges which is the main concern of transportation agencies and departments of transportation (DOTs) while the second section focuses on the variables related to user costs.

It should be mentioned that the following assumptions were considered in the analysis:

- Cost incurred to others during construction, such as lost business or added travel delay, were not considered,
- Maintenance of traffic on existing roadways during construction was not considered,
- Unusual or unforeseen construction difficulties and delays such as with materials, utilities, historic artifacts, environmental issues, etc., were ignored in the analysis,
- Some costs such as pavement markings, drainage, and guardrail installation were excluded due to their negligible effects on the total cost,
- Earthwork was not considered since no specific topography was targeted in the study, and
- The costs of increases or decrease in crashes during construction were not considered.

Thus, the cost estimate is for only the basic construction elements—bridge, pavement, and right of way (ROW)--for a generic case with no complications.

4.4.1 Infrastructure (DOT) Costs

The details of expenditures for building a new interchange in Michigan are provided by Tables 23, 24, and 25. Table 23 presents the bridge costs as those are one of the most expensive parts of any interchange. Note that all the unit costs of Table 23 and Table 24 were based on the most recent (revised in January 2017) cost estimate worksheet of Michigan Department of Transportation (MDOT 2017). The estimated price was observed the same (about $3.4 million) for all the interchanges except the Milwaukee A and B due to the two extra bridges. Since the Milwaukee A needed a narrower width for its first bridge (due to two fewer arterial lanes), its bridge price was estimated to be about $0.8 million less than the Milwaukee B.

Table 23- The estimated bridge costs of interchanges

Interchange	Number of Bridges	Bridge Width, ft [a]			Bridge Area [b] (sq ft)	Structure Cost [c] (Million $)	Pavement Cost [d] (Million $)	Total (Million $)
		#1	#2	#3				
DDI	1	104	-	-	14,560	3.20	0.23	3.43
Diamond	1	104	-	-	14,560	3.20	0.23	3.43
Milwaukee A	3	80	24	24	17,920	3.94	0.28	4.22
Milwaukee B	3	104	24	24	21,280	4.68	0.34	5.02
Parclo B	1	104	-	-	14,560	3.20	0.23	3.43
Synchronized	1	104	-	-	14,560	3.20	0.23	3.43

[a] Based on the number of arterial lanes (each =12 ft), two pedestrian paths (each = 10 ft), two rigid (or semi-rigid) guardrails with the required distance (each side) of 6 ft
[b] the bridge length was considered equal to 140 ft (eight lanes of 12 ft + 4 shoulders of 10 ft + median rigid guardrail with the width of 4 ft on the freeway)
[c] Unit cost of concrete Type = $220 per sq ft
[d] Pavement RC 12" = $16 per sq ft

Table 24- The estimated ramp costs of interchanges

Interchange	**Number of ramps**	**Ramps Area [a] (sq ft)**	**Pavement Cost [b] (Million $)**
DDI	4	220,000	3.52
Diamond	4	220,000	3.52
Milwaukee A	6	310,800	4.97
Milwaukee B	6	298,000	4.77
Parclo B	6	301,600	4.83
Synchronized	4	220,000	3.52

[a] Each ramp has two shoulders with the width of 8 ft. The length of loops already was considered in the bridge costs for the Milwaukee A and Milwaukee B.
[b] Pavement RC 12" = $16 per sq ft

Table 25- The estimated ROW cost of interchanges

Interchange	**ROW [a] (Acre)**	**ROW Cost (Million $)**		
		Undeveloped Land [b]	**Developed Land [c]**	**Average**
DDI	40	0.26	4.24	2.25
Diamond	40	0.26	4.24	2.25
Milwaukee A	40	0.26	4.24	2.25
Milwaukee B	40	0.26	4.24	2.25
Parclo B	75	0.49	7.95	4.22
Synchronized	40	0.26	4.24	2.25

[a] An average length of 50 ft was considered from the edge of pavement for the side slops of ramps
[b] $6,500 per acre
[c] $106,000 per acre

The cost of ramps was evaluated in Table 24. The ramp cost of the Milwaukee A, Milwaukee B, and parclo B were estimated to be about $1.3 to $1.5 million higher than the other interchanges due to the extra ramps for the left turn traffic. The Milwaukee A was more expensive than the Milwaukee B in this part because of its dual-lanes off-ramps.

ROW costs of interchanges were determined using a published report on average land values in the US in 2015 (MSN 2015). Note that the developed land was defined as the area with housing, roads, and other structures based on the published report. According to Table 25, ROW costs do not seem to be different in any of the interchanges except the parclo B which costs about two million dollars more. Readers should be aware that the synchronized interchange could end up being smaller than a standard diamond, DDI, Milwaukee A, or Milwaukee B since it has the potential to have its ramps pulled in toward the freeway depending on a number of factors such as sight distance for turning drivers. Also, the Milwaukee A and B have exactly the same footprint as a standard diamond and the only difference (related to the size) in comparison to the conventional diamond is their two extra loops. On the other hand, at a synchronized interchange a DOT may have to obtain more property, or at least negotiate for restricted access, along the arterial between the ramp terminals and the U-turn crossovers. The rough estimate in Table 25 that a synchronized interchange should have about the same ROW cost as the diamond and other designs considered thus could change to more or less ROW depending on the specific case.

Table 26 shows the total construction cost of interchanges based on the estimates of bridge, ramp, and ROW for each design.

Table 26- The estimated construction costs of interchanges in Michigan

Interchange	**Cost (Million $)**
DDI	9.2
Diamond	9.2
Milwaukee A	11.4
Milwaukee B	12.0
Parclo B	12.5
Synchronized	9.2

The diamond interchange, the DDI, and the synchronized interchange had the same estimated cost, $9.2 million, while the construction costs of the parclo B, Milwaukee B, and Milwaukee A interchanges were about $3.2, $2.8, and $2.2 million dollars higher, respectively.

4.4.2 Operational (Users) Benefits

Table 27 provides a summary of a literature review on the VOT. It is apparent from those studies that VOT is an important element in benefit-cost calculations for projects like interchange construction. Note that the popular method for estimating the VOT is by conducting user surveys. This method is known as stated preference (SP); however, there is also another method which estimates the value based on the realistic data from devices like the Global Positioning System (GPS). Small et al. (2005) concluded that there is no big difference between the estimation from the both methods while the SP is more popular and mostly easier for estimation. Based on Table 27, the VOT was chosen as $15 per hour. Then, as a comparison regarding the benefits of interchanges, the travel time savings of the designs in comparison to the conventional diamond (as the design with the worst travel time performance) were illustrated by Table 28. The $15 rate was multiplied by the values of Table 28, and the results were presented in Table 29 to show the value of the time saved when the conventional diamond gets improved to any of the other interchanges in this study. Note that the hourly traffic was considered as 10% of the average daily traffic (ADT) based on Roess et al. (2010) and the ADT was converted to annual average daily traffic (AADT) considering the adjustment factor of 0.89 (based on Michigan recommendation) for a weekday in June.

Table 27- The estimated value of vehicle travel time based on the previous studies

Research	**Country**	**Type of Data**	**VOT ($/hr)**
Asensio and Matas (2008)	Spain	SP	22.1
Small et al. (1999)	USA	SP	5.1
Lam and Small (2001)	USA	RP [b]	30.5
Brownstone and Small (2005)	USA	SP/RP	15.2
Small et al. (2005)	USA	SP/RP	16.1
Zhu (2010)	USA	RP	14
Sikka (2012)	USA	RP/SP	12.1
Devarasetty et al. (2012)	USA	RP	51
Carrion and Levinson (2013)	USA	RP	9.15
AVERAGE [e]	**USA**	**-**	**19.14**
MEDIAN	**USA**	**-**	**14.6**

[a] Stated Preference
[b] Revealed Preference

Table 28- The estimated value of vehicle travel time based on the previous studies

Improved by	Daily (hour) [a]				Annually (hour) [b]			
	Overall	Traffic Turning Conditions			Overall	Traffic Turning Conditions		
		High	Moderate	Low		High	Moderate	Low
DDI	390	594	471	136	126000	192000	153000	44100
Milwaukee A	677	720	735	636	219000	233000	238000	206000
Milwaukee B	829	907	884	755	269000	294000	287000	245000
Parclo B	542	498	619	568	176000	161000	201000	184000
Synchronized	363	416	317	410	117000	135000	102000	133000

[a] Hourly traffic was considered equal to 10% of ADT
[a] the daily traffic was assumed on a weekday in June. The adjustment factor for AADT = 0.89 based on MDOT)

Table 29- Value of travel time savings for improving the conventional diamond interchange [a]

Improved by	Daily ($) [a]				Annually (Million $)			
	Overall	Traffic Turning Conditions			Overall	Traffic Turning Conditions		
		High	Moderate	Low		High	Moderate	Low
DDI	5,850	8,910	7,060	2,040	1.89	2.88	2.30	0.66
Milwaukee A	10,200	10,800	11,000	9,540	3.29	3.50	3.57	3.09
Milwaukee B	12,400	13,600	13,300	11,300	4.04	4.41	4.31	3.68
Parclo B	8,130	7,470	9,300	8,520	2.64	2.42	3.02	2.76
Synchronized	5,450	6,240	4,800	6,150	1.76	2.03	1.53	2.00

[a] VOT = 15$/hour

As was predictable, the Milwaukee B had the highest benefits from travel time savings while the synchronized and DDI could be cheaper alternatives which were also able to save about two million dollars annually.

As the last analysis regarding the costs, benefits relative to the diamond were examined in two periods of short-term (1- year) and the long-term (5-year) in Table 30.

An annual discount (interest) rate of 3% was applied to this table based on U.S. inflation long-term average (Inflation data 2017). The net benefit evaluation revealed that the DDI and synchronized could be the best alternatives in the short-term with a benefit of about two million dollars; however, the Milwaukee B undoubtedly is the most beneficial design from a long-term viewpoint if all of the assumptions built into the analysis hold true.

Table 30- Net benefit for improving the conventional diamond interchange (Million $)

Improved by	1-Year Period				5-Year Period			
	Overall	Traffic Turning Conditions			Overall	Traffic Turning Conditions		
		High	Moderate	Low		High	Moderate	Low
DDI	1.89	2.88	2.30	0.66	10.03	15.29	12.21	3.50
Milwaukee A	1.09	1.30	1.37	0.89	18.56	19.88	20.32	17.30
Milwaukee B	1.24	1.61	1.51	0.88	22.69	25.02	24.39	20.42
Parclo B	-0.66	-0.89	-0.29	-0.54	13.36	11.96	15.74	14.11
Synchronized	1.76	2.03	1.53	2.00	9.34	10.78	8.12	10.62

4.4.3 Extra Design Considerations Regarding the New Interchanges

The main aim of this part was to elaborate on some other factors that might affect the construction costs of the new interchanges. This section presents information regarding the ROW, loop radii, median width, etc.

4.4.3.1 Milwaukee B

One of the most important geometric parameters of the Milwaukee B might be the loop radius since it plays a significant role in terms of safety, travel time, and cost. Designers have a trade-off in choosing the loop radius to provide the appropriate speed at a reasonable price. Table 31 summarizes the results of this trade-off. Note that the cost of the bridge was considered the same since the bridge length and width would be roughly the same for all the loop radii. Also, speeds higher than 45 mph were not considered because those high speeds are not popular for the loops (as well as the distance between ramps of more than 1300 ft is not typical) at service interchanges.

Table 31- Trade-off parameters regarding choosing the proper loop's radius on Milwaukee B

Loop Radius (ft) [a]	Loop Speed (mph)	Required Distance between Ramps (ft)	ROW		Pavement		Total (M$) [d]
			Required Area (Acre)	Cost (M$)	Required Length (ft) [b]	Cost (M$) [c]	
144	25	300	23	1.29	2394	0.92	3.80
231	30	480	33	1.86	2928	1.12	4.57
340	35	700	46	2.59	3500	1.34	5.52
485	40	1000	63	3.54	4260	1.64	6.77
643	45	1300	80	4.50	5088	1.95	8.04

[a] Maximum superelevation rate of 6% was considered based on Michigan DOT recommendation
[b] A 150-degree deflection angle was considered. The column represents the length of both the loops together.
[c] Pavement RC 12" = $16 per sq ft. Also, a 24-ft width was considered for each loop (1 lane + 2 shoulders of 6ft)
[d] the total bridge cost was included as $ 1.59 million for all the options based on Table 24

Based on Table 31, a wide range of cost from $3.8 million to $8.0 million was estimated based on the loop radius. As a general point of view, the high-speed loops might not be very beneficial; however, they can be one of the best options in improving the conventional parclo A or parclo B interchanges where a huge ROW is already provided. Of course, the sharpness of ramps (after the bridge structure in each direction of the freeway) is also an important factor which might limit the free space for building large-radius loops at parclo interchanges (especially, at parclo A interchanges). A large portion of the new loop for a Milwaukee B can be placed in what would have been the location of a parclo B loop ramp.

The distance between bridges is another important topic regarding the design of a Milwaukee B. The key point for selecting the distance should be the longitudinal grades of the loops. According to the Green Book (AASHTO 2011), a grade of more than 4% is not recommended for freeways. From another point of view, the bridge would be more expensive when a significant grade is located on the bridge. Therefore, considering the starting point of the bridge as the end point of the longitudinal grade, the minimum distance between bridges was estimated as 575 ft for reaching to the height of 23 ft (the bridge height). The length can be increased to 767 ft, and 1150 ft for grades of 3% and 2%, respectively, which raises the needed budget for pavement by about $ 0.15 million and $ 0.44 million, respectively. The ROW cost can increase as well if the angle of the ramps is sharp and extra space is required to the right of the freeway in which to fit a loop.

The proposed design of Milwaukee B has located a horizontal curve on the bridge. This usually makes the bridge more expensive in comparison to a straight bridge; however, no precise unit rate could be estimated for that since the cost increase depends on many parameters related to the horizontal curve such as the superelevation rate and its transition length. In fact, whether to use a curved or straight bridge is another important trade-off which must be determined by designers based on the specific situation of each project. As an alternative to avoid the occurrence of a horizontal curve on the bridge, the distance between ramps can be increased by as much as the length of the bridge (140 ft in this research) to provide an straight alignment. It means that the bridge would be straight by placing half of the loop on each side (before and after) the bridge. Considering the length of bridge equal to 140 ft in the ROW and pavement calculations, an extra expense of about $ 0.56 million was estimated for providing straight bridges on the Milwaukee B used in this research. The estimated price was calculated based on an extra 8 acres of ROW and 6,720 sq ft of pavement in this case. The author believes that it does not seem worthwhile to design straight bridges in most cases and the expense of locating a curve on the bridge would likely be much cheaper.

Another critical point in the design procedure of the Milwaukee B and synchronized interchanges is in the median between contraflow lanes. A strong barrier between directions of travel is recommended to reduce the driver errors related to wrong-way movements. However, more analysis (such as in a driver simulator) is required to determine how effective a strong median barrier between the contraflow lanes would work in comparison to an undivided roadway. In terms of cost, a strong barrier is estimated to increase the cost of a bridge by about $ 0.12 million and this amount can be raised to $ 0.36 million if the designer wishes to separate the contraflow lanes from oncoming (through) traffic with a barrier as well. Mountable barriers might be another option (instead of strong barriers) since they are more suitable for emergency vehicles and reduce space on the bridge.

4.4.3.2 Synchronized

This research considered a 600-ft distance between the ramps for synchronized since this length seemed to be the most appropriate choice to make a fair comparison among all the interchanges. However, there is no serious limitation to using the synchronized interchange with shorter distances between ramps (such as 200 ft) which would lead to a smaller ROW. The threat of spillback (from long queues) might be the main concern related to using the synchronized interchange with shorter distances between ramps. For this purpose, the queue length between the signals on arterial were extracted from VISSIM as shown in Table 32.

Table 32- The queue length on the traffic signals of arterial in synchronized (unit: ft)

Queue Length	High Turning	Moderate Turning	Low Turning
Between WB1 & WB2 [a]	836	608	440
Between WB2 & WB3	252	100	81
Between EB1 & EB2	698	419	260
Between EB2 & EB3	205	66	53
Western U-turn Storage Lane	609	431	369
Eastern U-turn Storage Lane	753	603	457

[a] the order of signals is based on the direction of vehicles (EB1 is the first signal on the left side)

The queue lengths revealed that the synchronized interchange as simulated did not face any spillback problem since the maximum queue length between ramps (between WB2 and WB3 in Table 32) was found as 252 ft, 100 ft, and 81 ft in the different turning conditions (for high, medium, and low turning, respectively). Therefore, a distance about 250-300 ft between the ramps would have been beneficial choices for the synchronized interchange to minimize the ROW costs in the cases simulated. If we used a shorter distance between ramp terminals and a spillback problem developed we could also retime the signals to clear the queue, as at a tight diamond interchange, but that would add delay overall.

One of the important issues for the design of the synchronized interchange is related to the location of the U-turn crossovers. Based on the collected geometric data

(Table 6), the distance from the ramps to U-turn crossovers of superstreet intersections varies from a minimum of 200 ft to a maximum of 2100 ft. The mean and median from the collected data were 730 and 650 feet, respectively. Three factors should be considered in the selection of this distance: (1) the queue length of U-turning traffic and the adjacent signal on the other side, (2) distance between the ramp and the U-turn for lane changes, and (3) the adjacent driveways.

The maximum queue lengths in the storage lanes of the U-turn crossovers were estimated as equal to 753 ft, 431 ft, and 369 ft for the high-turning, moderate-turning, and the low-turning cases simulated, respectively. On the other side of the U-turn, maximum queue lengths of 836 ft, 608 ft, and 440 ft were observed between the U-turn crossover and the ramp terminal for the high-turning, moderate-turning, and the low-turning cases simulated, respectively. This shows that the existing 800-ft distance from the ramps to U-turn crossovers was not always sufficient in high turning cases while designers can consider shorter distances in moderate and low turning conditions

Another design consideration for the synchronized interchange is the required width for the U-turns. Based on the presented geometric features for median U-turns in the Green Book (AASHTO 2011) the dimensions of the design vehicle are the main factors in determining the width of U-turn. In this research, a large truck was considered as the design vehicle and a 24-ft roadway width with a 45-ft loon radius could provide the appropriate design for the turning vehicles. Without a loon, the minimum width of arterial for accommodating the U-turns can be estimated to be about 75 ft considering two lanes in each direction as well as one U-turn lane. In comparison to a conventional diamond with two lanes in each direction, an extra space of about 43,000 sq ft (with the distance between ramps and U-turn crossovers = 800 ft) is required for a synchronized interchange which costs about $ 0.15 million (ROW cost = $60,000, pavement cost = $90,000).

4.4.3.3 Placing of the New Interchanges

One of the advantages that both the new interchanges provide is that their signals are independent on each side of the arterial. This point makes the designer able to have different types of symmetric and non-symmetric designs by shifting the ramps in the cases with ROW limitations.

As a test of the possibility of replacing failing conventional interchanges with the new ones, the geometric features of 28 selected interchanges in Michigan were compared with the required dimension of the new interchanges to introduce some suggestions for the improvement projects. It must be mentioned that the provided suggestions were just based on the geometry and no traffic analyses were included in this part. No special problem was found in replacing 21 of the interchanges and the main issue regarding the other seven interchanges was the large budget for improvement. In fact, the Milwaukee B generally cannot be recommended when the freeway is located on the top of arterial since substantial earthwork is required to make an underpass for the loops or the loops would have to be built on the third level. The main concern for the synchronized interchange was related to locating the U-turn crossovers as they might be costly in locations with narrow arterials.

Table 33- Test of replacing the new interchanges with the existing interchanges

Location of the Interchange	Existing Design	Length btw Ramps	Suggested Alternative	Suggested Loop's Speed	Any Important Considerations?
Haggerty Connector & 12 Mile Rd	Parclo A	1400 ft	Milwaukee B	40 mph	Loop radius cannot be larger due to the ROW limitations
I-275 & Ann Arbor Rd	Parclo A	1300 ft	Milwaukee B	45 mph	-
I-275 & Ford Rd	Parclo A	1500 ft	None	-	Freeway is located on the top of the arterial, a replacement would need huge earthwork
I-275 & Eureka Rd	Parclo A	1200 ft	Synchronized	-	Milwaukee B is not possible due to ROW limitations
Haggerty Rd & Detroit Industrial Expy	Parclo A	1400 ft	Milwaukee B	45 mph	-
I-275 & 6 Mile Rd	Parclo A	1300 ft	Milwaukee B	45 mph	-
Belleville Rd & Detroit Industrial Expy	Parclo A	1600 ft	Milwaukee B	30 mph	Angle of ramps would not allow a larger loop
I-96 & Novi Rd	Parclo A	1000 ft	Milwaukee B	35 mph	Angle of ramps would not allow a larger loop
I-96 & Fowlerville Rd	Parclo B	1250 ft	Milwaukee B	40 mph	It would be expensive to buy the adjacent commercial stores
I-75 & 14 Mile Rd	Parclo B	1250 ft	None	-	Freeway is located on the top of the arterial, a replacement would need huge earthwork
I-96 & Kensington Rd	Parclo B	1250 ft	None	-	A replacement would be expensive
I-96 & Milford Rd	Parclo AB	1200 ft	None	-	Freeway is located on the top of the arterial, a replacement would need huge earthwork
I-96 & Latson Rd	Diamond	1500 ft	Milwaukee B	40 mph	Angle of ramps would not allow a larger loop
I-275 & Ecorse Rd	Diamond	1600 ft	Milwaukee B	45 mph	-
US-23 & US-12	Diamond	1400 ft	Milwaukee B	30 mph	Angle of ramps would not allow a larger loop
I-275 & Sibley Rd	Diamond	1650 ft	Milwaukee B	45 mph	It would be expensive to buy the adjacent commercial stores
I-94 & Van Dyke Rd	Diamond	300 ft	None	-	Expensive to provide the required space for U-turns
I-94 & Candieux Rd	Diamond	300 ft	None	-	Expensive to provide the required space for U-turns
I-96 & S Wright Rd	Diamond	500 ft	Synchronized	-	-
I-96 & Jordan Lake Rd	Diamond	550 ft	Synchronized	-	The effect of the adjacent driveway should be considered
M-10 Fwy & Forest Ave	Diamond	250 ft	None	-	Expensive to provide the required space for U-turns
I-96 & Nash Hwy	Diamond	550 ft	Synchronized	-	-
M-10 Fwy & Linwood Rd	Diamond	250 ft	None	-	Expensive to provide the required space for U-turns
I-96 & 48th Ave	Diamond	500 ft	Synchronized	-	-
I-96 & 112th Ave	Diamond	500 ft	Synchronized	-	-
I-94 & Telegraph Rd	SPI	550 ft	Synchronized	-	-
I-96 & Beck Rd	SPI	550 ft	Synchronized	-	-
I-96 & S Wixom Rd	SPI	500 ft	Synchronized	-	-

Note that three existing SPIs were examined in Table 33 as well, since they can be another target for improvement. Of course, this research did not consider any traffic analysis to compare the SPI and the synchronized interchange; however, the advantage for the synchronized interchange in efficiently moving through traffic on the arterial is apparent.

4.5 Validation

Validation was not defined as a primary goal in the scope of this research since there were numerous previous studies that validated VISSIM, Synchro, and SSAM as mentioned in the literature review. Moreover, the existing research had reviewed the safety performance of two new designs which have not been built yet. Regarding the validation of VISSIM as a tool for this type of work, Schroeder et al. (2014) compared field data to simulation results for four DDIs and concluded that VISSIM simulation could provide satisfactory results in operational studies of DDIs. To support those results, in this effort travel times from VISSIM modeling were compared with the estimated real travel times from probe vehicle data at three existing service interchanges including one diamond and two parclo A designs. Morning and the afternoon peak hours of each interchange were analyzed by ten repetitions in VISSIM. The date of traffic counts was between 2012 to 2016 for the three interchanges (note that probe data became popular since the early 2010s and the available probe data mostly belongs to the last five years). The three interchanges examined consisted two parclo, and one diamond interchanges and all were chosen from Michigan. Unfortunately, there was no probe data for the only existing Milwaukee A yet. Assumptions made for this validation exercise included:

- Due to the inconsistency between the date of traffic counts and available signal timings, a Synchro model was used to obtain optimum values of signal data for using in VISSIM.
- All the operational speeds were selected based on the speed limit of the interchange.
- No pedestrians and bicycles were considered in the VISSIM modeling.

The comparison of travel time between VISSIM and vehicle probe data is summarized in Table 34. The mean difference between the measured and simulated travel times was 2.33 sec (higher for VISSIM) which demonstrated an insignificant difference at the 0.05 level with an F value of 1.22 in ANOVA.

Table 34- The comparison of travel time between VISSIM models and probe data

Location	**Type**	**VISSIM (sec)**		**Probe Data (sec)**		**Overall Mean Difference (sec)**	**Overall Mean Difference (%)**
		AM Peak Hour	**PM Peak Hour**	**AM Peak Hour**	**PM Peak Hour**		
I-94 Fwy@16 Mile Rd, MI	Parclo	27.1	34.2	24.9	32.4	2.0	6
M-10 Fwy@Linwood Rd, MI	Diamond	24.8	27.0	22.8	25.2	1.9	7
Telegraph Rd@Ecorse Rd, MI	Parclo	30.6	32.2	28.2	28.4	3.1	10

Fig. 14 indicates an example of the origin and destination of travel times in two directions (EB and WB) of an interchange modeled by VISSIM.

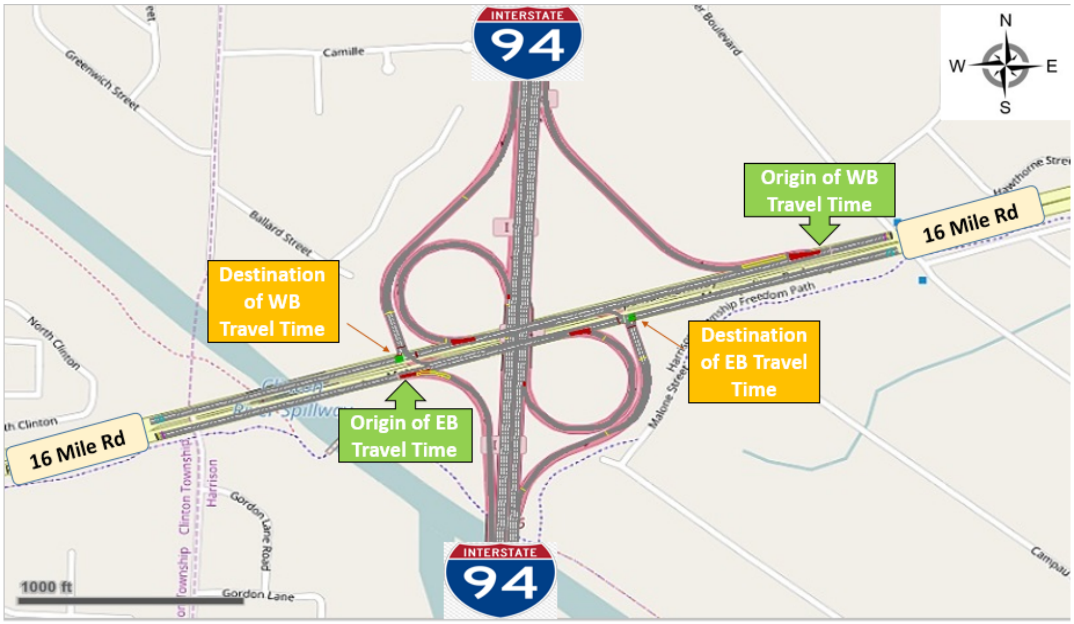


Figure 15- VISSIM model of the interchange I-94@16 Mile Rd in Michigan

Based on Essa and Sayed (2015), the validation of SSAM models was not found essential when the delay outcomes from VISSIM seem accurate. Essa and Sayed (2015) conducted a two-step calibration on VISSIM to increase the consistency between safety results of SSAM and field data. The first step focused on calibrating the delay time of VISSIM while the second part was related to driver behavior calibration. Interestingly, the effect of the first step was observed to be more significant for SSAM accuracy, and an appropriate correlation between simulated and field-measured conflicts can be expected even by ignoring the second step (driver behavior) of calibration. Therefore, the existing study conducted a validation only for the travel time performance and no validation was done for SSAM.

CHAPTER 5 DATA ANALYSIS AND DISCUSSION

This research evaluated the performance of two new interchange designs in comparison to four existing interchanges. The simulation experiment covered a wide range of traffic conditions. As the primary contribution of the research, both the new designs were introduced as appropriate alternative designs which can improve the failing conventional interchanges. Overall, the results should be helpful to highway agencies making choices on interchange design and trying to serve all roadway users well. This manuscript opened a new window to notify researchers, designers, and the transportation agencies that these new designs have potential and deserve attention in some cases.

5.1 Traffic Operation

The Milwaukee B, one of the new designs, showed the best performance among all the interchanges. Its mean travel time was an average of 9% lower than the Milwaukee A, and 17% lower than the parclo B, which was the best conventional interchange. The Milwaukee B was also the only design without a strong relationship to V/C: it performed well with moderate or high traffic demand.

The other new interchange design tested, the synchronized interchange, provided great performance when the through traffic was the dominant traffic volume in the interchange. In this condition, the travel time of the synchronized interchange was 20% lower on average than the DDI and 27% lower than the diamond. Since the synchronized, DDI, and diamond interchanges require approximately the same right-of-way (ROW), the synchronized design can be introduced as a substitute for a jammed diamond interchange with a dominant through traffic flow, while DDI might remain as a popular alternative when there is a high left turn volume ratio.

From a general point of view, the results of traffic operation can be divided into two groups based on the geometry of the interchanges tested. The Milwaukee B, Milwaukee A, and parclo B are the interchanges which need more ROW. The Milwaukee B provided the best operation in this category regarding travel time and can be introduced a substitute for the conventional parclo interchanges. On the other hand, in the division of smaller interchanges, the DDI and synchronized both performed better than the diamond and designers could choose the most appropriate alternative to the diamond based on the turning traffic ratio.

5.2 Pedestrians

The conventional diamond showed the best pedestrian operation among all the interchanges considered in the research. The Milwaukee B and synchronized interchanges trailed the diamond, Milwaukee A, and parclo B in travel times, but appear to offer relatively good pedestrian safety compared to other designs. Relatively poor pedestrian service is expected from the DDI since it got the worst ranking in all the MOEs. The diamond had the worst performance for vehicle travel time, and pedestrians affected the vehicle travel time at a diamond far more than at other designs, so a trade-off for the diamond between good pedestrian service and relatively poor vehicle service seems clear.

Turning volume ratio and traffic distribution was sometimes important variables in explaining travel time and the number of stops for pedestrians while the percentage of heavy vehicles was not found to be significant at the levels simulated.

At the levels tested, simulated pedestrians did not impact the vehicle travel time significantly except in the case of the diamond interchange. The impact of pedestrians seemed to be higher as the traffic operations became more critical to handle. For example, pedestrians increased the vehicle travel time only about one second for the DDI during high turning tests, but the impact was raised to 13 seconds in the low turning condition when the DDI should have a more difficult time handling the demand.

5.3 Safety

In general, all the interchanges involved in this research had 14 conflict points in their geometry except the conventional diamond with 18 and the new Milwaukee B with 12. Regarding the number of unusual maneuvers, the DDI, Milwaukee A, and Milwaukee B had two unusual movements while the synchronized had four. No unusual maneuvers exist in the conventional designs of diamond and parclo B.

The DDI and the new Milwaukee B were the safest interchanges from the viewpoint in terms of conflict frequency. They also did well for the number of stops. The geometry of the DDI reduces the conflicting angles between vehicles to provide the minimum number of crossing conflicts. On the other hand, the geometry might also cause safety problems regarding unusual maneuvers and wrong way movements due to its unique pattern in comparison to the other designs. The Milwaukee B seemed okay in this way; however, the main concern was related to the high speed of conflicts as well as the low TTC.

The new synchronized interchange performed well in some ways. It was similar in conflict frequency to the parclo B and Milwaukee A. However; it had the best performance on the SSAM parameters related to crash severity. The synchronized interchange was consistently a better performer with low turning conditions.

The traffic turning condition was found to be the most important variable in explaining conflict frequency for the Milwaukee A and synchronized interchanges, while the truck volume percentage and the traffic distribution has the highest impact on the conflict frequency for the Milwaukee B and parclo B, respectively. The conflict frequency did not show any dependency on any of the traffic variables for the DDI.

The conventional diamond had the worst performance in all safety aspects tested except the parameters related to the severity of crashes (conflicting speed and TTC). This was probably due to the low speeds in the tests of the diamond that were successfully concluded. Since the synchronized and DDI have almost the same size of right-of-way as the conventional diamond, they might be considered as safe alternatives for failing diamonds.

5.4 Costs

The construction cost of all the DDI, conventional diamond and synchronized interchanges was estimated about $9.2 million while the Milwaukee A, Milwaukee B, and parclo B interchanges were more expensive with costs of $11.4 million, $12 million, and $12.5 million, respectively. Travel time savings made the Milwaukee B the most beneficial alternative for improving a current failing interchange in the long term. The Milwaukee B is able to return a benefit of about $17 million in a 5-year period that more than covers its construction costs. The benefit should be more significant when the costs of crash reduction are combined with the benefits of travel time saving.

Based on a review of the possibility of replacing a sample of existing interchanges with the proposed new ones, no specific problem was seen in 75% of the locations; however, the distance between the ramps and the width of the arterial were identified as the key factors in fitting the Milwaukee B and synchronized designs. The synchronized interchange, due to its perfect progression system, did not experience long queues between its signals and a short distance (about 300 ft) between ramps can be enough to avoid a spillback threat. This point makes the synchronized a cheaper choice than DDI in its competition for being the most appropriate alternative to the conventional diamond interchange.

5.5 Summary

Table 35 has summarized the results of this research. Note that the overall ranking (the last column) was presented giving the same weight to each MOE, while the ranking can be different based on different policies of agencies.

Based on Table 35, Milwaukee B undoubtedly is the best design from an overall term of view. All the other interchanges have almost the same score but with different advantages and disadvantages. Designers should choose the most appropriate option based on the priorities and policies of projects. If the traffic operation and safety performance MOEs are considered to be most important, the conventional diamond would not be competitive with the other designs anymore.

Table 35- The summary of conclusions

Interchange	Traffic Operation	Pedestrian Performance [a]	Safety Performance	Cost Estimation [b]	Total Score (Stars)
Milwaukee B	******	*****	******	***	20
Milwaukee A	*****	*****	**	***	16
Parclo B	****	****	****	***	15
Synchronized	***	**	****	******	15
DDI	***	*	*****	******	15
Diamond	*	******	*	******	14

[a] the effect of pedestrian safety was ignored in this column since it is considered in the column of safety performance
[a] the effect of benefits was ignored in this column since it is considered in the column of traffic operation

5.6 Recommendations

As a recommendation for further studies, the drivers' behavior should be modeled in the new interchanges using a driving simulator laboratory. Driver behavior modeling might be essential research before constructing the new designs to analyze the reactions and feedbacks of drivers to the geometry and to design appropriate traffic control devices.

At the same time as this manuscript was being prepared, the North Carolina Department of Transportation was conducting the initial analyses of building the first synchronized interchange in North Carolina. Once the new designs have been built, it is clear that the before-after studies would be the primary task of researchers in the future to investigate the safety operation of new interchanges based on the real crash statistics.

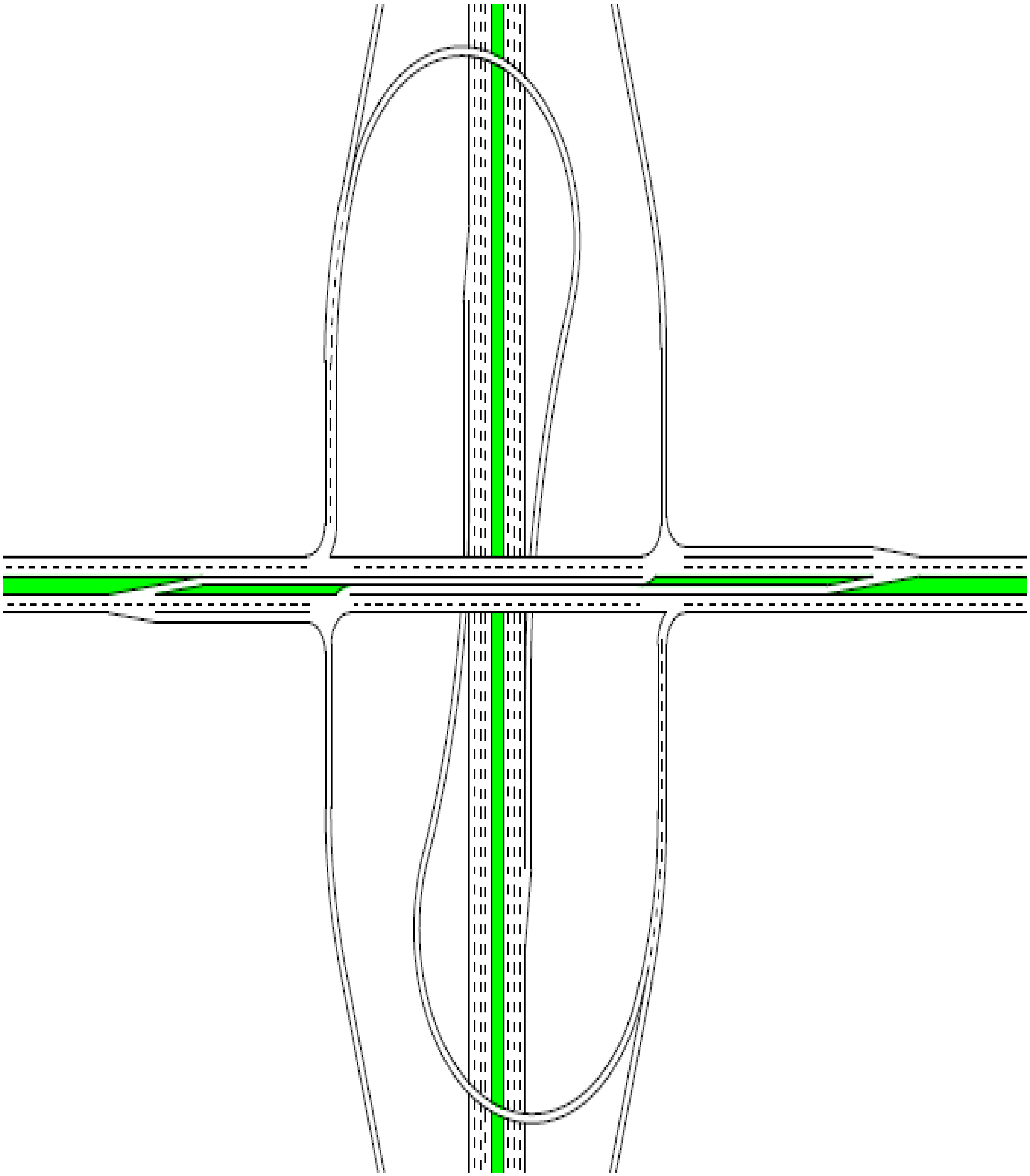
One of the required studies related to the synchronized design is investigating the effect of adjacent driveways (land uses and streets) on the traffic operation and safety of the segment between ramps and U-turn crossovers. This will be important for operations, safety, and for the impacts on businesses at those locations.

It is also recommended to conduct more studies on the new designs regarding the optimization of traffic signals. How to construct the new interchanges while maintaining traffic on an existing diamond would be interesting to study. In addition, the performance of new designs and other types of interchanges for driverless vehicles should be modeled in future research.

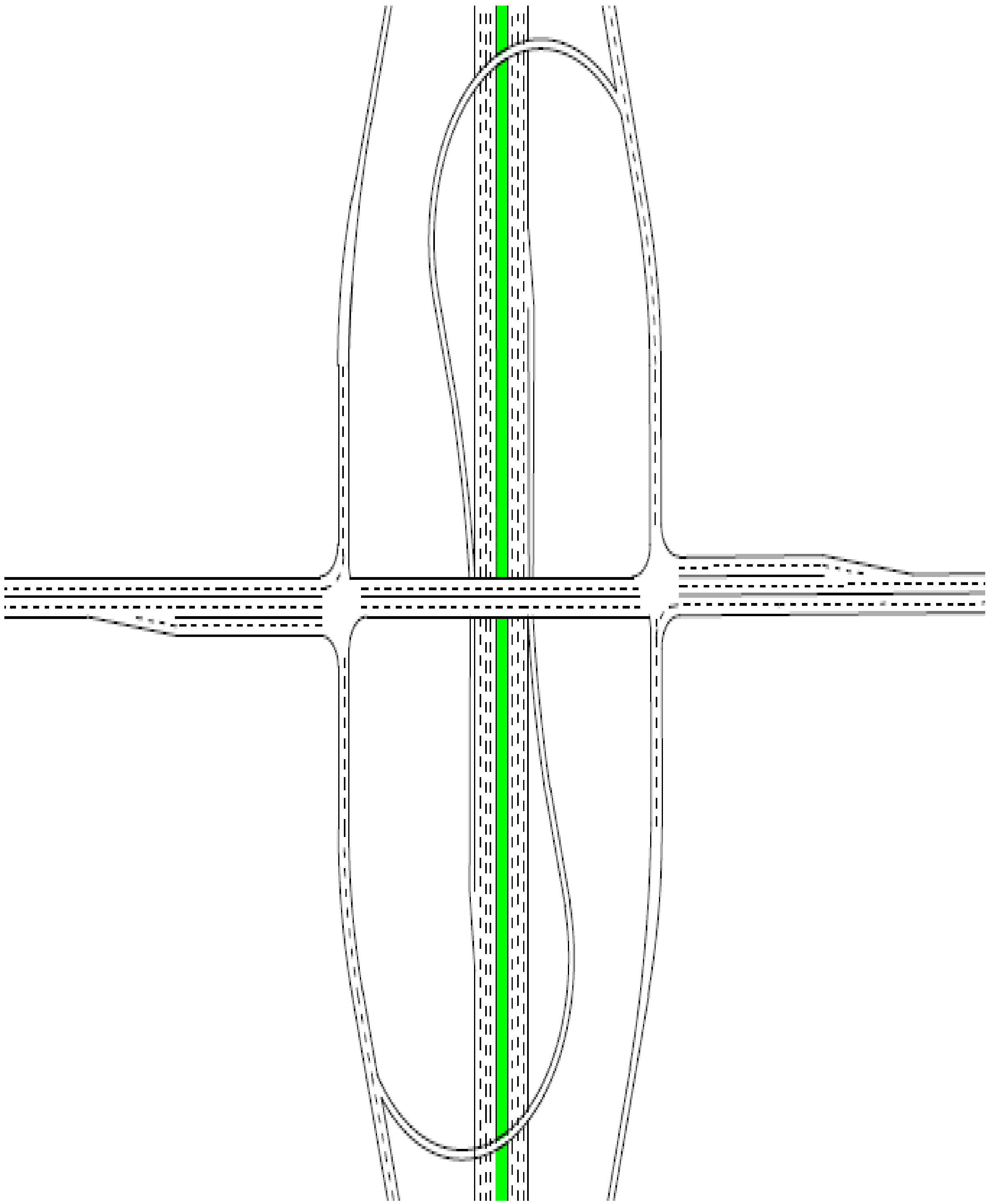
APPENDIX A

Drawings

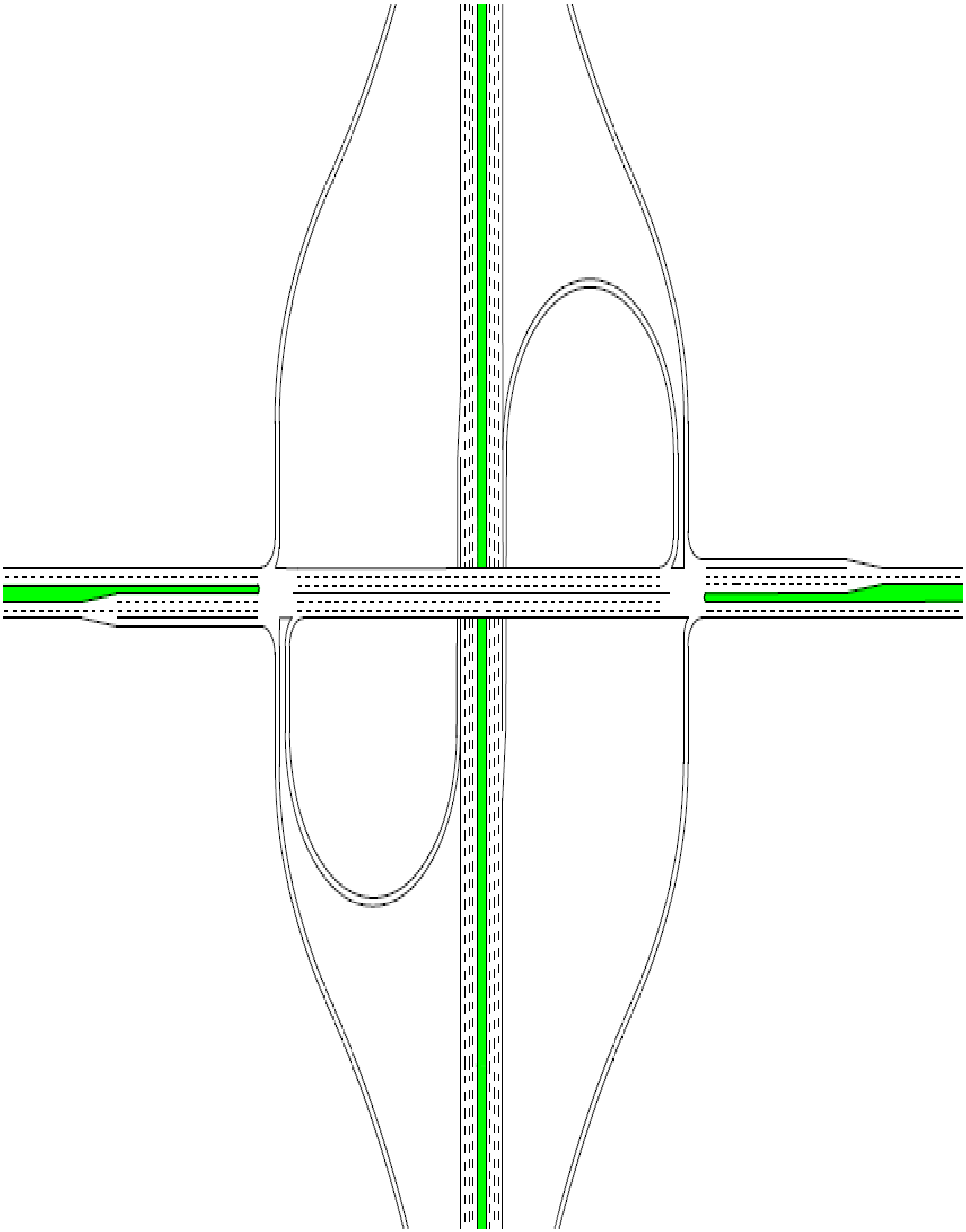
- **Milwaukee B (not to scale)**



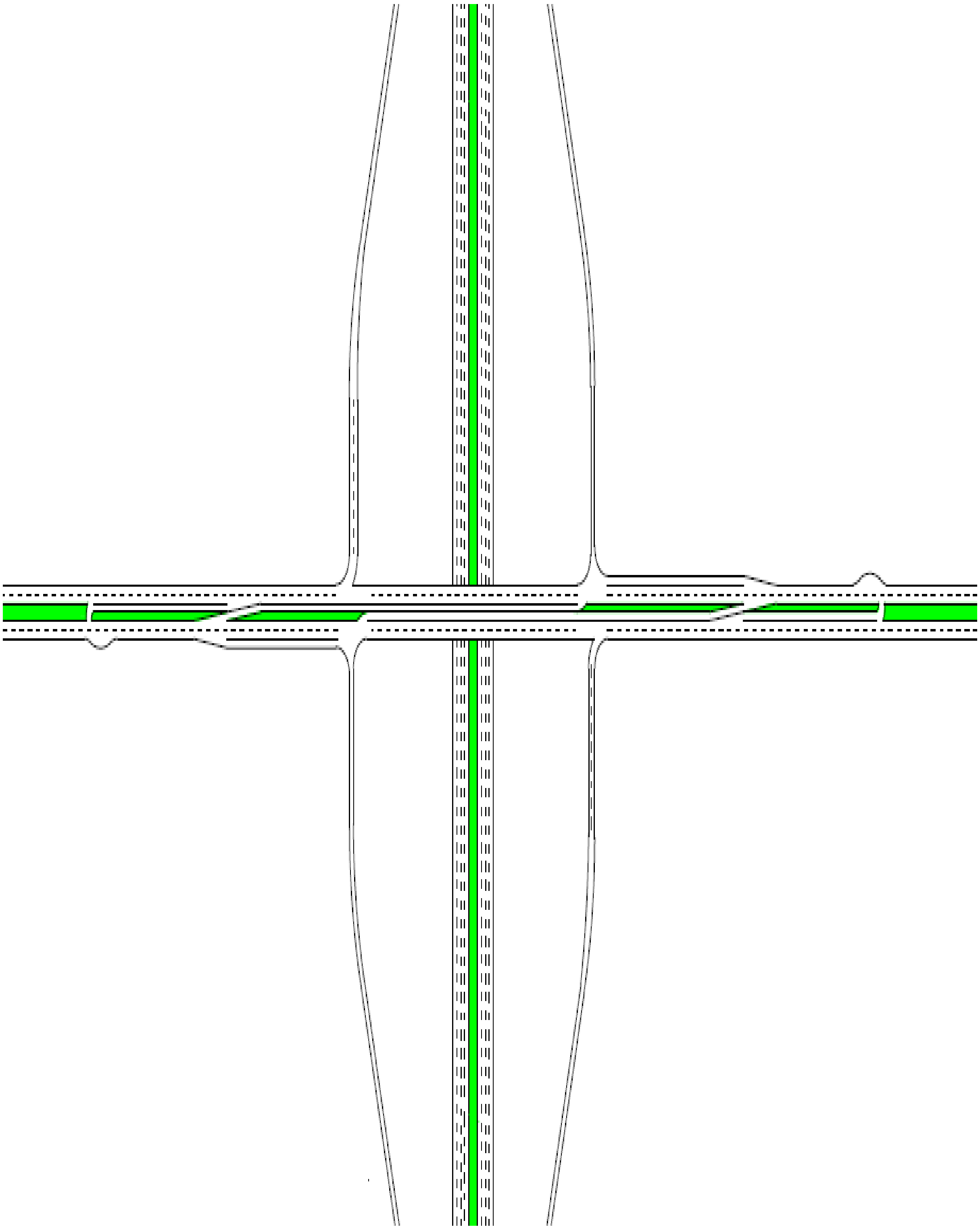
- **Milwaukee A (not to scale)**



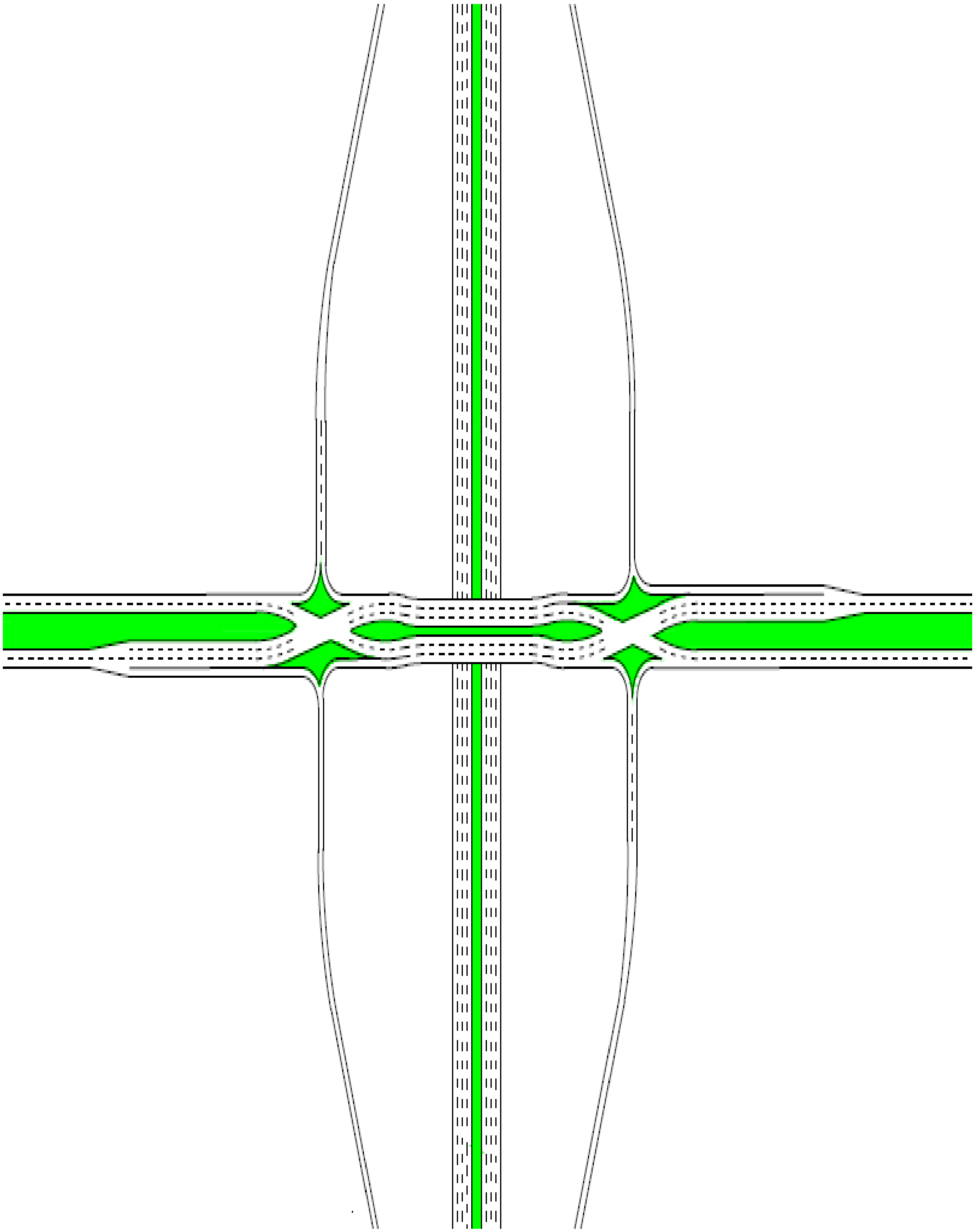
- **Parclo B (not to scale)**



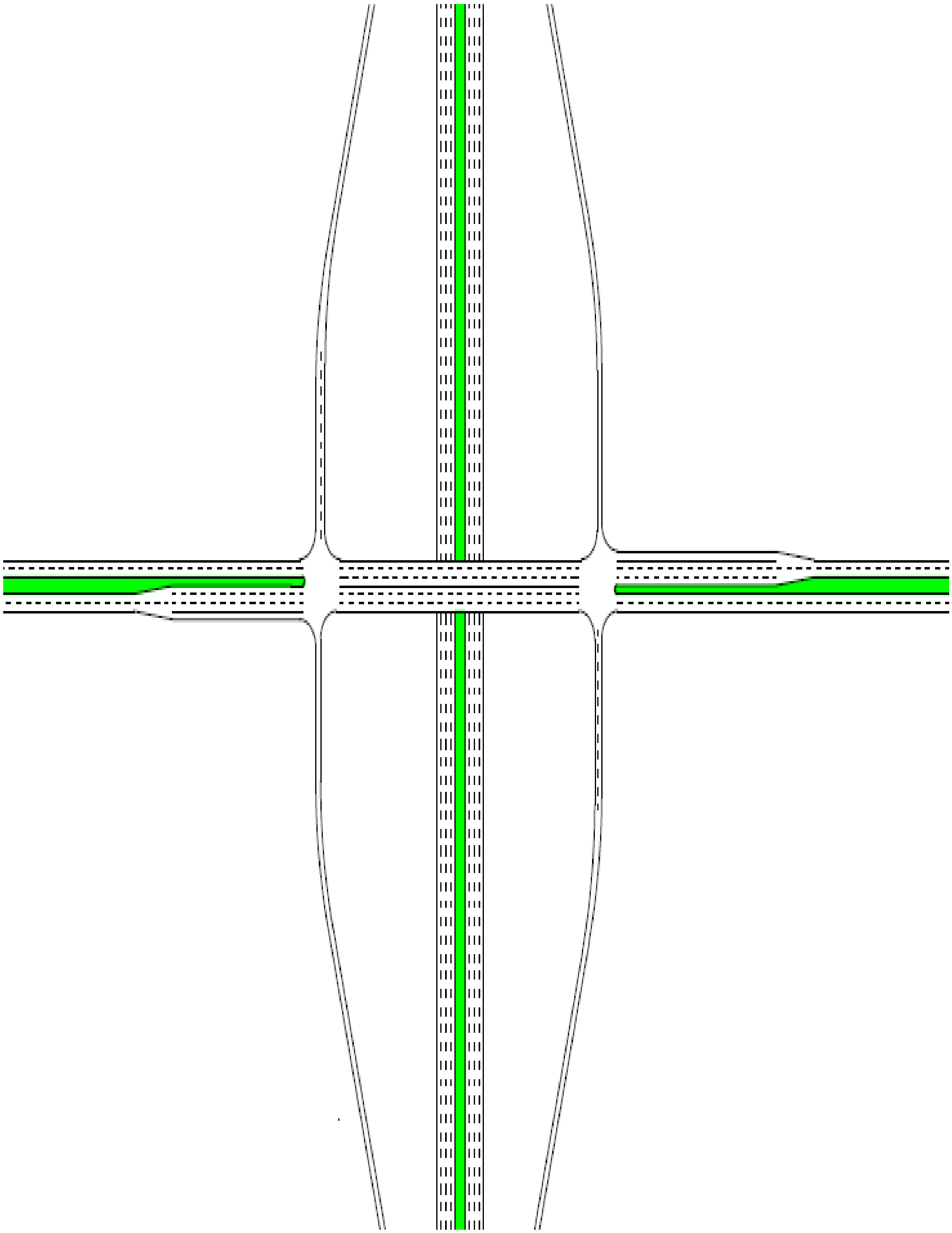
- **Synchronized (not to scale)**



- **DDI (not to scale)**



- **Diamond (not to scale)**



APPENDIX B

Detail of Scenarios

Number	V/C=1 in	Traffic of Movements	Traffic of EB/WB	Traffic of NB/SB	Truck %
1	Diamond	LT = T = RT	EB = WB	NB = SB	4
2	Diamond	LT = T = RT	EB = WB	NB = 0.75 SB	4
3	Diamond	LT = T = RT	EB = WB	NB = 0.5 SB	4
4	Diamond	LT = T = RT	EB = WB	0.75 NB = SB	4
5	Diamond	LT = T = RT	EB = WB	0.5 NB = SB	4
6	Diamond	LT = T = RT	EB = 0.75 WB	NB = SB	4
7	Diamond	LT = T = RT	EB = 0.75 WB	NB = 0.75 SB	4
8	Diamond	LT = T = RT	EB = 0.75 WB	NB = 0.5 SB	4
9	Diamond	LT = T = RT	EB = 0.75 WB	0.75 NB = SB	4
10	Diamond	LT = T = RT	EB = 0.75 WB	0.5 NB = SB	4
11	Diamond	LT = T = RT	EB = 0.5 WB	NB = SB	4
12	Diamond	LT = T = RT	EB = 0.5 WB	NB = 0.75 SB	4
13	Diamond	LT = T = RT	EB = 0.5 WB	NB = 0.5 SB	4
14	Diamond	LT = T = RT	EB = 0.5 WB	0.75 NB = SB	4
15	Diamond	LT = T = RT	EB = 0.5 WB	0.5 NB = SB	4
16	Diamond	LT = 0.66 T = RT	EB = WB	NB = SB	4
17	Diamond	LT = 0.66 T = RT	EB = WB	NB = 0.75 SB	4
18	Diamond	LT = 0.66 T = RT	EB = WB	NB = 0.5 SB	4
19	Diamond	LT = 0.66 T = RT	EB = WB	0.75 NB = SB	4
20	Diamond	LT = 0.66 T = RT	EB = WB	0.5 NB = SB	4
21	Diamond	LT = 0.66 T = RT	EB = 0.75 WB	NB = SB	4
22	Diamond	LT = 0.66 T = RT	EB = 0.75 WB	NB = 0.75 SB	4
23	Diamond	LT = 0.66 T = RT	EB = 0.75 WB	NB = 0.5 SB	4
24	Diamond	LT = 0.66 T = RT	EB = 0.75 WB	0.75 NB = SB	4
25	Diamond	LT = 0.66 T = RT	EB = 0.75 WB	0.5 NB = SB	4
26	Diamond	LT = 0.66 T = RT	EB = 0.5 WB	NB = SB	4
27	Diamond	LT = 0.66 T = RT	EB = 0.5 WB	NB = 0.75 SB	4
28	Diamond	LT = 0.66 T = RT	EB = 0.5 WB	NB = 0.5 SB	4
29	Diamond	LT = 0.66 T = RT	EB = 0.5 WB	0.75 NB = SB	4
30	Diamond	LT = 0.66 T = RT	EB = 0.5 WB	0.5 NB = SB	4
31	Diamond	LT = 0.5 T = RT	EB = WB	NB = SB	4
32	Diamond	LT = 0.5 T = RT	EB = WB	NB = 0.75 SB	4
33	Diamond	LT = 0.5 T = RT	EB = WB	NB = 0.5 SB	4
34	Diamond	LT = 0.5 T = RT	EB = WB	0.75 NB = SB	4
35	Diamond	LT = 0.5 T = RT	EB = WB	0.5 NB = SB	4
36	Diamond	LT = 0.5 T = RT	EB = 0.75 WB	NB = SB	4
37	Diamond	LT = 0.5 T = RT	EB = 0.75 WB	NB = 0.75 SB	4
38	Diamond	LT = 0.5 T = RT	EB = 0.75 WB	NB = 0.5 SB	4
39	Diamond	LT = 0.5 T = RT	EB = 0.75 WB	0.75 NB = SB	4
40	Diamond	LT = 0.5 T = RT	EB = 0.75 WB	0.5 NB = SB	4
41	Diamond	LT = 0.5 T = RT	EB = 0.5 WB	NB = SB	4
42	Diamond	LT = 0.5 T = RT	EB = 0.5 WB	NB = 0.75 SB	4
43	Diamond	LT = 0.5 T = RT	EB = 0.5 WB	NB = 0.5 SB	4
44	Diamond	LT = 0.5 T = RT	EB = 0.5 WB	0.75 NB = SB	4
45	Diamond	LT = 0.5 T = RT	EB = 0.5 WB	0.5 NB = SB	4
46	Diamond	LT = T = RT	EB = WB	NB = SB	8

Number	V/C=1 in	Traffic of Movements	Traffic of EB/WB	Traffic of NB/SB	Truck %
47	Diamond	LT = T = RT	EB = WB	NB = 0.75 SB	8
48	Diamond	LT = T = RT	EB = WB	NB = 0.5 SB	8
49	Diamond	LT = T = RT	EB = WB	0.75 NB = SB	8
50	Diamond	LT = T = RT	EB = WB	0.5 NB = SB	8
51	Diamond	LT = T = RT	EB = 0.75 WB	NB = SB	8
52	Diamond	LT = T = RT	EB = 0.75 WB	NB = 0.75 SB	8
53	Diamond	LT = T = RT	EB = 0.75 WB	NB = 0.5 SB	8
54	Diamond	LT = T = RT	EB = 0.75 WB	0.75 NB = SB	8
55	Diamond	LT = T = RT	EB = 0.75 WB	0.5 NB = SB	8
56	Diamond	LT = T = RT	EB = 0.5 WB	NB = SB	8
57	Diamond	LT = T = RT	EB = 0.5 WB	NB = 0.75 SB	8
58	Diamond	LT = T = RT	EB = 0.5 WB	NB = 0.5 SB	8
59	Diamond	LT = T = RT	EB = 0.5 WB	0.75 NB = SB	8
60	Diamond	LT = T = RT	EB = 0.5 WB	0.5 NB = SB	8
61	Diamond	LT = 0.66 T = RT	EB = WB	NB = SB	8
62	Diamond	LT = 0.66 T = RT	EB = WB	NB = 0.75 SB	8
63	Diamond	LT = 0.66 T = RT	EB = WB	NB = 0.5 SB	8
64	Diamond	LT = 0.66 T = RT	EB = WB	0.75 NB = SB	8
65	Diamond	LT = 0.66 T = RT	EB = WB	0.5 NB = SB	8
66	Diamond	LT = 0.66 T = RT	EB = 0.75 WB	NB = SB	8
67	Diamond	LT = 0.66 T = RT	EB = 0.75 WB	NB = 0.75 SB	8
68	Diamond	LT = 0.66 T = RT	EB = 0.75 WB	NB = 0.5 SB	8
69	Diamond	LT = 0.66 T = RT	EB = 0.75 WB	0.75 NB = SB	8
70	Diamond	LT = 0.66 T = RT	EB = 0.75 WB	0.5 NB = SB	8
71	Diamond	LT = 0.66 T = RT	EB = 0.5 WB	NB = SB	8
72	Diamond	LT = 0.66 T = RT	EB = 0.5 WB	NB = 0.75 SB	8
73	Diamond	LT = 0.66 T = RT	EB = 0.5 WB	NB = 0.5 SB	8
74	Diamond	LT = 0.66 T = RT	EB = 0.5 WB	0.75 NB = SB	8
75	Diamond	LT = 0.66 T = RT	EB = 0.5 WB	0.5 NB = SB	8
76	Diamond	LT = 0.5 T = RT	EB = WB	NB = SB	8
77	Diamond	LT = 0.5 T = RT	EB = WB	NB = 0.75 SB	8
78	Diamond	LT = 0.5 T = RT	EB = WB	NB = 0.5 SB	8
79	Diamond	LT = 0.5 T = RT	EB = WB	0.75 NB = SB	8
80	Diamond	LT = 0.5 T = RT	EB = WB	0.5 NB = SB	8
81	Diamond	LT = 0.5 T = RT	EB = 0.75 WB	NB = SB	8
82	Diamond	LT = 0.5 T = RT	EB = 0.75 WB	NB = 0.75 SB	8
83	Diamond	LT = 0.5 T = RT	EB = 0.75 WB	NB = 0.5 SB	8
84	Diamond	LT = 0.5 T = RT	EB = 0.75 WB	0.75 NB = SB	8
85	Diamond	LT = 0.5 T = RT	EB = 0.75 WB	0.5 NB = SB	8
86	Diamond	LT = 0.5 T = RT	EB = 0.5 WB	NB = SB	8
87	Diamond	LT = 0.5 T = RT	EB = 0.5 WB	NB = 0.75 SB	8
88	Diamond	LT = 0.5 T = RT	EB = 0.5 WB	NB = 0.5 SB	8
89	Diamond	LT = 0.5 T = RT	EB = 0.5 WB	0.75 NB = SB	8
90	Diamond	LT = 0.5 T = RT	EB = 0.5 WB	0.5 NB = SB	8
91	DDI	LT = T = RT	EB = WB	NB = SB	4
92	DDI	LT = T = RT	EB = WB	NB = 0.75 SB	4

Number	V/C=1 in	Traffic of Movements	Traffic of EB/WB	Traffic of NB/SB	Truck %
94	DDI	LT = T = RT	EB = WB	0.75 NB = SB	4
95	DDI	LT = T = RT	EB = WB	0.5 NB = SB	4
96	DDI	LT = T = RT	EB = 0.75 WB	NB = SB	4
97	DDI	LT = T = RT	EB = 0.75 WB	NB = 0.75 SB	4
98	DDI	LT = T = RT	EB = 0.75 WB	NB = 0.5 SB	4
99	DDI	LT = T = RT	EB = 0.75 WB	0.75 NB = SB	4
100	DDI	LT = T = RT	EB = 0.75 WB	0.5 NB = SB	4
101	DDI	LT = T = RT	EB = 0.5 WB	NB = SB	4
102	DDI	LT = T = RT	EB = 0.5 WB	NB = 0.75 SB	4
103	DDI	LT = T = RT	EB = 0.5 WB	NB = 0.5 SB	4
104	DDI	LT = T = RT	EB = 0.5 WB	0.75 NB = SB	4
105	DDI	LT = T = RT	EB = 0.5 WB	0.5 NB = SB	4
106	DDI	LT = 0.66 T = RT	EB = WB	NB = SB	4
107	DDI	LT = 0.66 T = RT	EB = WB	NB = 0.75 SB	4
108	DDI	LT = 0.66 T = RT	EB = WB	NB = 0.5 SB	4
109	DDI	LT = 0.66 T = RT	EB = WB	0.75 NB = SB	4
110	DDI	LT = 0.66 T = RT	EB = WB	0.5 NB = SB	4
111	DDI	LT = 0.66 T = RT	EB = 0.75 WB	NB = SB	4
112	DDI	LT = 0.66 T = RT	EB = 0.75 WB	NB = 0.75 SB	4
113	DDI	LT = 0.66 T = RT	EB = 0.75 WB	NB = 0.5 SB	4
114	DDI	LT = 0.66 T = RT	EB = 0.75 WB	0.75 NB = SB	4
115	DDI	LT = 0.66 T = RT	EB = 0.75 WB	0.5 NB = SB	4
116	DDI	LT = 0.66 T = RT	EB = 0.5 WB	NB = SB	4
117	DDI	LT = 0.66 T = RT	EB = 0.5 WB	NB = 0.75 SB	4
118	DDI	LT = 0.66 T = RT	EB = 0.5 WB	NB = 0.5 SB	4
119	DDI	LT = 0.66 T = RT	EB = 0.5 WB	0.75 NB = SB	4
120	DDI	LT = 0.66 T = RT	EB = 0.5 WB	0.5 NB = SB	4
121	DDI	LT = 0.5 T = RT	EB = WB	NB = SB	4
122	DDI	LT = 0.5 T = RT	EB = WB	NB = 0.75 SB	4
123	DDI	LT = 0.5 T = RT	EB = WB	NB = 0.5 SB	4
124	DDI	LT = 0.5 T = RT	EB = WB	0.75 NB = SB	4
125	DDI	LT = 0.5 T = RT	EB = WB	0.5 NB = SB	4
126	DDI	LT = 0.5 T = RT	EB = 0.75 WB	NB = SB	4
127	DDI	LT = 0.5 T = RT	EB = 0.75 WB	NB = 0.75 SB	4
128	DDI	LT = 0.5 T = RT	EB = 0.75 WB	NB = 0.5 SB	4
129	DDI	LT = 0.5 T = RT	EB = 0.75 WB	0.75 NB = SB	4
130	DDI	LT = 0.5 T = RT	EB = 0.75 WB	0.5 NB = SB	4
131	DDI	LT = 0.5 T = RT	EB = 0.5 WB	NB = SB	4
132	DDI	LT = 0.5 T = RT	EB = 0.5 WB	NB = 0.75 SB	4
133	DDI	LT = 0.5 T = RT	EB = 0.5 WB	NB = 0.5 SB	4
134	DDI	LT = 0.5 T = RT	EB = 0.5 WB	0.75 NB = SB	4
135	DDI	LT = 0.5 T = RT	EB = 0.5 WB	0.5 NB = SB	4
136	DDI	LT = T = RT	EB = WB	NB = SB	8
137	DDI	LT = T = RT	EB = WB	NB = 0.75 SB	8
138	DDI	LT = T = RT	EB = WB	NB = 0.5 SB	8
139	DDI	LT = T = RT	EB = WB	0.75 NB = SB	8
140	DDI	LT = T = RT	EB = WB	0.5 NB = SB	8

Number	V/C=1 in	Traffic of Movements	Traffic of EB/WB	Traffic of NB/SB	Truck %
141	DDI	LT = T = RT	EB = 0.75 WB	NB = SB	8
142	DDI	LT = T = RT	EB = 0.75 WB	NB = 0.75 SB	8
143	DDI	LT = T = RT	EB = 0.75 WB	NB = 0.5 SB	8
144	DDI	LT = T = RT	EB = 0.75 WB	0.75 NB = SB	8
145	DDI	LT = T = RT	EB = 0.75 WB	0.5 NB = SB	8
146	DDI	LT = T = RT	EB = 0.5 WB	NB = SB	8
147	DDI	LT = T = RT	EB = 0.5 WB	NB = 0.75 SB	8
148	DDI	LT = T = RT	EB = 0.5 WB	NB = 0.5 SB	8
149	DDI	LT = T = RT	EB = 0.5 WB	0.75 NB = SB	8
150	DDI	LT = T = RT	EB = 0.5 WB	0.5 NB = SB	8
151	DDI	LT = 0.66 T = RT	EB = WB	NB = SB	8
152	DDI	LT = 0.66 T = RT	EB = WB	NB = 0.75 SB	8
153	DDI	LT = 0.66 T = RT	EB = WB	NB = 0.5 SB	8
154	DDI	LT = 0.66 T = RT	EB = WB	0.75 NB = SB	8
155	DDI	LT = 0.66 T = RT	EB = WB	0.5 NB = SB	8
156	DDI	LT = 0.66 T = RT	EB = 0.75 WB	NB = SB	8
157	DDI	LT = 0.66 T = RT	EB = 0.75 WB	NB = 0.75 SB	8
158	DDI	LT = 0.66 T = RT	EB = 0.75 WB	NB = 0.5 SB	8
159	DDI	LT = 0.66 T = RT	EB = 0.75 WB	0.75 NB = SB	8
160	DDI	LT = 0.66 T = RT	EB = 0.75 WB	0.5 NB = SB	8
161	DDI	LT = 0.66 T = RT	EB = 0.5 WB	NB = SB	8
162	DDI	LT = 0.66 T = RT	EB = 0.5 WB	NB = 0.75 SB	8
163	DDI	LT = 0.66 T = RT	EB = 0.5 WB	NB = 0.5 SB	8
164	DDI	LT = 0.66 T = RT	EB = 0.5 WB	0.75 NB = SB	8
165	DDI	LT = 0.66 T = RT	EB = 0.5 WB	0.5 NB = SB	8
166	DDI	LT = 0.5 T = RT	EB = WB	NB = SB	8
167	DDI	LT = 0.5 T = RT	EB = WB	NB = 0.75 SB	8
168	DDI	LT = 0.5 T = RT	EB = WB	NB = 0.5 SB	8
169	DDI	LT = 0.5 T = RT	EB = WB	0.75 NB = SB	8
170	DDI	LT = 0.5 T = RT	EB = WB	0.5 NB = SB	8
171	DDI	LT = 0.5 T = RT	EB = 0.75 WB	NB = SB	8
172	DDI	LT = 0.5 T = RT	EB = 0.75 WB	NB = 0.75 SB	8
173	DDI	LT = 0.5 T = RT	EB = 0.75 WB	NB = 0.5 SB	8
174	DDI	LT = 0.5 T = RT	EB = 0.75 WB	0.75 NB = SB	8
175	DDI	LT = 0.5 T = RT	EB = 0.75 WB	0.5 NB = SB	8
176	DDI	LT = 0.5 T = RT	EB = 0.5 WB	NB = SB	8
177	DDI	LT = 0.5 T = RT	EB = 0.5 WB	NB = 0.75 SB	8
178	DDI	LT = 0.5 T = RT	EB = 0.5 WB	NB = 0.5 SB	8
179	DDI	LT = 0.5 T = RT	EB = 0.5 WB	0.75 NB = SB	8
180	DDI	LT = 0.5 T = RT	EB = 0.5 WB	0.5 NB = SB	8

- **Traffic Volume of scenarios (veh/lane/direction)**

Number	LT of EB	T of EB	RT of EB	LT of WB	T of WB	RT of WB	LT of NB	RT of NB	LT of SB	RT of SB
1	428	428	428	428	428	428	428	428	428	428
2	428	428	428	428	428	428	366	366	490	490
3	428	428	428	428	428	428	285	285	571	571
4	428	428	428	428	428	428	490	490	366	366
5	428	428	428	428	428	428	571	571	285	285
6	366	366	366	490	490	490	428	428	428	428
7	366	366	366	490	490	490	366	366	490	490
8	366	366	366	490	490	490	285	285	571	571
9	366	366	366	490	490	490	490	490	366	366
10	366	366	366	490	490	490	571	571	285	285
11	285	285	285	571	571	571	428	428	428	428
12	285	285	285	571	571	571	366	366	490	490
13	285	285	285	571	571	571	285	285	571	571
14	285	285	285	571	571	571	490	490	366	366
15	285	285	285	571	571	571	571	571	285	285
16	375	563	375	375	563	375	375	375	375	375
17	375	563	375	375	563	375	320	320	427	427
18	375	563	375	375	563	375	249	249	497	497
19	375	563	375	375	563	375	427	427	320	320
20	375	563	375	375	563	375	497	497	249	249
21	321	482	321	429	642	429	373	373	373	373
22	321	482	321	429	642	429	320	320	427	427
23	321	482	321	429	642	429	249	249	497	497
24	321	482	321	429	642	429	427	427	320	320
25	321	482	321	429	642	429	497	497	249	249
26	250	375	250	500	750	500	373	373	373	373
27	250	375	250	500	750	500	320	320	427	427
28	250	375	250	500	750	500	249	249	497	497
29	250	375	250	500	750	500	427	427	320	320
30	250	375	250	500	750	500	497	497	249	249
31	333	667	333	333	667	333	333	333	333	333
32	333	667	333	333	667	333	285	285	381	381
33	333	667	333	333	667	333	222	222	444	444
34	333	667	333	333	667	333	381	381	285	285
35	333	667	333	333	667	333	444	444	222	222
36	285	571	285	381	761	381	333	333	333	333
37	285	571	285	381	761	381	285	285	381	381
38	285	571	285	381	761	381	222	222	444	444
39	285	571	285	381	761	381	381	381	285	285
40	285	571	285	381	761	381	444	444	222	222
41	222	444	222	444	888	444	333	333	333	333
42	222	444	222	444	888	444	285	285	381	381
43	222	444	222	444	888	444	222	222	444	444
44	222	444	222	444	888	444	381	381	285	285
45	222	444	222	444	888	444	444	444	222	222
46	428	428	428	428	428	428	428	428	428	428

Number	LT of EB	T of EB	RT of EB	LT of WB	T of WB	RT of WB	LT of NB	RT of NB	LT of SB	RT of SB
47	428	428	428	428	428	428	366	366	490	490
48	428	428	428	428	428	428	285	285	571	571
49	428	428	428	428	428	428	490	490	366	366
50	428	428	428	428	428	428	571	571	285	285
51	366	366	366	490	490	490	428	428	428	428
52	366	366	366	490	490	490	366	366	490	490
53	366	366	366	490	490	490	285	285	571	571
54	366	366	366	490	490	490	490	490	366	366
55	366	366	366	490	490	490	571	571	285	285
56	285	285	285	571	571	571	428	428	428	428
57	285	285	285	571	571	571	366	366	490	490
58	285	285	285	571	571	571	285	285	571	571
59	285	285	285	571	571	571	490	490	366	366
60	285	285	285	571	571	571	571	571	285	285
61	375	563	375	375	563	375	375	375	375	375
62	375	563	375	375	563	375	320	320	427	427
63	375	563	375	375	563	375	249	249	497	497
64	375	563	375	375	563	375	427	427	320	320
65	375	563	375	375	563	375	497	497	249	249
66	321	482	321	429	642	429	373	373	373	373
67	321	482	321	429	642	429	320	320	427	427
68	321	482	321	429	642	429	249	249	497	497
69	321	482	321	429	642	429	427	427	320	320
70	321	482	321	429	642	429	497	497	249	249
71	250	375	250	500	750	500	373	373	373	373
72	250	375	250	500	750	500	320	320	427	427
73	250	375	250	500	750	500	249	249	497	497
74	250	375	250	500	750	500	427	427	320	320
75	250	375	250	500	750	500	497	497	249	249
76	333	667	333	333	667	333	333	333	333	333
77	333	667	333	333	667	333	285	285	381	381
78	333	667	333	333	667	333	222	222	444	444
79	333	667	333	333	667	333	381	381	285	285
80	333	667	333	333	667	333	444	444	222	222
81	285	571	285	381	761	381	333	333	333	333
82	285	571	285	381	761	381	285	285	381	381
83	285	571	285	381	761	381	222	222	444	444
84	285	571	285	381	761	381	381	381	285	285
85	285	571	285	381	761	381	444	444	222	222
86	222	444	222	444	888	444	333	333	333	333
87	222	444	222	444	888	444	285	285	381	381
88	222	444	222	444	888	444	222	222	444	444
89	222	444	222	444	888	444	381	381	285	285
90	222	444	222	444	888	444	444	444	222	222
91	533	533	533	533	533	533	533	533	533	533
92	533	533	533	533	533	533	457	457	609	609
93	533	533	533	533	533	533	355	355	710	710

Number	LT of EB	T of EB	RT of EB	LT of WB	T of WB	RT of WB	LT of NB	RT of NB	LT of SB	RT of SB
94	533	533	533	533	533	533	609	609	457	457
95	533	533	533	533	533	533	710	710	355	355
96	457	457	457	609	609	609	533	533	533	533
97	457	457	457	609	609	609	457	457	609	609
98	457	457	457	609	609	609	355	355	710	710
99	457	457	457	609	609	609	609	609	457	457
100	457	457	457	609	609	609	710	710	355	355
101	355	355	355	710	710	710	533	533	533	533
102	355	355	355	710	710	710	457	457	609	609
103	355	355	355	710	710	710	355	355	710	710
104	355	355	355	710	710	710	609	609	457	457
105	355	355	355	710	710	710	710	710	355	355
106	400	600	400	400	600	400	400	400	400	400
107	400	600	400	400	600	400	343	343	457	457
108	400	600	400	400	600	400	267	267	533	533
109	400	600	400	400	600	400	457	457	343	343
110	400	600	400	400	600	400	533	533	267	267
111	343	514	343	457	686	457	400	400	400	400
112	343	514	343	457	686	457	343	343	457	457
113	343	514	343	457	686	457	267	267	533	533
114	343	514	343	457	686	457	457	457	343	343
115	343	514	343	457	686	457	533	533	267	267
116	267	400	267	533	800	533	400	400	400	400
117	267	400	267	533	800	533	343	343	457	457
118	267	400	267	533	800	533	267	267	533	533
119	267	400	267	533	800	533	457	457	343	343
120	267	400	267	533	800	533	533	533	267	267
121	320	640	320	320	640	320	320	320	320	320
122	320	640	320	320	640	320	275	275	365	365
123	320	640	320	320	640	320	213	213	427	427
124	320	640	320	320	640	320	365	365	275	275
125	320	640	320	320	640	320	427	427	213	213
126	275	550	275	365	730	365	320	320	320	320
127	275	550	275	365	730	365	275	275	365	365
128	275	550	275	365	730	365	213	213	427	427
129	275	550	275	365	730	365	365	365	275	275
130	275	550	275	365	730	365	427	427	213	213
131	213	427	213	427	853	427	320	320	320	320
132	213	427	213	427	853	427	275	275	365	365
133	213	427	213	427	853	427	213	213	427	427
134	213	427	213	427	853	427	365	365	275	275
135	213	427	213	427	853	427	427	427	213	213
136	533	533	533	533	533	533	533	533	533	533
137	533	533	533	533	533	533	457	457	609	609
138	533	533	533	533	533	533	355	355	710	710
139	533	533	533	533	533	533	609	609	457	457
140	533	533	533	533	533	533	710	710	355	355

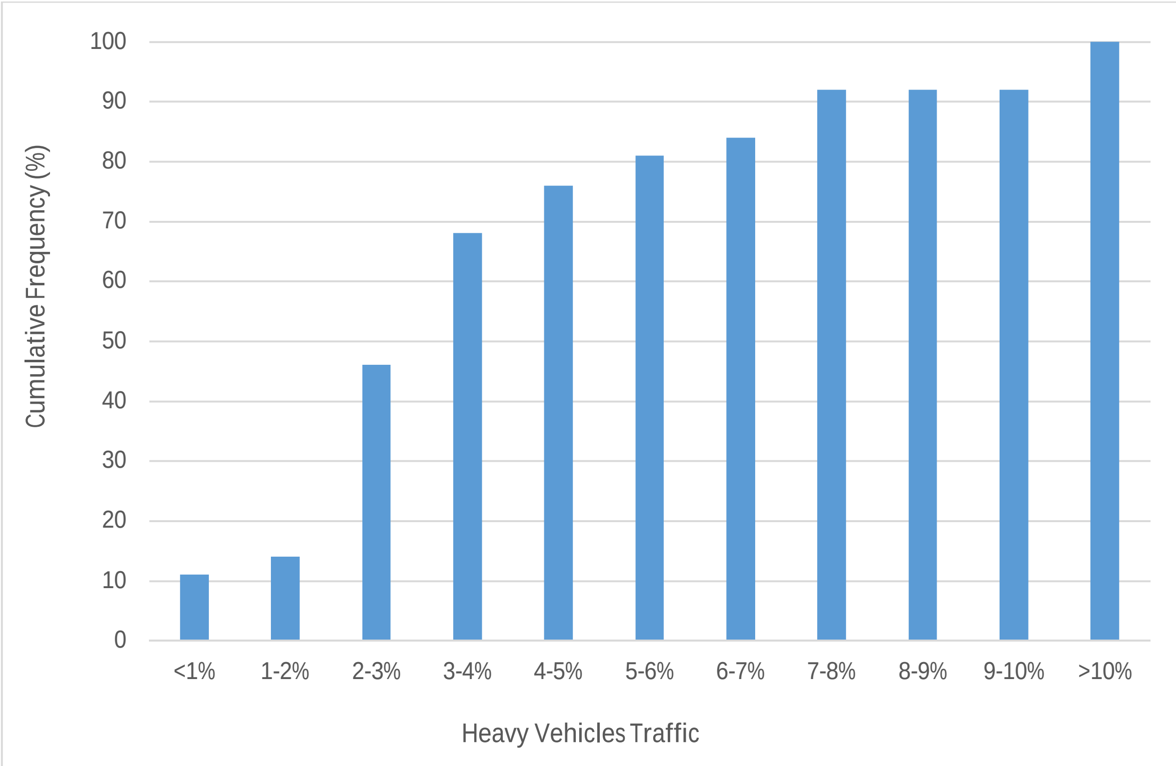
Number	LT of EB	T of EB	RT of EB	LT of WB	T of WB	RT of WB	LT of NB	RT of NB	LT of SB	RT of SB
141	457	457	457	609	609	609	533	533	533	533
142	457	457	457	609	609	609	457	457	609	609
143	457	457	457	609	609	609	355	355	710	710
144	457	457	457	609	609	609	609	609	457	457
145	457	457	457	609	609	609	710	710	355	355
146	355	355	355	710	710	710	533	533	533	533
147	355	355	355	710	710	710	457	457	609	609
148	355	355	355	710	710	710	355	355	710	710
149	355	355	355	710	710	710	609	609	457	457
150	355	355	355	710	710	710	710	710	355	355
151	400	600	400	400	600	400	400	400	400	400
152	400	600	400	400	600	400	343	343	457	457
153	400	600	400	400	600	400	267	267	533	533
154	400	600	400	400	600	400	457	457	343	343
155	400	600	400	400	600	400	533	533	267	267
156	343	514	343	457	686	457	400	400	400	400
157	343	514	343	457	686	457	343	343	457	457
158	343	514	343	457	686	457	267	267	533	533
159	343	514	343	457	686	457	457	457	343	343
160	343	514	343	457	686	457	533	533	267	267
161	267	400	267	533	800	533	400	400	400	400
162	267	400	267	533	800	533	343	343	457	457
163	267	400	267	533	800	533	267	267	533	533
164	267	400	267	533	800	533	457	457	343	343
165	267	400	267	533	800	533	533	533	267	267
166	320	640	320	320	640	320	320	320	320	320
167	320	640	320	320	640	320	275	275	365	365
168	320	640	320	320	640	320	213	213	427	427
169	320	640	320	320	640	320	365	365	275	275
170	320	640	320	320	640	320	427	427	213	213
171	275	550	275	365	730	365	320	320	320	320
172	275	550	275	365	730	365	275	275	365	365
173	275	550	275	365	730	365	213	213	427	427
174	275	550	275	365	730	365	365	365	275	275
175	275	550	275	365	730	365	427	427	213	213
176	213	427	213	427	853	427	320	320	320	320
177	213	427	213	427	853	427	275	275	365	365
178	213	427	213	427	853	427	213	213	427	427
179	213	427	213	427	853	427	365	365	275	275
180	213	427	213	427	853	427	427	427	213	213

APPENDIX C

Data Collected

- Truck Volume

Number	Address	Type of Area	Truck % of Total	Year
1	I-85 NB Ramps & SR 1675 Glenn SCH Rd	Suburban	2.6	2012
2	I-85 SB Ramps & SR 1675 Glenn SCH Rd	Suburban	3.1	2012
3	I-85 SB Ramps & NC 56 (During a.m peak hour)	Suburban	4.7	2012
4	I-85 SB Ramps & NC 56 (During p.m peak hour)	Suburban	2.4	2012
5	I-40 EB Ramps & NC-42	Suburban	2.8	2012
6	I-40 WB Ramps & NC-42	Suburban	3.2	2012
7	I-40 WB Ramps & NC 210	Suburban	3.7	2012
8	I-40 EB Ramps & NC 210	Suburban	4.4	2012
9	I-95 SB Ramps & SR 2339 Bagley Rd	Suburban	11.7	2011
10	I-95 NB Ramps & SR 2339 Bagley Rd	Suburban	23.2	2011
11	I-40 WB Ramp & Hanes Mall Blvd	Suburban	0.6	2011
12	I-40 EB Ramp & SR 4315	Suburban	0.8	2012
13	I-485 & NC 51 (Charlotte Outer Loop)	Suburban	2.6	2013
14	I-485 & SR 1009	Suburban	0.7	2013
15	I-485 SB Ramps & SR 3174	Suburban	2.3	2012
16	I-485 (Outer Loop) & SR 3624	Suburban	2.4	2012
17	US 29 & US 74	Suburban	0.8	2011
18	I-77 SB Ramp & US 74	Suburban	2.5	2011
19	I-277 (Inn Loop) & US 74	Suburban	3.3	2011
20	I-77 EB Ramp & SR2108	Suburban	10.3	2013
21	I-85 & NC 25	Suburban	5.6	2013
22	I-40 EB Ramp & SR 2158	Suburban	2.6	2012
23	I-40 WB Ramp & SR 2158	Suburban	2.7	2012
24	I-240 WB Ramps & NC 191	Suburban	2.5	2011
25	I-240 EB Ramps & NC 191	Suburban	4.2	2011
26	I-40 EB Ramp & US 19	Suburban	3.3	2011
27	I-40 WB Ramp & US 19	Suburban	3.3	2011
28	I-40 EB Ramp & US 25	Suburban	1.7	2011
29	I-40 WB Ramp & US 25	Suburban	2.0	2011
30	I-40 WB Ramp & US 221	Suburban	7.5	2012
31	I-40 EB Ramp & US 221	Suburban	7.9	2012
32	I-26 WB Ramp & SR 1783	Suburban	3.2	2011
33	I-26 EB Ramp & SR 1783	Suburban	3.6	2011
34	I-26 WB Ramp & US 25	Suburban	6.4	2011
35	I-26 EB Ramp & US 25	Suburban	5.6	2011
36	I-26 EB Ramp & SR 1142	Suburban	2.7	2011
37	I-26 WB Ramp & SR 1142	Suburban	7.0	2011
Ave	**Average of Truck %**		**4.3**	
Median	**Median of Truck %**		**3.2**	
Min	**Minumum of Truck %**		**0.6**	
Max	**Maximum of Truck %**		**23.2**	
SD	**Standard Devision of Truck %**		**4.1**	
N	**Number of Samples**		**37.0**	
K	**Level of Confidence of 95%**		**1.96**	
E	**Predicted Error of Samples**		**1.32**	



Cumulative Frequency (%)
100
90
80
70
60
50
40
30
20
10
0
<1%
1-2%
2-3%
3-4%
4-5%
5-6%
6-7%
7-8%
8-9%
9-10%
>10%
Heavy Vehicles Traffic

- **Interchange Geometric Data**

Number	Address	Type	Radius-Loop	Distance btw Nodes
1	Haggerty Connector and 12 Mile Rd, MI	Parclo	260	1400
2	I-275 and Ann Arbor Rd, MI	Parclo	260	1300
3	I-275 and Ford Rd, MI	Parclo	270	1400
4	I-275 and Eureka Rd, MI	Parclo	260	1600
5	Haggerty Rd & Detroit Industrial Expy, MI	Parclo	200	1000
6	I-275 and 6 Mile Rd, MI	Parclo	280	1500
7	Belleville Rd & Detroit Industrial Expy, MI	Parclo	230	1250
8	I-96 and Novi Rd, MI	Parclo	225	1250
9	I-96 and Novi Rd, MI	Parclo	280	1250
10	I-96 and Fowlerville Rd, MI	Parclo	230	1200
11	I-75 and 14 Mile Rd, MI	Parclo	240	1200
12	I-96 and Kensington Rd, MI	Parclo	240	1350
13	I-96 and Milford Rd, MI	Parclo	280	1300
14	I-275 and 7 Mile Rd, MI	Parclo	260	1500
15	I-96 and Latson Rd, MI	Diamond	No Loop	1500
16	I-275 and Ecorse Rd, MI	Diamond	No Loop	1600
17	US-23 and US-12, MI	Diamond	No Loop	1400
18	I-275 and Sibley Rd, MI	Diamond	No Loop	1650
19	I-94 and Van Dyke Rd, MI	Diamond	No Loop	300
20	I-94 and Candieux Rd, MI	Diamond	No Loop	300
21	M-10 Fwy and Forest Ave, MI	Diamond	No Loop	250
22	M-10 Fwy and Linwood Rd, MI	Diamond	No Loop	250
23	I-75 and University Dr, MI	DDI	No Loop	1300
24	I-44 and Kansas Expressway, MO	DDI	No Loop	600
25	US-60 and National Ave, MO	DDI	No Loop	700
26	I-15 and American Fork, UT	DDI	No Loop	800
27	I-15 and Timpanogos Hwy, UT	DDI	No Loop	700
28	I-590 and South Winton Road, NY	DDI	No Loop	600
29	US-129 and Middlesettlements Rd, TN	DDI	No Loop	700
30	I-846 and 27th St, WI	Milwaukee A	230	600

Number	Radius-RT of OnRamp	Radius-RT of OffRamp	Ave Length-OnRamps	Ave Length-OffRamps
1	60	30	2300	2400
2	60	60	1800	1800
3	60	60	1900	2000
4	50	55	2400	2400
5	45	30	1800	1800
6	50	50	1800	2600
7	50	30	2000	2000
8	30	50	2400	2600
9	30	50	2300	2600
10	50	No Ramp	1800	No Ramp
11	50	50	1750	1300
12	30	No Ramp	1300	No Ramp
13	50	50	1800	2000
14	45	25	1700	2200
15	40	40	2200	2200
16	50	50	1900	2200
17	50	50	1500	2000
18	45	45	2100	2100
19	35	30	1000	1200
20	30	25	900	1200
21	20	20	900	900
22	20	20	1000	1000
23	Not Signalized	Not Signalized	1300	2800
24	Not Signalized	Not Signalized	1000	1000
25	Not Signalized	Not Signalized	1300	1400
26	Not Signalized	Not Signalized	1200	1800
27	Not Signalized	Not Signalized	2600	1800
28	Not Signalized	Not Signalized	1700	1400
29	Not Signalized	Not Signalized	800	1200
30	Not Signalized	Not Signalized	1800	1800

- **Superstreet intersection Geometric Data**

Number	Address	Radius of U-Turns	Width of Median
1	US-281 and Stone Oak Pkwy	40	20
2	US-1 and Camp Easter Rd	40	10
3	US-15 and Old Frederick Rd	30	35
4	US-17 and Goodman Rd	35	30
5	NC-55 and New Hill Rd	25	8
6	W Big Beaver Rd and Lakeview Dr	40	35
7	US-15 and Erwin Rd	20	8
8	US-15 and Erwin Rd	35	130
9	US-17 and Sloop Point Rd	25	15
10	US-301 and MD-313	25	30
11	Plum Road and US 231	20	25
12	Retail Drive and US 231	15	20
13	Ohio 4 & Hamilton Mason Road	25	20
14	Long Lake & Corporate Drive	30	45

Number	Radius of Loon	Ave Distance from Center to U-Turns	Type	State
1	45	1200	Signalized	TX
2	45	1200	Stop-controlled	NC
3	-	2400	Merge-controlled	MD
4	45	900	Signalized	NC
5	50	1300	Signalized	NC
6	-	500	Signalized	MI
7	40	700	Signalized	NC
8	-	700	Signalized	NC
9	45	1100	Stop-controlled	NC
10	-	1400	Merge-controlled	MD
11	-	800	Signalized	AL
12	-	600	Signalized	AL
13	60	1000	Signalized	OH
14	-	600	Signalized	MI

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ABSTRACT

EVALUATION OF MILWAUKEE B AND SYNCHRONIZED AS NEW SERVICE INTERCHANGE DESIGNS

by

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These days, alternative interchanges are attracting the attention of transportation agencies and designers more than ever. Most of the existing interchanges in the U.S were built in the 1950s and 1960s when traffic volume was much lower, and the type of vehicles and driving habits were completely different. Moreover, the knowledge of highway design and safety is more developed now, and this provides an appropriate situation to increase the efficiency of interchanges regarding traffic operation and safety using alternative interchanges.

This research evaluated the performance of two proposed service interchange designs—the synchronized design which is related to a superstreet intersection and the Milwaukee B design that is related to a parclo B design--as possible substitutes where existing interchanges are failing. Over 1700 simulation tests modeled the traffic operation, pedestrian performance, and safety of six different interchanges (two new and four existing interchanges) in different conditions of traffic volume, traffic distribution, left/right turning volume ratios, and heavy vehicle percentage. Then, a cost estimation and validation procedure were also conducted to complete the analysis.

Overall, the Milwaukee B showed the best traffic operation among all the interchanges. The synchronized interchange looks promising as a substitute for a diamond interchange with dominant through traffic. The synchronized and diverging diamond interchanges (DDI) showed almost the same results while handling moderate levels of turning volume; however, the synchronized performed better than the DDI in low turning volumes while the DDI can be a better choice in high turning ratios. Regarding the safety, the DDI and Milwaukee B were the safest designs based on observed conflicting interactions in the simulation models; however, the DDI did not seem as reliable from the viewpoint of unusual maneuvers and wrong way movements. The new synchronized interchange, the parclo B, and the Milwaukee A (an existing interchange in Milwaukee, WI) showed the same rate of conflicts between vehicles. The synchronized interchange may be advantageous because it was estimated to reduce the severity of crashes due to fewer crossing conflicts, a lower speed of conflicts, and a higher time to collision. The results of the pedestrian analysis indicated that a relatively safe condition is expected for pedestrians in the proposed new designs in comparison to the existing interchanges. The DDI, one of the most popular alternative interchanges, showed the worst performance in all the aspects of the pedestrian analysis.

AUTOBIOGRAPHICAL STATEMENT

The first traces of my interest to civil engineering go back to my childhood when I was always looking for any single chance to go to construction zones with my father. As a 5-year old child, watching some activities like an explosion for the excavation were seemed very interesting. Moreover, the encourages of my father, as my childhood's hero, made me determined from that time to follow his way as a civil engineer.

I obtained a seat to start my undergraduate program at Azad University in 2007. The first year of my undergraduate program was pretty good; however, I suddenly found myself in doubt for selecting my future among the education, sport, and music. Honestly, I was also somehow disappointed for selecting the civil engineering since most the courses until that stage was involved with logic and calculations, and I was thinking that I should have chosen the architecting as my career. Searching for creativity was the reason that I got interested in transportation after taking highway design in Winter 2010. The highway design made me determined to follow my civil engineering dream again. Then, I started preparing myself for the entrance exam of Master program.

I was accepted at Azad University again to start my Master program in 2011. At that stage, I was experienced enough to focus only on my education and quit whatever else that could be disturbing on my way to become a highway designer. Until my Master program, I did not have any clear idea about conducting research and writing articles. As a funny example, I was always hearing from the instructors the word "ISI" while I did not have any idea about it. One day, I decided to ask one of my classmates about the meaning of that, and he made fun of me introducing it as "International Soccer Institute." The abbreviation means "International Scientific Indexing" ☺. After that, my master program was involved with research and I could publish more than 10 technical papers (three of them in ISI journals). The interest that I got in researching on highway design leaded me to apply for a Ph.D. after my graduation of master studies in 2013.

I was lucky that I got the admission of Wayne State University which had high-level faculty members like Prof. Hummer, who is a famous researcher in the field of transportation. The first year of my program was spent mostly passing classes and doing data collection as a part-time job; however, the second year of my program was surely an unforgettable year for me either from the viewpoint of education or my personal life because of finding good and supportive friends. I could make a dream of mine by teaching two classes in this year. In addition to the great experiences that I got during the teaching, it provided some of the happiest days of my life in the U.S. (or maybe my entire life). Fortunately, my performance was evaluated very good by the students as well. The last year of my education at WSU was mostly focused on my dissertation. I studied the performance of two new interchange designs under the supervision of Prof. Hummer, and we could get interesting results which can be useful for highway designers. I would like to name the last 3-year of my life at WSU as the period of "challenge for making bittersweet decisions."