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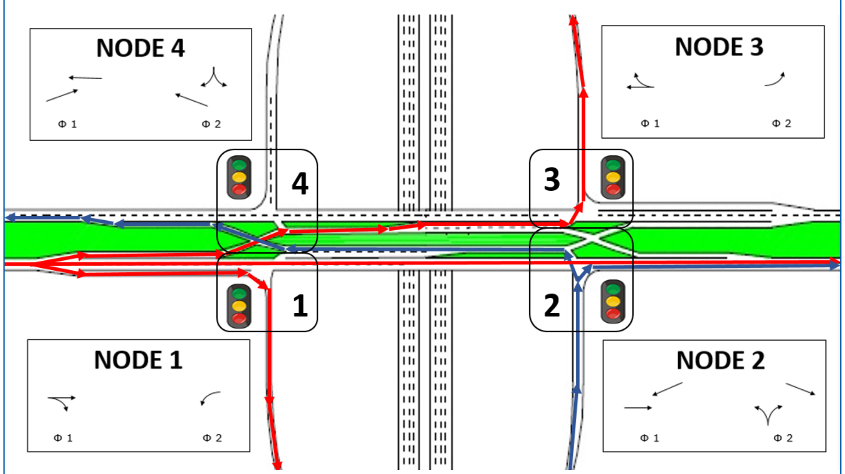
PROPOSING THE SUPER DDI DESIGN TO IMPROVE THE PERFORMANCE OF FAILING SERVICE INTERCHANGES IN DENVER METRO, COLORADO



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16. Abstract

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PROPOSING THE SUPER DDI DESIGN TO IMPROVE THE PERFORMANCE OF FAILING SERVICE INTERCHANGES IN DENVER METRO, COLORADO

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Disclaimer

ABSTRACT

This study evaluates two versions of the new super diverging diamond interchange (Super DDI) design as a possible alternative to improve the performance of failing service interchanges in the Denver, Colorado, metro. Three interchanges, Interstate 225 and Mississippi Ave, Interstate 25 and 120th Ave, and Interstate 25 and Hampden Ave, were identified in Denver as the potential candidates to model for future retrofit. Several microsimulation models were created to test four interchange designs. i.e., existing conventional diamond interchange (CDI), diverging diamond interchange (DDI), Super DDI-1, and Super DDI-2. The traffic operation, safety, and pedestrian performance were evaluated using the combination of VISSIM, Synchro, and Surrogate Safety Assessment Model (SSAM) analyzing tools. As an important finding from this research, Super DDI designs outperformed DDI in terms of vehicular operation and safety when considering adjacent signals and when higher traffic demand exists, while DDI performed similarly or sometimes insignificantly better compared with Super DDI if no adjacent intersections were located in the vicinity and if the demand was lower than DDI's capacity. On the other hand, the analysis of pedestrian performance showed that a relatively safe condition is expected for pedestrians in the proposed new Super DDI designs compared with CDI and DDI.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

EXECUTIVE SUMMARY

In response to efforts made to upgrade the performance of old and failing service interchanges, communities across the nation have seen a paradigm shift toward alternative interchange designs. Nowadays, innovative interchanges are drawing attention to transportation agencies and designers for their potential in accommodating higher traffic demand, reducing delays, and mitigating congestion. Most of the existing conventional diamond interchanges (CDI) in the United States were designed and constructed about 60 to 70 years ago, and at the time such high traffic volumes were not anticipated. Throughout the decades, vehicle types and driving habits have significantly changed. According to the Federal Highway Administration (FHWA), there was an average increase of about 3.6 million vehicles each year since 1960 in the United States. These increased traffic demands have raised many operational and safety concerns while traveling through conventional interchanges, leading to deteriorated traffic conditions especially during peak hours in big urban areas. The FHWA, the National Highway Traffic Safety Administration (NHTSA) and the Federal Motor Carrier Safety Administration (FMSCA) are working with the National Safety Council (NSC) to develop a coordinated approach (i.e., zero vision mission) to reaching zero deaths within the next 30 years. Considering the growing traffic demands, safety, and budget constraints, the investigation of alternative interchange designs has become a vital task for highway engineers.

Based on the studies conducted by Molan et al. in 2019, the super diverging diamond interchange (Super DDI) was proposed as an alternative design, which has the potential to mitigate the problems of the DDI. This study aims to advance the current research by introducing two versions of Super DDI design and evaluate the vehicular operation, safety, and pedestrian performance using real-world locations. As part of a comprehensive research effort to improve the performance of failing service interchanges in the mountain-plains region, the study identified three interchanges (Interstate 225 and Mississippi Ave, Interstate 25 and 120th Ave, and Interstate 25 and Hampden Ave) in Denver, Colorado, as the potential candidates to model for future retrofit. Four interchange designs (existing CDI, DDI, Super DDI-1, and Super DDI-2) were tested in this study, in which three alternative designs were considered to make a reasonable comparison with the existing CDI. The analysis was conducted using the combination of VISSIM (2020 version), Synchro (version 11), and SSAM analyzing tools. The study first applied Synchro to attain optimum signal timing and cycle length. Signal data were then imported into VISSIM to estimate the performance of each interchange design. After that, VISSIM trajectory files generated from each simulation were analyzed through the SSAM to determine the types and number of conflicts for each design.

In order to examine traffic operation and safety, a comprehensive series of microsimulation models (192 scenarios with 960 runs) were created with three peak hours (AM, Noon, and PM) for existing (the year 2020) and projected (the year 2030) traffic volumes. The study considered two simulation networks: (i) when no adjacent traffic signal exists, to determine how the four interchange designs would perform if there are no adjacent signals or they are far away from the interchange; and (ii) when there are two adjacent traffic signals, to evaluate the performance of the four interchanges in a bigger corridor with signal coordination needed. All the simulation scenarios were calibrated and validated until the satisfactory GEH statistics were met. The study considered travel time and maximum queue length as the primary measures for evaluating traffic operation, while the frequency and type of simulated conflicts and the number of vehicle stops were used to assess safety. In summary, the results indicate that Super DDI designs outperformed DDI when considering adjacent signals and when higher traffic demand exists, while DDI performed similarly or sometimes insignificantly better compared with Super DDI if no adjacent intersections were located in the vicinity and if the demand was lower than DDI's capacity. Therefore, it might be concluded that Super DDI designs should perform better in urban (or populated suburban) areas due to the higher traffic demand and a greater number of adjacent intersections compared with rural areas. The possible reasons behind the notable improvement in traffic operation of Super DDI are attributed to the fact that the design experiences a lower demand on each traffic signal (due to better distribution), and the signals could be coordinated to provide a perfect two-way progression system on the arterial.

While analyzing pedestrian performance, the research team did not find any significant difference in three different peaks and hence considered only PM peak hours. Due to the presence of very low pedestrians, the study considered eight arbitrary distributions of pedestrian volume. Based on this information, different alternatives were created, each design having 16 different scenarios, adding up to a total of 192 scenarios with 960 runs. Pedestrian operation was evaluated in terms of travel time, number of stops, and waiting time, while pedestrian safety was analyzed based on a surrogate performance measure called design flag, introduced by the new National Cooperative Highway Research Program (NCHRP-948) guideline. The study also examined the impact of pedestrians on vehicular operation. The results indicated that a relatively safe condition is expected for pedestrians in the proposed new Super DDI designs compared with CDI and DDI. On the other hand, one of the most popular alternative interchanges, DDI showed concerns in all the aspects of the pedestrian analysis. Despite the very good performance of CDI in terms of pedestrian travel time, pedestrians could have a significant negative impact on vehicle travel time.

Despite the comprehensive simulation series and the analysis conducted in this paper, more studies are recommended in the future using a driving simulator laboratory to evaluate drivers' behavior in Super DDI. Moreover, future studies could further evaluate the Super DDI's pedestrian and bicycle facilities. Findings from this study are expected to help transportation managers and policymakers take necessary actions and decide on management strategies for implementing appropriate alternative interchanges.

1. INTRODUCTION

Due to the efforts made to upgrade the performance of old and failing service interchanges, communities across the nation have seen a paradigm shift toward alternative interchange designs. Nowadays, innovative interchanges are drawing attention to transportation agencies and designers for their potential in accommodating higher traffic demand, reducing delays, and mitigating congestion. In fact, alternative interchanges are designed to take full advantage of road resources and provide the best benefit to different road users.

Most of the existing conventional diamond interchanges (CDI) in the United States were designed and constructed about 60 to 70 years ago, and at the time such high traffic volumes were not anticipated. Throughout the decades, vehicle types and driving habits have significantly changed. According to the Federal Highway Administration (FHWA), there was an average increase of about 3.6 million vehicles each year since 1960 in the United States (*1*). These increased traffic demands have raised many operational and safety concerns while traveling through conventional interchanges, leading to deteriorated traffic conditions especially during peak hours in big urban areas. The FHWA, the National Highway Traffic Safety Administration (NHTSA) and the Federal Motor Carrier Safety Administration (FMSCA) are working with the National Safety Council (NSC) to develop a coordinated approach (i.e., zero vision mission) to reaching zero deaths within the next 30 years. Considering the growing traffic demands, safety, and budget constraints, the investigation of alternative interchange designs has become a vital task for highway engineers.

The diverging diamond interchange (DDI) is one example of searching for new designs to solve problems related to existing (conventional) interchanges. This manuscript introduces the two versions of super diverging diamond interchange (Super DDI) as alternative designs, which are predicted to show high potential to mitigate the problems of failing service interchanges in terms of vehicular operation, safety, and pedestrian performance through the use of microsimulation modeling tools.

1.1 Study Objectives

This study is aimed at achieving three objectives. The first is to identify the most failing and hazardous interchanges in the Denver, Colorado, metro and provide a ranking based on the current traffic operation and safety statistics for future interchange improvements. The second objective is to evaluate the performance of Super DDI in terms of traffic operation, safety, and pedestrian performance. The final objective is to compare the performance of Super DDI design as possible substitute in comparison to DDI, which is the most popular alternative interchange in this era.

1.2 Study Benefits

The research presented in this study helps improve the performance of failing service interchanges in the Mountain-Plains Region (MPR) that can be replaced by either DDI or Super DDI. Figure 1.1 shows the interchanges considered in this study. The research team considered three alternative designs, DDI, Super DDI-1, and Super DDI-2, to make a reasonable comparison with the existing CDIs. Also, each design is kept within the existing right-of-way (ROW) to keep construction costs low. The other alternative designs, such as single point diamond interchange (SPDI) and roundabout diamond interchange, were not included in this study. SPDI is not considered a competitive alternative by most agencies recently due to its considerable construction cost. Also, the capacity of roundabout diamond interchanges would be incomparable with the other designs considered in this study.

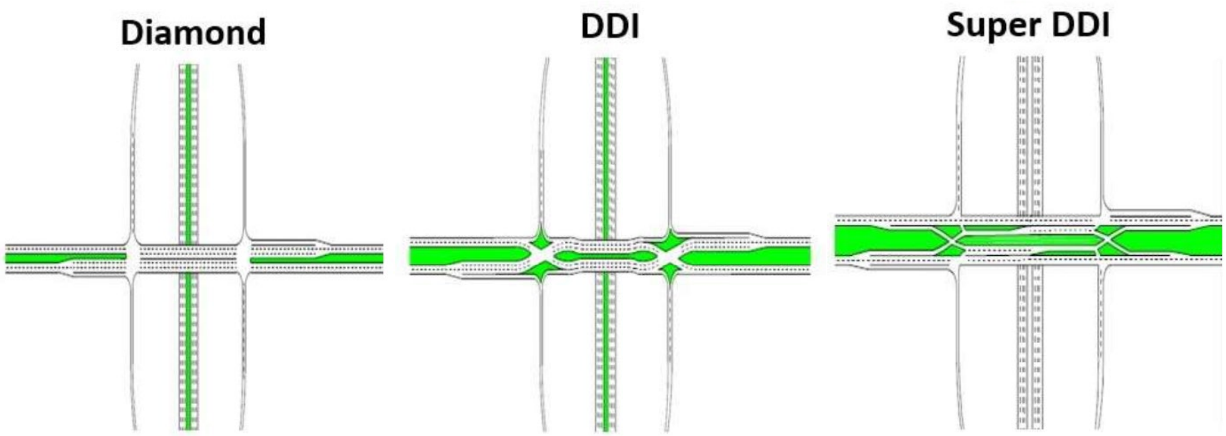


Figure 1.1 Diagrams of interchange types considered in this study (not to scale) (*2*)

As one of the contributions in this study, the performance of the Super DDI was compared with CDI and DDI, including adjacent intersections (in addition to the two intersections of each interchanges). In other words, two networks were considered in this study: (i) when there are only two intersections in the network, and (ii) when there are four intersections (two intersections for the interchange selected and two adjacent intersections). To provide an example, Figure 1.2 shows the two networks considered in one of the case study sites.

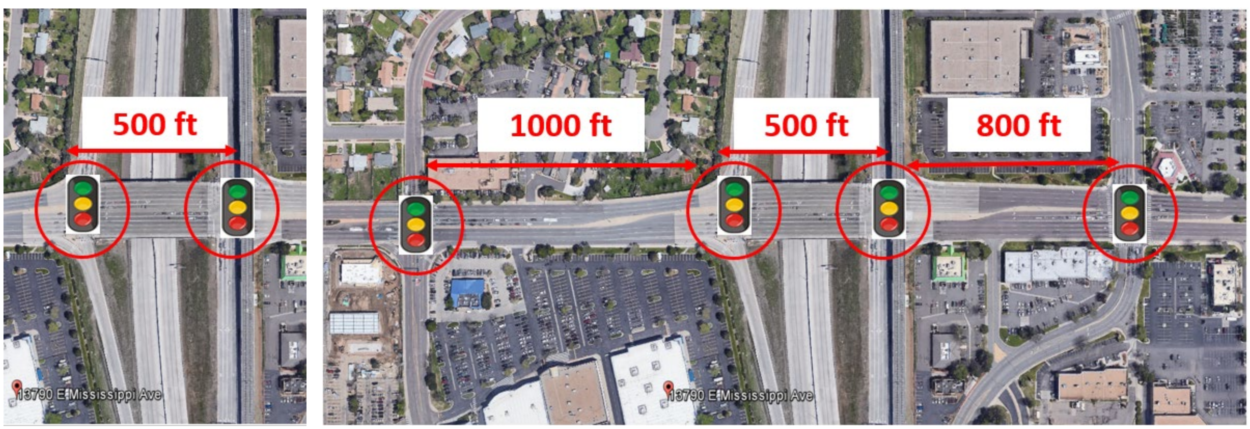


Figure 1.2 Network-1 showing two intersections (left image) and Network-2 showing four intersections (right image)

Findings from this study are expected to help transportation managers and policymakers take necessary actions and decide on management strategies for implementing appropriate interchange designs. In the following paragraph, a description of the characteristics and geometry of Super DDI design is provided to make a clear view for the readers.

1.3 Super Diverging Diamond Interchange

The idea of the Super DDI design was developed during a previous study by the second author of the manuscript, Amir Molan, on alternative interchanges where the synchronized interchange (inspired by the superstreet intersection design) was found to perform very well in high through-traffic conditions (*3*). The performance of Super DDI was found very promising compared with other interchange designs based on the previous studies (*4, 5*). This is mainly because Super DDI combines the characteristics of the synchronized interchange and DDI to boost the performance in both high and low turning traffic conditions. Figure 1.3 shows a sketch of the Super DDI design indicating the direction of left-turn traffic from eastbound (EB on the arterial) and northbound (NB on the freeway ramp) by red and blue lines, respectively. Westbound (WB) and southbound (SB) also follow similar left-turn patterns. Figure 1.3 also indicates that the left turns from the arterial cross each other, whereas all the through and right-turning traffic follow a conventional route.

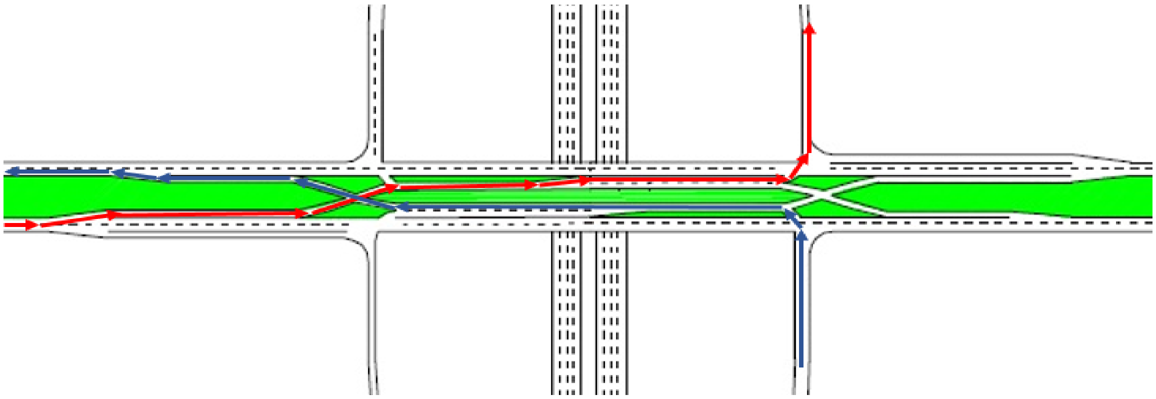


Figure 1.3 Super DDI geometry (the red and blue lines demonstrate the movement of left-turn traffic from EB and from NB ramp-not to scale) (*4*)

The traffic signal phasing of the Super DDI, shown in Figure 1.4, plays the primary role in resulting shorter vehicle travel times compared with typical interchanges with full signals. The Super DDI can offer a perfect two-way signal progression system using half signals that affect only one direction of the arterial instead of full signals that affect both directions of the arterial. The through movements travel independently and the vehicle might only need to stop one time when the left-turn phases are green. In the cases with adjacent traffic signals in the corridor, through traffic might even pass the interchange with no stop because of stopping at the adjacent signal. Moreover, the signal cycle length for a Super DDI would be shorter than a DDI due to having lower traffic volume involved in each node. An example of the perfect progression for through traffic of a Super DDI is illustrated in Figure 1.5.

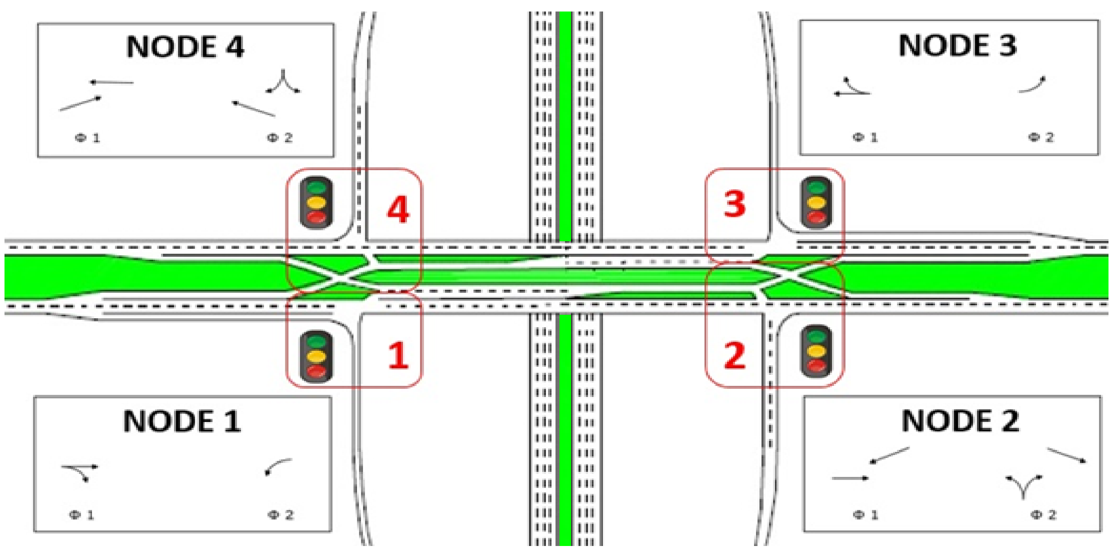


Figure 1.4 Super DDI signal phasing diagram (*4*)

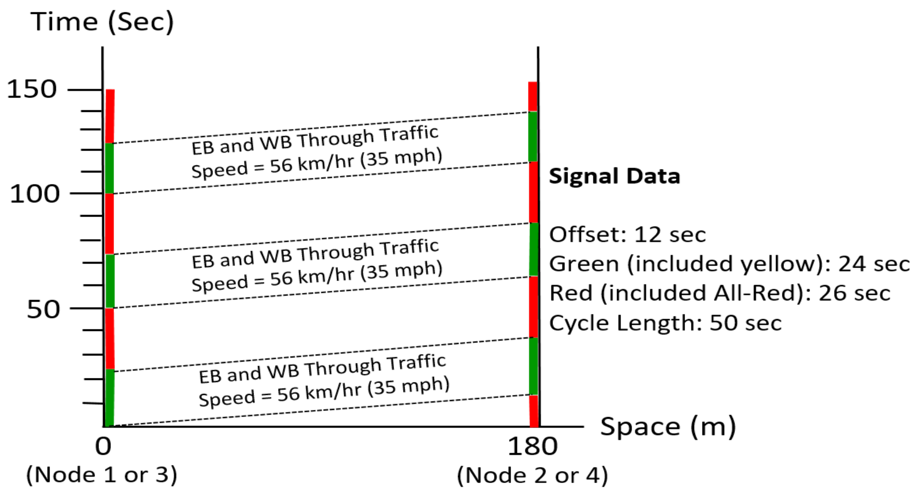


Figure 1.5 An example of the progression system for through traffic in Super DDI (*4*)

Figure 1.6 provides two alternative pedestrian paths for the Super DDI. Among the alternative routes, a side path (red line) would be the best option in terms of pedestrian operation and safety. However, selecting the middle path (blue line) would have to cross four signals (for traveling north-south direction), resulting in longer travel times, which is similar to the typical middle path for pedestrians in DDIs. The blue route would be applicable if there was a shorter bridge width.

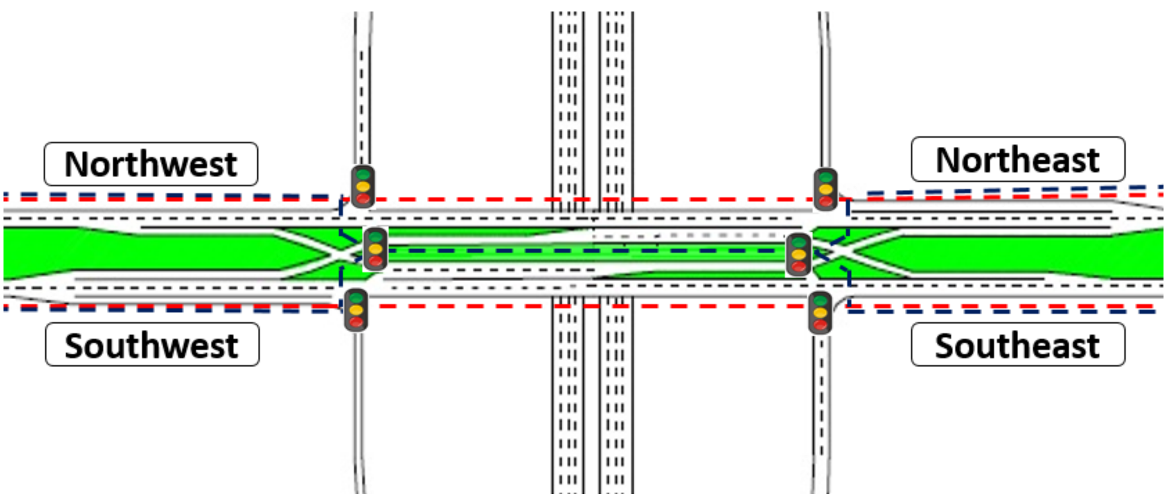


Figure 1.6 Proposed pedestrian paths in Super DDI design (red line = side path, blue line = middle path) (*5*)

The study introduces two different forms of Super DDI design as shown in Figures 1.7 and 1.8. For the first version (Super DDI-1), there are two left-turn lanes on each side of the arterial for turning onto the on-ramp and one left-turn lane for vehicles turning from the off-ramp onto the arterial. The second version (Super DDI-2) is the opposite of version 1. It has one lane for left turns onto the on-ramp from the arterial and two lanes for getting off from the off-ramp. The first design is good for high left-turn volumes from the arterial to the on-ramp, while the second design is better with relatively few left turns for this movement.

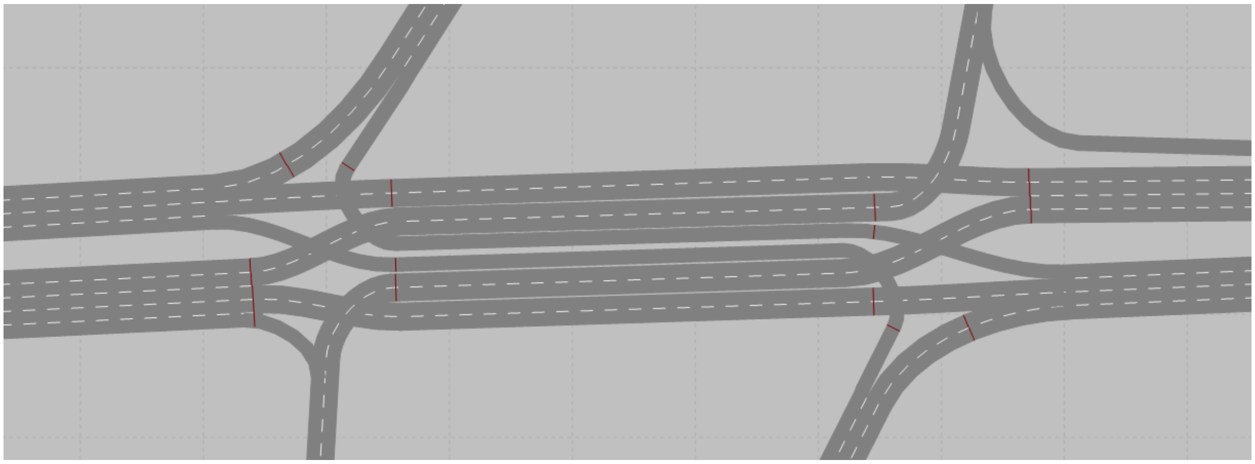


Figure 1.7 Proposed Super DDI Version 1 (Super DDI-1)

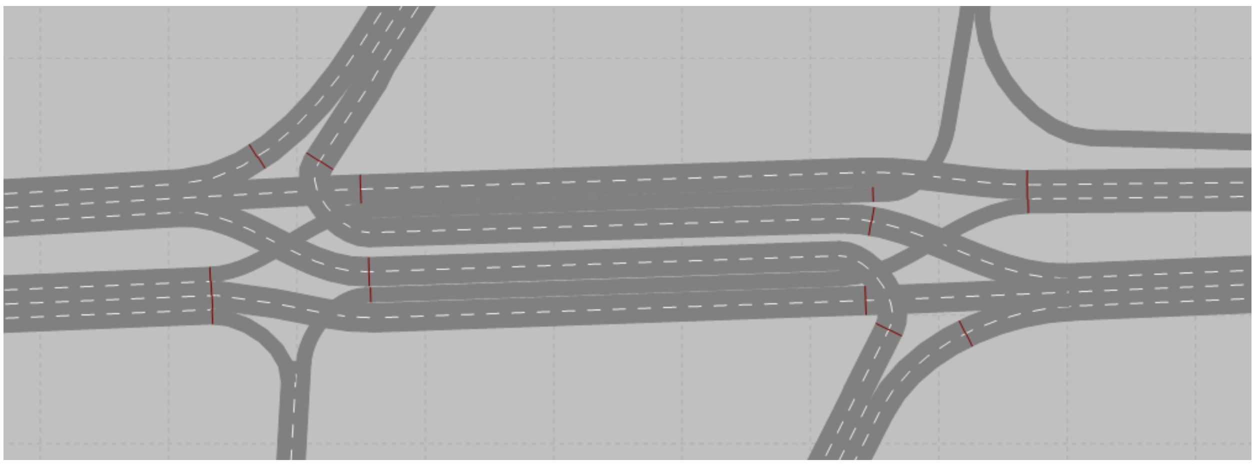


Figure 1.8 Proposed Super DDI Version 2 (Super DDI-2)

2. LITERATURE REVIEW

2.1 Alternative Interchanges

Jughandles in the 1950s and median U-turns in the 1960s were the first alternative designs built in the United States. However, the concept of alternative designs gained more attention in the early 1990s with increased traffic and limited budgets. The next effort in developing alternative designs included roundabouts and SPDIs, which became popular in the 1990s. The single traffic signal was the main appealing feature of the SPDI design that led to its good traffic operation. However, the huge expense of the wide bridge required by SPDI and free-flowing (un-signalized) pedestrian-vehicle conflicts discouraged most of the traffic agencies to make it the first choice.

Alternate design research concerning DDIs began with the first publication by Chlewicki (*6*) and has picked up with many other studies that followed (*7-12*). These studies showed that DDIs can provide good traffic operation (especially for high left-turning traffic) and improve safety performance in terms of minimizing conflict points as compared with single point and conventional diamond interchanges. In comparison to a conventional diamond interchange, DDI was found to reduce delay by 15% to 60% and increase throughput by 10% to 30% for higher traffic volumes, while the performance was found to remain similar for lower volumes (*13*). However, the relatively unsatisfying performance of DDIs in cases of high through traffic and free-flowing vehicle-pedestrian conflicts made their construction questionable, especially in urban areas with a higher demand of both motorized and non-motorized users. Intersection spacings are often short near interchanges in many urban areas and it could be difficult to provide an appropriate two-way progression system for through traffic on the arterial by constructing a DDI.

Other research studies conducted by Molan and Hummer (*14, 15*) introduced synchronized, also known as superstreet intersection, and Milwaukee B interchange designs. The results indicated that Milwaukee B outperforms all interchanges in terms of traffic operation. However, the design limitations included its huge construction costs for having two additional bridges when compared with a DDI. On the other hand, the new synchronized interchange was found to improve vehicle safety by making vehicles conduct a U-turn instead of a conventional left turn. The independent signal operation features on both of the main arterial directions allow a better progression system for through traffic in this design. However, the design was found to struggle with high left-turn and through traffic demands. Therefore, the study concluded that the synchronized interchange performs better with low turning traffic while the DDI is dominant in scenarios with high turning traffic (*14*). While analyzing the queue lengths, the Milwaukee B design was found to have the shortest mean queue lengths, whereas similar mean queue lengths were found in synchronized and DDI (*15*). Molan and Hummer (2020) introduced two new alternative designs called offset diamond interchange (ODI) and Parclo progress A, which aims to mitigate the shortcomings of failing service interchanges (*2, 16*). The results showed significant travel time reduction in ODI when compared with conventional diamond, Parclo A, and the DDI.

The Super DDI, proposed by Molan et al. (2019), was found as a promising alternative service interchange that can improve the performance of failing interchanges (*4*). The other interchanges considered in the research to compare with Super DDI included Milwaukee B, Milwaukee A, Parclo B, Synchronized, DDI, and conventional diamond. The findings of the research indicated that Super DDI could perform significantly better than the existing designs under the conditions tested. The study also showed an average of 18% lower travel time and 49% higher rate of completed tests (as an indicator of capacity) by the Super DDI design when compared with a typical DDI. Another recent study conducted by Mohamed et al. (2020) considered eight interchange designs, including Super DDI, while evaluating the safety performance of the new mega elliptical roundabout interchange (*17*). The study found a significantly better performance of Super DDI compared with DDI in terms of safety.

2.2 Analysis Techniques

Several studies that analyze the performance of newly proposed interchanges have been found in the literature. Table 2.1 shows a summary of previous research efforts in evaluating the performance of innovative designs considering the names of alternative interchanges, analysis methods, and the inclusion of evaluation techniques. As a whole, two commonalities are found among these research studies. First, all of these studies analyzed alternative interchanges, where some investigated operational performance (*2, 4, 6, 7, 14, 15, 18-20, 22, 24-28*), some evaluated safety performance (*5, 17, 23*), some analyzed both traffic operation and safety (*16*), and some explored pedestrian performance (*21*). Second, the current body of literature on alternative interchanges mostly used VISSIM microsimulation tools to examine the interchange efficiency.

Crash analysis using observational crash data is a common method of investigating transportation safety (*29-34*). However, the limitations of this method included randomness of the crashes in evaluating the safety impacts of any improvement, underreporting issues related to crash records, and the inability to assess the safety of the new types of interchanges. Therefore, a new alternative tool called the Surrogate Safety Assessment Model (SSAM) is most widely used to identify safety aspects of unconventional interchanges today (*5, 17, 23, 35*).

Table 2.1 Alternative service interchange studies

Authors	Alternative Interchange	Method/Analyzing Tool	Evaluation
Mohamed et al. (2020) (*17*)	Mega elliptical roundabouts	VISSIM microsimulation and SSAM	Safety
Molan and Hummer (2020) (*2*)	Offset diamond interchange (ODI)	VISSIM microsimulation	Operation
Molan and Hummer (2020) (*16*)	Parclo progressA interchange	VISSIM microsimulation and SSAM	Operation and safety
Molan and Hummer (2020) (*15*)	Synchronized and Milwaukee B interchange	VISSIM microsimulation	Operation
Molan et al. (2019) (*4*)	Super DDI	VISSIM microsimulation	Operation
Molan et al. (2019) (*5*)	Super DDI	VISSIM microsimulation and SSAM	Safety
Mohamed et al. (2019) (*18*)	Mega elliptical roundabouts	VISSIM microsimulation	Operation
Sultana et al. (2018) (*19*)	Alternative diamond interchanges	VISSIM microsimulation	Operation
Molan and Hummer (2018) (*14*)	Synchronized and Milwaukee B interchange	VISSIM microsimulation	Operation
Sutherland et al. (2018) (*20*)	Displaced partial cloverleaf (DPC) interchange	VISSIM microsimulation	Operation
Molan and Hummer (2018) (*21*)	Synchronized and Milwaukee B interchange	VISSIM microsimulation	Pedestrian
Zhao et al. (2018) (*22*)	Median U-turn in Parclo interchange	Mixed-integer nonlinear program	Operation
Molan and Hummer (2017) (*23*)	Synchronized and Milwaukee B interchange	VISSIM microsimulation and SSAM	Safety
Reid et al. (2008) (*24*)	Dual-system urban interchange (DSUI)	VISSIM microsimulation	Operation
Eyler (2005) (*25*)	Reverse Parclo B	VISSIM microsimulation	Operation
Bared et al. (2005) (*7*)	Double crossover intersection and DDI	VISSIM microsimulation	Operation

Table 2.1 continued

Jones and Selinger (2003) (*26*)	Single-point urban interchange (SPUI)	CORSIM simulation	Operation
Thompson et al. (2003) (*27*)	W-interchange	CORSIM simulation	Operation
Chlewicki (2003) (*6*)	Synchronized split-phasing interchange and DDI	SimTraffic simulation	Operation
Bonneson, 1992 (*28*)	SPUI	1985 Highway Capacity Manual	Operation

2.3 Pedestrian Studies

There have been numerous pedestrian studies found in the literature, which analyzed and identified the contributing factors associated with pedestrian crashes at intersections (*36-44*). In general, pedestrian risks were found to decrease with increasing pedestrian flows or activities based on the previous studies (*36, 40, 43, 44*). While analyzing the association between intersection characteristics and pedestrian crash risk, more pedestrian crashes were likely to occur at intersections with more right-turn-only lanes, nonresidential driveways within 50 feet, commercial properties within 0.1 mile, and a greater percentage of residents within 0.25 miles who were younger than age 18 (*39*). Another research study conducted by Brosseau et al. (2013) recognized long waiting time for the walk interval, short walk interval, and the high turning volume of vehicles at conflict points on permissive green signals as the essential variables for pedestrian crashes at intersections (*41*). The study also revealed that pedestrians tend to commit a violation either when the clearance time (i.e., flashing “DO NOT WALK” interval) is longer or shorter than needed. Lee et al. (2019) introduced various surrogate measures for pedestrian exposure at intersections, including the presence of schools, car-ownership, pavement condition, sidewalk width, bus ridership, intersection control type, and presence of sidewalk barriers (*44*). Throughout the decades, several strategies were investigated to reduce pedestrian-vehicle crashes. As one example, the leading pedestrian interval (LPI) strategy was found to reduce pedestrian-vehicle crashes by 58.7% (*36*). Zegeer et al. (2012) also documented potential countermeasures and strategies for improving pedestrian safety from an international perspective (*45*).

Regarding the pedestrian operational performance, the existing methodology in the Highway Capacity Manual (HCM) cannot provide the necessary accuracy for estimating delay and level of service (LOS) (*46*). Apart from ignoring the effect of right-turning traffic, the HCM method also did not consider other possible parameters such as 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 (*47*). Zhao and Liu (2017) proposed a new pedestrian control delay model, which was found to increase the estimation accuracy by 20% when compared with the HCM model (*48*). While optimizing traffic signals at an intersection, pedestrian traffic typically receives minor priority compared with vehicular traffic. To address this issue, Yu et al. (2017) developed a unified signal timing optimization framework to optimize signals for pedestrians and vehicles at a signalized intersection simultaneously (*49*).

Recently, a new guide for alternative intersections and interchanges (AIIs) has been developed to assist transportation practitioners in improving and integrating pedestrian and bicycle safety considerations at AIIs through planning, design, and operational treatments (*50*). The guide documents current best practices for measuring the effectiveness of AII treatments by evaluating the safety and operational outcomes. In addition, the guide introduces “design flags,” as a surrogate for quantitative performance measures for pedestrians and bicyclists, which have been developed from research that includes literature reviews, focus groups with users of these facilities, online surveys, expert panels, and practitioner experience.

3. RESEARCH METHODOLOGY

Simulation modeling has been found as the most appropriate evaluation technique for this study since field testing was not possible for analyzing the proposed design not being built. PTV VISSIM (2020 version), SSAM, and Synchro (version 11) were selected as the main analysis tools for this research. VISSIM is the most widely used microsimulation package as it is able to model various aspects of transportation engineering from the research on user behavior (*51, 52*) or vehicle dynamics (*53, 54*) to research related to operation and safety of transportation infrastructures (*2-5, 14-18, 55, 56*). SSAM, introduced in a Federal Highway Administration (FHWA) report (*57*), is a combination of microscopic simulation and automated conflict analysis to study the frequency and character of narrowly averted vehicle-to-vehicle collisions (or near misses) considering a threshold for the time-to-collision (TTC) in traffic. Synchro, a popular macroscopic software, can optimize signal timing and analyze the performance of signalized intersections based on the Highway Capacity Manual (2016). This study first applied Synchro to attain optimum signal timing and cycle length. Then, signal data were imported into VISSIM to estimate the performance of each interchange design. After that, VISSIM trajectory files generated from each simulation were analyzed through the SSAM to determine the types and number of conflicts for each design.

3.1 Site Selection, Data Collection, and Geometry Design

As part of a comprehensive study, the research team investigated the most failing and hazardous interchanges in the Denver, Colorado, metro using the critical lane volume (CLV) method. Out of 62 service interchanges, the research team identified the top 11 failing interchanges as shown in Figure 3.1. These 11 interchanges are failing because the volume-to-capacity (v/c) ratios for the interchanges were found to be over one. Among these, three interchanges (I-225 and Mississippi Ave; I-25 and 120th Ave; and I-25 and Hampden Ave) were identified as potential candidates for future retrofits to be replaced by either DDI or Super DDI (marked by the red box in Figure 3.1) because of available right-of-way (ROW).

Ranking	Interchanges	Type	Super DDI could be the Preferance?
1	**Hampden & I-25**	**Diamond**	**YES**
2	Belleview & I-25	Diamond	Maybe
3	**Mississippi Ave & I-225**	**Diamond**	**YES**
4	Kipling St & I-70	Diamond	YES
5	Orchard & I-25	Diamond	Maybe
6	290-TOWER & I-70	Parclo A	Maybe
7	CountyLine & I-25	Parclo A	Maybe
8	Arapahoe & I-25	Diamond	YES
9	**120th Ave & I-25**	**Diamond**	**YES**
10	SH-7 & I-25	Diamond	YES
11	Dry Creek Rd & I-25	Diamond	Maybe

Figure 3.1 Top 11 most failing and hazardous interchanges in Denver, Colorado, metro

In general, there were three reasons for selecting those interchanges. First, the v/c ratios for all three interchanges were over one (which means demand is more than the capacity) in 2018. Second, the selected interchanges have 10 traffic lanes on the bridge, which provides an appropriate bridge width for constructing either a DDI or a Super DDI. Third, reviewing historical crashes, nearly 1,400 crashes (by summing up for the three interchanges) occurred during the last five years (2016-2020).

All the required traffic data associated with the selected interchange were obtained from the Denver Regional Council of Governments and the Colorado Department of Transportation (CDOT), or by contacting cities. Specifically, the traffic proportions, turning movements, and pedestrian movements were collected for an hour at the AM, Noon, and PM peaks. Note that traffic volumes of the adjacent intersections at one of the case study sites, I-25 and Hampden Ave, were not available. Therefore, the adjacent signals were excluded in the network of this interchange. The study also utilized other non-traditional sources, such as Google Map and Google Earth Pro, to determine the geometrical features as well as the posted speed limits of the network. The obtained traffic volume was projected to the year 2020 and 2030 assuming an annual growth rate of 2% to simulate how the models would function in the existing and future conditions, respectively. It should be noted that the effect of connected and autonomous vehicles (CAVs) could be considered for evaluating the performance in 2040 or 2045 (considering a design year of 20-25 years). Since evaluating CAVs was out of the scope of the research, 2030 was considered as the design year for predicting the future operation of the interchanges because only the minority portion of vehicular traffic should be CAVs in 2030. Based on the field data, the truck composition applied in the analysis was 2% and 5% depending on whether the traffic came from an adjacent street or an off-ramp, respectively. Tables 3.1 and 3.2 show the existing (2020) mean traffic volumes on each turning movement of the selected interchanges and the corresponding adjacent intersections.

The research team did not find any significant difference in analyzing pedestrian performance in three different peaks and hence considered only PM peak hours for conducting pedestrian analysis. However, the pedestrian volume collected from the CDOT was very low. Therefore, the study considered various distributions of pedestrian volume, where 45 and 90 pedestrians per hour per route were selected to represent moderate and high demand, respectively. There are three reasons for choosing relatively high pedestrian volume in the simulation. First, other than a few downtown areas where these interchanges would not be applicable, the United States has few pedestrian demands at crossing interchanges other than those simulated in this study. Second, lower pedestrian demands would lead to instability in the results due to insufficient sample sizes. Third, different pedestrian demand levels up to a high level would help provide performance variation at various designs since pedestrians tend to travel in packs or bunches rather than lanes, causing each other little extra delay. Among the two alternative pedestrian paths for the Super DDI design shown in Figure 1.3, this study only considered the side path alternative for the pedestrian analysis. All the designs tested in the simulation models had two pedestrian paths that were 10 feet wide on the side except for the DDI, which had the path in the middle of the bridge. The choice between a side path and a middle path depends on the context; however, the research team considered the middle path for DDI due to safety concerns (i.e., to avoid free flow vehicle-pedestrian conflicts) with crossing the left entrance to the on-ramp in the side paths of DDIs.

The geometric layout of the investigated interchanges with the corresponding pedestrian paths (in green lines) is shown in Figures 3.2, 3.3, and 3.4. The figures also demonstrate the ability to implement all of the designs within the original ROW limits. Noted that no widening of the bridge would be necessary to deploy the alternative designs. Based on the FHWA's DDI Informational Guide (*58*), a 45-degree angle was considered for the crossovers in both DDI and Super DDI designs to minimize the wrong-way potential. As illustrated in Figures 3.2-3.4, all the designs have 10 traffic lanes on the bridge.

Table 3.1 Entry traffic volume (2020) for the selected interchanges

Location	Time	Arterial (EB)			Arterial (WB)			Ramp (NB)		Ramp (SB)		Total
		LT	T	RT	LT	T	RT	LT	RT	LT	RT	
I-225 and Mississippi Ave	AM	393	1226	338	355	2045	500	354	356	319	617	6503
	Noon	555	1504	381	395	1481	514	379	377	430	600	6616
	PM	512	1930	453	446	1799	408	397	441	470	412	7268
I-25 and 120th Ave	AM	463	883	705	703	2801	381	1070	644	255	746	8651
	Noon	325	1201	804	785	1785	304	698	740	269	343	7254
	PM	710	1948	1097	919	1737	384	637	780	427	525	9164
I-25 and Hampden Ave	AM	172	1138	1386	835	2046	944	776	423	442	108	8270
	Noon	201	1255	939	609	1585	744	980	658	729	218	7918
	PM	116	1776	1023	597	2044	679	1095	606	892	174	9002

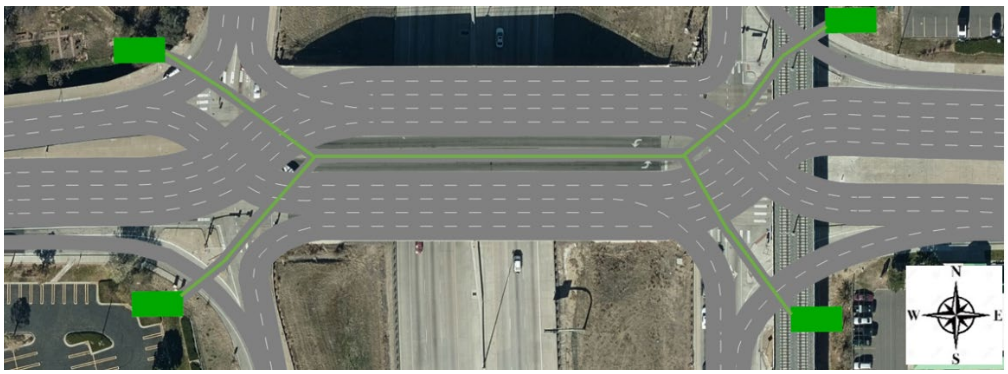
Note: LT = Left Turn, T = Through, RT = Right Turn

Table 3.2 Entry traffic volume (2020) for the adjacent intersections

Adjacent Intersections		Time	EB			WB			NB			SB			Total
			LT	T	RT	LT	T	RT	LT	T	RT	LT	T	RT	
I-225 and Mississippi Ave	Potomac St	AM	42	1158	167	706	1698	150	77	69	353	125	122	37	4704
		Noon	33	1458	136	512	1358	98	123	75	444	131	132	66	4566
		PM	47	1708	90	427	1570	134	146	138	628	208	158	73	5327
	Abilene St	AM	178	1202	228	39	2071	91	294	151	27	53	75	170	4579
		Noon	380	1326	244	84	1250	108	293	204	85	133	159	358	4624
		PM	357	1757	350	99	1640	124	341	217	96	117	206	304	5608
I-25 and 120th Ave	Huron St	AM	101	1113	232	435	1839	143	232	482	260	409	826	211	6283
		Noon	199	1184	115	299	1361	169	206	495	367	383	418	124	5320
		PM	265	1635	310	401	1434	106	349	1230	792	546	685	203	7956
	Grant St	AM	173	865	62	83	2237	24	174	52	50	43	76	156	3995
		Noon	133	1563	143	183	1586	45	292	83	174	110	58	179	4549
		PM	158	2121	223	209	1590	33	276	93	323	90	87	301	5504

Note: LT = Left Turn, T = Through, RT = Right Turn

Existing CDI

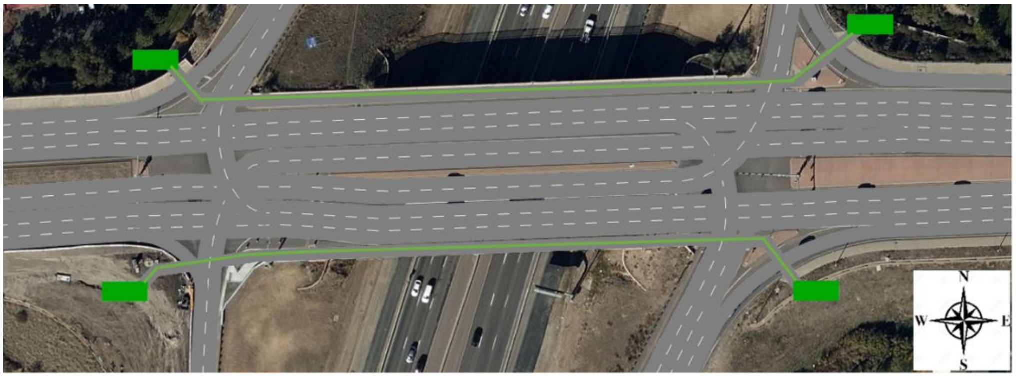


Possible DDI Configuration

Possible Super DDI-1 Configuration

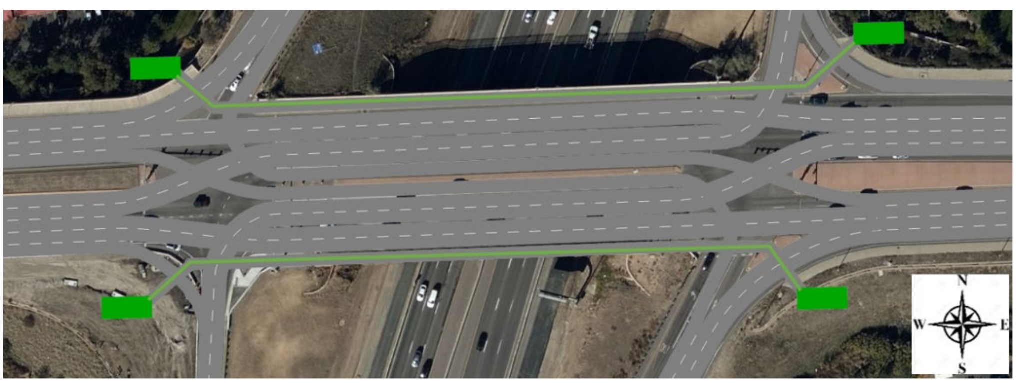
Possible Super DDI-2 Configuration

Figure 3.2 Geometric layout of the investigated interchanges at I-225 and Mississippi Ave

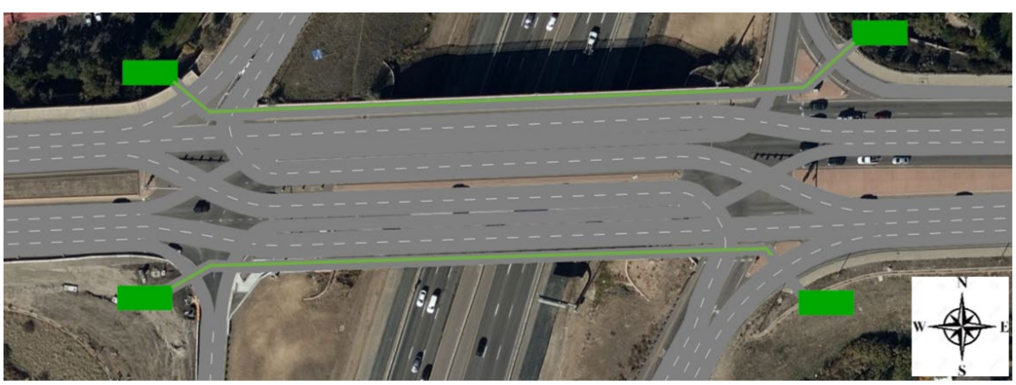


Existing CDI

Possible DDI Configuration

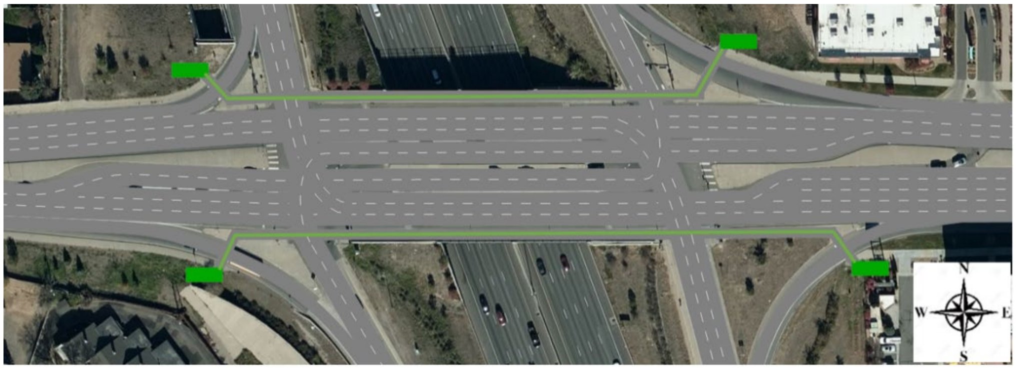


Possible Super DDI-1 Configuration



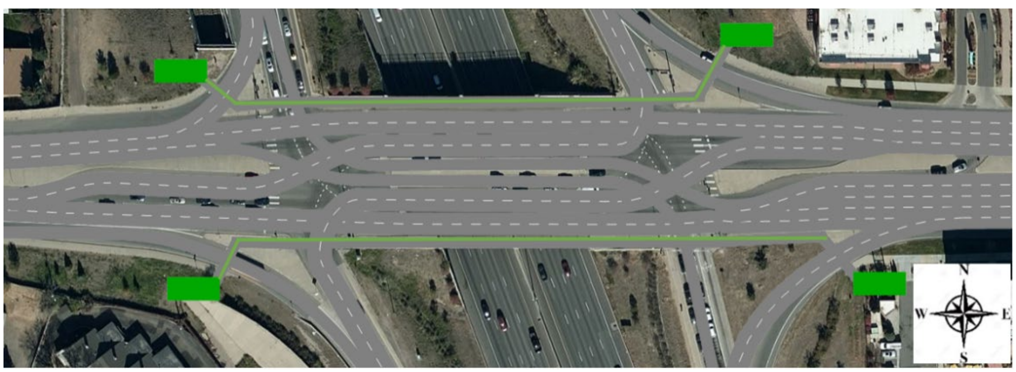
Possible Super DDI-2 Configuration

Figure 3.3 Geometric layout of the investigated interchanges at I-25 and 120th Ave

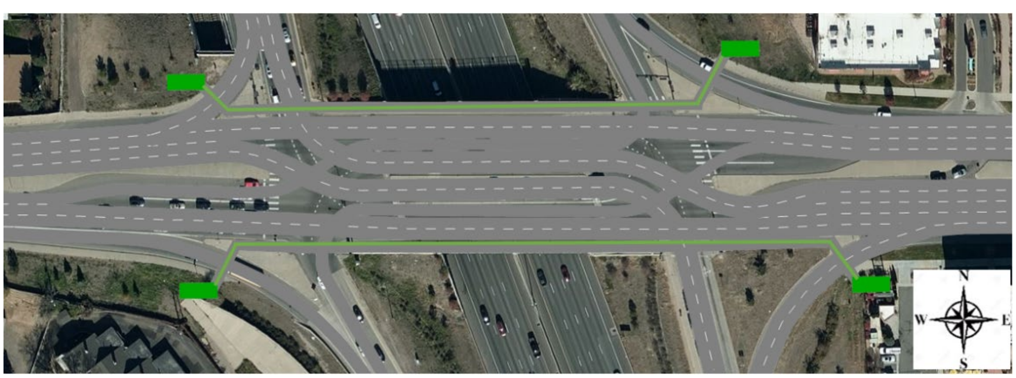


Existing CDI

Possible DDI Configuration



Possible Super DDI-1 Configuration



Possible Super DDI-2 Configuration

Figure 3.4 Geometric layout of the investigated interchanges at I-25 and Hampden Ave

3.2 Simulation Modeling Procedure

3.2.1 Traffic Signal Design

There is no doubt that signal timing and phasing play an important role in the performance of an interchange. In order to ensure the accuracy of signal timing in the simulation, all the signals were designed and optimized using Synchro. Table 3.3 displays the cycle lengths used for each model that was determined with guidance from Synchro. Note that all traffic signals in the system had the same or multiple of the maximum cycle length to provide a signal progression system. Also, maximum and minimum cycle lengths were set equal to 180 and 40 seconds in Synchro.

Table 3.3 Traffic signal cycle lengths (in seconds) for various designs at the selected interchanges

Location	Design		2020			2030		
			AM	Noon	PM	AM	Noon	PM
I-225 and Mississippi Ave	Without adjacent signals	CDI	90	80	90	150	110	180
		DDI	60	50	60	75	60	80
		Super DDI-1	55	55	90	65	100	140
		Super DDI-2	60	60	90	75	90	150
	With adjacent signals	CDI	175	125	135	175	165	180
		DDI	145	145	150	150	145	150
		Super DDI-1	75	55	60	75	70	75
		Super DDI-2	75	55	60	75	70	75
I-25 and 120th Ave	Without adjacent signals	CDI	150	100	150	150	150	150
		DDI	130	80	140	150	150	150
		Super DDI-1	75	75	75	75	75	75
		Super DDI-2	75	75	75	75	75	75
	With adjacent signals	CDI	150	110	150	150	150	150
		DDI	150	145	145	150	150	150
		Super DDI-1	75	65	75	75	75	75
		Super DDI-2	75	55	75	75	70	75
I-25 and Hampden Ave	Without adjacent signals	CDI	150	100	160	180	170	180
		DDI	60	60	90	80	85	150
		Super DDI-1	75	75	75	75	75	75
		Super DDI-2	75	75	75	75	75	75

Higher cycle lengths tend to increase pedestrian waiting time, which ultimately results in the possibility of committing violations (e.g., jaywalking). The clearance time of the pedestrian signal is another important factor to be considered for pedestrian safety. Since the field signal data for the selected locations were not available, the study considered a 7-second clearance time at one-lane crossings and added 3.5 seconds for any additional lane based on the previous studies (*5, 21*). Note that all the minimum green times were satisfied during the signal design. Pedestrians at the on-ramp and off-ramp crossings at the CDI and DDI have four free-flowing conflicts with the vehicle movements, and hence pedestrians had to yield before crossing in those cases. As for the Super DDI design, there were signals at every pedestrian crossing except for the free-flowing right-turn vehicles entering the on-ramp. Table 3.4 provides the cycle lengths with the red interval for pedestrians used for each model that was determined with guidance from Synchro.

Table 3.4 Average signal cycle length with the corresponding red interval of pedestrians

Interchange Type	Traffic Volume Year	I-225 and Mississippi Ave		I-25 and 120th Ave		I-25 and Hampden Ave	
		CL (sec)	R (sec)	CL (sec)	R (sec)	CL (s)	R (sec)
CDI	2020	90	28	150	47	160	42
	2030	180	47	150	47	180	46
DDI	2020	60	36	140	76	90	51
	2030	80	46	150	81	150	81
Super DDI-1	2020	60	25	75	35	75	32
	2030	75	30	75	35	75	32
Super DDI-2	2020	60	27	75	35	75	30
	2030	75	30	75	35	75	30

Note: CL = Average cycle length of the scenarios, and R = Average red interval of pedestrians (clearance time of pedestrians is included)

3.2.3 Simulation Scenarios

In VISSIM, the geometry of each design was created using its built-in background maps. The volume inputs with routing decisions were added for further model development. The geometry was also checked through Google Street View to ensure that the models represent actual conditions. The available field speed data were used to create custom cumulative probability functions for vehicle speeds, with the posted speed being considered as the 85th percentile. Reduced speed areas were placed where vehicles made any turnings (e.g., right turns on and off of the freeway) to accurately simulate driving behavior. Each design was tested considering with and without adjacent intersections (except for I-25 and Hampden Ave), and three different peak periods. Based on this information, different alternatives were created, and microsimulation models were developed for the existing (the year 2020) and projected (the year 2030) traffic volumes, each location having 48 different scenarios, adding up to a total of 120 scenarios, as shown in Table 3.5.

Due to the presence of very low pedestrians, the study considered eight arbitrary distributions of pedestrian volume. There were four pedestrian origin points, each having one possible route summed up to a total of four routes (from southeast to southwest and vice versa, from northeast to the northwest and vice versa). Note that no pedestrians crossed the arterial. In other words, they only crossed the bridge. In order to perform pedestrian analysis, microsimulation models were developed for the existing (the year 2020) and projected (the year 2030) traffic volumes only for the PM peak period, each design having 16 different scenarios, adding up to a total of 192 scenarios, as shown in Table 3.6. Table 3.7 illustrates eight arbitrary distributions of pedestrian volume considered for this study.

Table 3.5 Simulation scenarios defined for the operational and safety analysis

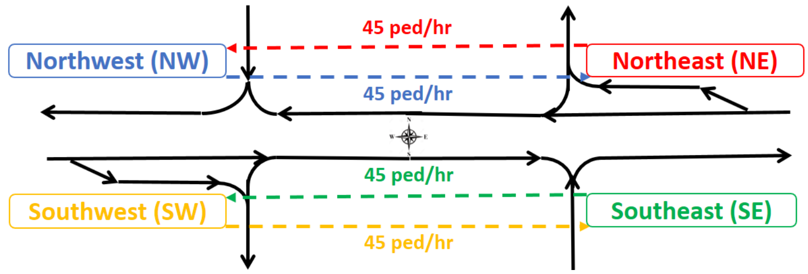
Location	Interchange design	Corridor	Traffic volume		Total scenarios
			Year	Peak hour	
I-225 and Mississippi Ave I-25 and 120th Ave I-25 and Hampden Ave	CDI DDI Super DDI-1 Super DDI-2	With/without adjacent intersections (except only for Hampden Ave)	2020 2030	AM Noon PM	120 scenarios with 600 simulation run

Table 3.6 Simulation scenarios defined for the pedestrian analysis

Location	Interchange design	Pedestrian volume	Traffic volume		Total scenarios
			Year	Peak hour	
I-225 and Mississippi Ave I-25 and 120th Ave I-25 and Hampden Ave	CDI DDI Super DDI-1 Super DDI-2	Eight arbitrary distributions of pedestrians demonstrated in Table 3.6	2020 2030	PM	192 scenarios with 960 simulation run

The simulation models were run for 4,500 seconds (75 minutes) with 900 seconds (15 minutes) as warm-up time. To include the effect of different simulation seeds in the analysis, each VISSIM simulation test was run five times, while an average of them was chosen as the representative outcome of each test. Also, a factorial analysis method was applied to ensure that many more than just two samples contributed to any comparison made in the analysis. For this reason, a two-way analysis of variance (ANOVA) with post hoc tests at 95% confidence level (p-value = 0.05) was considered to identify the significant differences between the performance measures of the interchanges using R statistical software.

Table 3.7 Distribution of pedestrian volume

<table>
<tr><th>Pedestrian Volume Distribution</th><th>Description of the Scenario</th></tr>
<tr><td>1. All 45</td><td>
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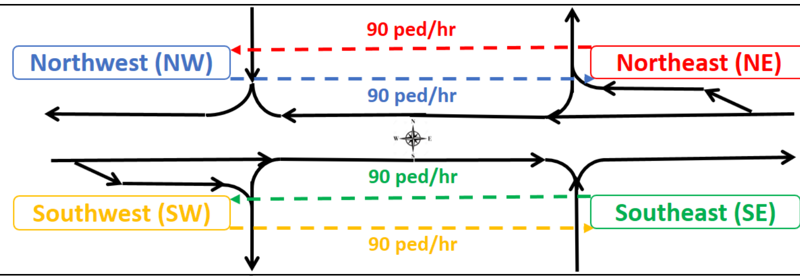
<tr><td>2. All 90</td><td>
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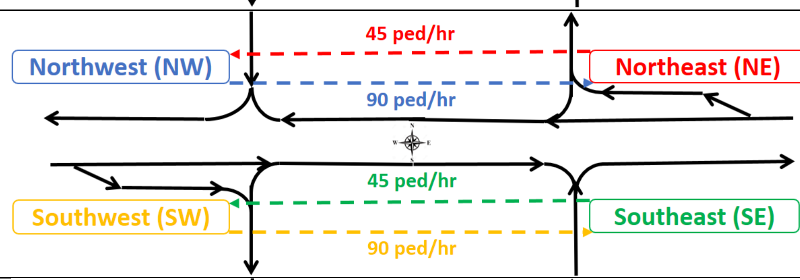
<tr><td>3. E 45 - W 90</td><td>
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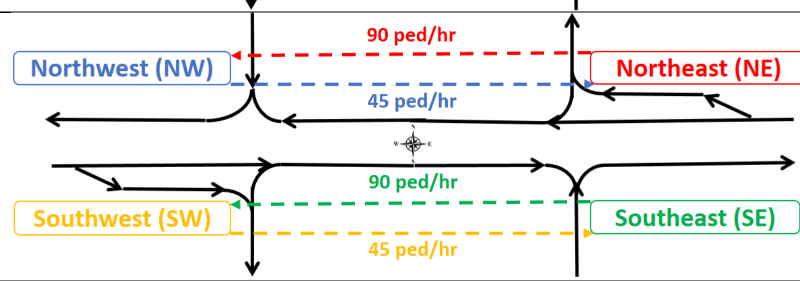
<tr><td>4. E 90 - W 45</td><td>
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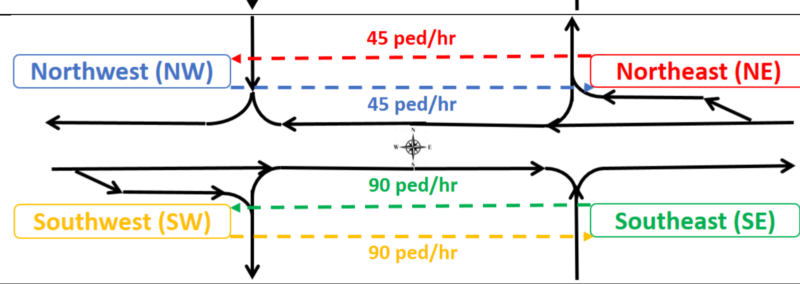
<tr><td>5. N 45 - S 90</td><td>
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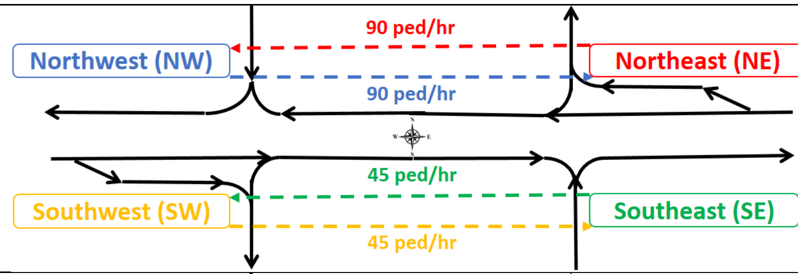
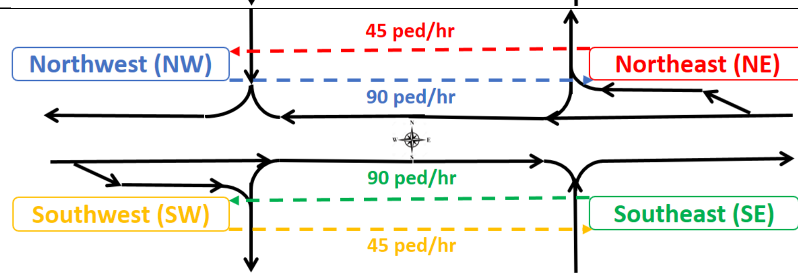
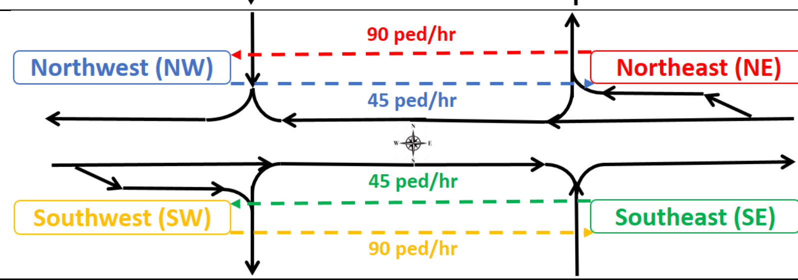
Table 3.7 continued

6. N 90 - S 45	
7. NE & SW 45 - NW & SE 90	
8. NE & SW 90 - NW & SE 45	

3.2.4 Pedestrian Behavior

Pedestrian and driver behaviors are crucial in planning, designing, and operating highway facilities. Pedestrian crossing outside of a marked or unmarked crosswalk, also defined as jaywalking, is one of those pedestrian behaviors that may highly impact safety and operations. However, jaywalking was not considered in the models since the main focus of this study was to evaluate pedestrian performance in the newly proposed interchange rather than investigating pedestrian behaviors. Pedestrians typically adjust their speed based on many traffic and non-traffic parameters, including age, gender, type of crossing, conflicting traffic volume, time of day, and day of the week. From a review of previous studies, it was found that pedestrian speed follows a normal distribution with 70% to 80% of the speed observations near the average. Ishaque and Noland (2009) indicated 4.6 to 5.8 feet per second (fps) of pedestrian speeds in their sample (*59*), while a range from 4 to 4.5 fps was reported by Marisamynathan and Vedagiri (*60*). Another study in Poland mentioned the range of pedestrian speeds from 3.6 to 4.6 fps (*42*). The current research applied pedestrian speeds according to field data collection done by a previous study on pedestrian performance at superstreet intersections (*61*). Based on that study, pedestrian speeds were categorized into two groups: (i) 91% as walking pedestrians with a mean speed of 5 fps, and (ii) the remaining 9% as running pedestrians with a mean speed of 9.6 fps. The priority rule of vehicles and pedestrians on the free-flow crossings was set in such a way that drivers had to stop for the pedestrians when pedestrians could find a minimum gap of 3 seconds or longer to initiate a crossing.

3.3 Calibration and Validation

The first model built on VISSIM was CDI considering with and without adjacent intersections. The CDI model is the baseline model for other configurations to be modeled after. However, before analyzing the existing CDI model, it was calibrated and validated. Model calibrations include setting the driving behavior of the model to ensure that they are reflective of real driving behavior and that origin and destination points are directed correctly. Mean vehicle speeds were set at 40 mph for passenger cars and 35 mph for trucks on the arterials. As for the ramps, the mean vehicle speed of 35 mph was set at I-225 and Mississippi Ave, while 45 mph was set at the other two locations. Based on data collected in previous studies (*62, 63*), 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 3.5 seconds or more in traffic on the main route based on typical driving regulations in most U.S. states. While designing signal timing, yellow and all-red intervals of 4 and 2 seconds were chosen based on the Manual on Uniform Traffic Control Devices (MUTCD) (*64*).

Once the models are calibrated, the next step is validation where the simulated traffic volumes of at least five runs are compared to the real-world hourly traffic. Note that we are testing new alternative designs that are not built in the location, so we could only validate the existing (2020) design. The simulated traffic volumes are compared with the real world volumes using the GEH statistic formula, as shown in Equation 1 (*65*):

$$GEH = \sqrt{\frac{2(M-C)^2}{M+C}} \tag{1}$$

Where M is the hourly traffic volume from the simulated model and C is the corresponding real-world traffic. For the baseline model, a GEH value of less than 5 indicates a well-calibrated model (i.e., a good match with the field data). If the values fall between 5 and 10, they might still be acceptable, but further calibration is recommended. Values greater than 10 demonstrate serious calibration issues with the model, warranting additional adjustments. Tables 3.8, 3.9, and 3.10 show the GEH values for the existing CDI for various scenarios at the selected locations. As seen, the calculated GEH statistics met the requirements (less than 5), which indicated that the model was successfully calibrated. The calibrated driver behavior information was then applied to the other interchange designs in VISSIM.

Table 3.8 GEH statistics for I-225 and Mississippi Ave interchange

Scenario		Movement		Mean of each turning movement		GEH statistics	< 5?
				Field volume	Simulated volume		
Without adjacent intersections	2020 AM	Arterial	Left turn	374	313	3.29	Yes
			Through	1636	1727	2.23	Yes
			Right turn	419	360	3.02	Yes
		Ramp	Left turn	337	343	0.35	Yes
			Right turn	487	480	0.32	Yes
	2020 Noon	Arterial	Left turn	475	392	3.99	Yes
			Through	1493	1520	0.71	Yes
			Right turn	448	372	3.73	Yes
		Ramp	Left turn	405	406	0.05	Yes
			Right turn	489	487	0.07	Yes
	2020 PM	Arterial	Left turn	479	395	4.04	Yes
			Through	1865	1956	2.09	Yes
			Right turn	431	358	3.68	Yes
		Ramp	Left turn	434	431	0.14	Yes
			Right turn	427	429	0.10	Yes
With adjacent intersections	2020 AM	Arterial	Left turn	374	429	2.74	Yes
			Through	1636	1663	0.68	Yes
			Right turn	419	432	0.61	Yes
		Ramp	Left turn	337	343	0.35	Yes
			Right turn	487	480	0.32	Yes
	2020 Noon	Arterial	Left turn	475	393	3.96	Yes
			Through	1493	1568	1.93	Yes
			Right turn	448	399	2.36	Yes
		Ramp	Left turn	405	418	0.67	Yes
			Right turn	489	479	0.43	Yes
	2020 PM	Arterial	Left turn	479	481	0.09	Yes
			Through	1865	1892	0.63	Yes
			Right turn	431	487	2.64	Yes
		Ramp	Left turn	434	442	0.41	Yes
			Right turn	427	427	0.02	Yes

Table 3.9 GEH statistics for I-25 and 120th Ave interchange

Scenario		Movement		Mean of each turning movement		GEH statistic	< 5?
				Field volume	Simulated volume		
Without adjacent intersections	2020 AM	Arterial	Left turn	583	499	3.61	Yes
			Through	1842	1852	0.23	Yes
			Right turn	543	483	2.65	Yes
		Ramp	Left turn	663	663	0.02	Yes
			Right turn	695	689	0.23	Yes
	2020 Noon	Arterial	Left turn	555	466	3.96	Yes
			Through	1493	1666	4.35	Yes
			Right turn	554	498	2.44	Yes
		Ramp	Left turn	484	495	0.50	Yes
			Right turn	542	537	0.22	Yes
	2020 PM	Arterial	Left turn	815	745	2.51	Yes
			Through	1843	1945	2.36	Yes
			Right turn	741	653	3.33	Yes
		Ramp	Left turn	532	643	4.58	Yes
			Right turn	653	550	4.20	Yes
With adjacent intersections	2020 AM	Arterial	Left turn	583	489	4.08	Yes
			Through	1842	1852	0.23	Yes
			Right turn	543	485	2.58	Yes
		Ramp	Left turn	663	547	4.70	Yes
			Right turn	695	613	3.23	Yes
	2020 Noon	Arterial	Left turn	555	503	2.26	Yes
			Through	1493	1592	2.52	Yes
			Right turn	554	496	2.53	Yes
		Ramp	Left turn	484	475	0.41	Yes
			Right turn	542	554	0.51	Yes
	2020 PM	Arterial	Left turn	815	690	4.54	Yes
			Through	1843	1938	2.20	Yes
			Right turn	741	652	3.37	Yes
		Ramp	Left turn	532	525	0.30	Yes
			Right turn	653	641	0.45	Yes

Table 3.10 GEH statistics for I-25 and Hampden Ave interchange

Scenario		Movement		Mean of each turning movement		GEH statistic	< 5?
				Field volume	Simulated volume		
Without adjacent intersections	2020 AM	Arterial	Left turn	504	410	4.40	Yes
			Through	1592	1441	3.88	Yes
			Right turn	1165	1027	4.18	Yes
		Ramp	Left turn	609	598	0.45	Yes
			Right turn	266	270	0.24	Yes
	2020 Noon	Arterial	Left turn	405	337	3.56	Yes
			Through	1420	1566	3.78	Yes
			Right turn	842	750	3.26	Yes
		Ramp	Left turn	855	870	0.53	Yes
			Right turn	438	448	0.45	Yes
	2020 PM	Arterial	Left turn	357	279	4.38	Yes
			Through	1910	1748	3.79	Yes
			Right turn	851	722	4.62	Yes
		Ramp	Left turn	994	917	2.48	Yes
			Right turn	390	351	2.05	Yes

3.4 Measures of Effectiveness (MOEs)

The study considered travel time and maximum queue length from the VISSIM output as the primary measures for evaluating traffic operation, while the frequency and type of simulated conflicts and the number of vehicle stops were used to assess safety. The safety analysis was performed using SSAM, which estimates the frequency and speed of near-crash events in traffic based on the TTC threshold. Reviewing the previous studies, a 1.5-second TTC (SSAM's default) was found as the most popular threshold to examine the conflicting interactions on SSAM (*66-68*). Therefore, this study also considered a 1.5 seconds of TTC threshold to compare the results among the new Super DDI, DDI, and the conventional diamond interchange. Other measures on SSAM analysis were set as default which includes 5 seconds for post-encroachment time (PET), 30 degrees for rear-end angle, and 80 degrees for crossing angle. For this study, the safety evaluation was based on three categories of measure of effectiveness (MOE): (i) the frequency and type of simulated conflicts; (ii) the maximum speed, average TTC, and PET value of conflicting vehicles to provide an insight of severity; and (iii) the number of vehicle stops indicating driver comfortability.

While analyzing pedestrian performance, pedestrian operation was evaluated in terms of travel time, number of stops, and waiting time. The study also examined the impact of pedestrians on vehicular travel time. Pedestrian safety was analyzed based on a surrogate performance measure called design flags assessment proposed by the new NCHRP guide on pedestrian and bicyclist safety (*50*). These design flags are not only unique to evaluate the performance measures of the alternative designs but also applicable to design safe pedestrian and bicycle facilities for each alternative, whether traditional or AIIs. The analysis includes two types of design flags: (i) red flags, indicating design elements directly related to a safety concern for pedestrians or bicyclists, and (ii) yellow flags, indicating design elements negatively affecting user comfort (i.e., experiencing stress while walking or cycling). Figure 3.5 shows an example of two-directional pedestrian paths from each origin as part of the flag assessment. As mentioned earlier, the study tested only the side paths (north and south in Figure 3.5) in the simulation models. However, all possible pedestrian paths (north, south, east, and west in Figure 3.5) were considered to conduct design flags. There are 20 design flag criteria that apply to either the pedestrian movement, the bicyclist movement, or both. Table 3.11 provides the summary of 13 design flags used for pedestrian assessment with the threshold information of yellow and red flags.

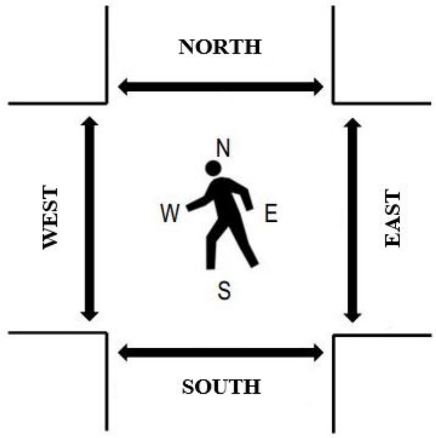


Figure 3.5 Pedestrian route and direction for flag assessment

Table 3.11 Design flags description for pedestrian assessment (*50*)

No	Flag Description	Measure of Effectiveness	Yellow Flag Threshold	Red Flag Threshold
1	Motor vehicle right turns	Vehicle turning speed and volume	<= 20 mph and <= 50 vph	> 20 mph or > 50 vph
2	Uncomfortable/tight walking environment	Walkway width	<5' if traffic present on one side; <10' if traffic present on two sides	N/A
3	Non-intuitive motor vehicle movements	Vehicle acceleration profile	Vehicle decelerating	Vehicle accelerating or free-flowing
4	Crossing yield-controlled or uncontrolled vehicle paths	Vehicle speed and volume	<= 20 mph and <= 50 vph	> 20 mph or > 50 vph
5	Indirect paths	Out of direction travel distance	90'	135'
6	Executing unusual movements	Local expectation	Path does not match expectation	N/A
7	Multilane crossing	Number of lanes	2-3 lanes	> 3 lanes
8	Long red times	Delay	30 seconds	45 seconds
9	Undefined crossings at intersections	Path markings	Unmarked crossing	N/A
10	Motor vehicle left turns	Vehicle turning speed and volume	<= 20 mph and <= 50 vph	> 20 mph or > 50 vph
11	Intersecting driveways and side streets	Number of access points in the area of influence	1-2	> 2
12	Sight distance for gap acceptance movements	Sight distance	N/A	Less than required for vehicle speed
13	Grade change	Percentage of grade	±3-5%	> ±5%

4. RESULTS AND DISCUSSIONS ON OPERATIONAL EFFICIENCY

The following paragraphs demonstrate the major findings from the operational analysis. The main performance measures investigated for evaluating traffic operation include vehicle travel time and maximum queue length.

4.1 Vehicle Travel Time

The first operational performance measure examined was vehicle travel time. The mean travel times of each design at three different locations are shown in Tables 4.1 – 4.3, considering the absence and presence of adjacent intersections. The results also include the percentage of completed tests for each design. Due to interchanges' different capacities, some of the designs could not complete the scenarios with a high traffic demand range. As seen from the results, CDI and DDI could not complete some of the tests with at least 90% of the entry traffic volume because of their lower capacity, especially in high demand conditions of 2030 tests. However, an exception was observed for the scenarios at I-225 and Mississippi interchange without adjacent intersections where CDI and DDI were able to accommodate all entry traffic volumes. This could be because I-225 and Mississippi interchanges had relatively low traffic volumes compared with other locations. The travel times of the incomplete tests were substantially longer than other tests due to the spillback caused by a large portion of vehicles being stopped for a long time. Therefore, those incomplete models were removed from the travel time evaluation to make a fair (with the same traffic demand) comparison. On the other hand, two versions of the Super DDI design were found to complete all the tests, indicating a higher capacity to deal with high demands.

In general, CDI performed worst in all scenarios in terms of travel time performance, while both Super DDI-1 and Super DDI-2 were found as potential competitors to DDI. Among the three peak hours, Noon had the least volume with the lowest travel time, and the PM had the highest volume with the longest travel time. This was fairly consistent throughout the designs investigated at various locations.

The travel time performance at I-225 and Mississippi Ave, as shown in Table 4.1, indicated that all the designs performed similarly with statistically insignificant differences based on ANOVA. However, while considering adjacent signals, both Super DDIs had significantly lower travel times compared with CDI designs. On average, Super DDI-1 was found to reduce travel times by 22% and 12% as compared with the CDI and DDI, respectively, whereas the corresponding reductions were 24% and 14% for Super DDI-2 when compared with the CDI and DDI, respectively. Also, the capacity of both Super DDI designs was found to be higher compared with CDI and DDI designs. This should be attributed to the fact that through traffic could experience a perfect two-way progression system with almost no stops at the interchange. In fact, reviewing simulation animations, it was found that through traffic demand could cross at least three (out of four) of the traffic signals in a green indication. As discussed earlier and shown in Figure 1.4, through traffic demand could cross both the half signals of the Super DDI in green. In addition, since the spacing between two adjacent full signals on all the case study sites is about 0.5 miles (as shown in Figure 1.2), which is an appropriate spacing for providing a good two-way progression, the majority portion of the through traffic could also cross the second full signal of their route in a green interval.

As for the I-25 and 120th Ave location, Super DDIs outperformed the rest of the designs at all scenarios in terms of travel time performance. Considering no adjacent intersections, as shown in Table 4.2, both Super DDI designs were found to reduce travel times by 38% and 21% (on average) compared with the CDI and DDI, respectively, whereas the corresponding reductions were found to be 34% and 11% (on average) for the scenarios considering adjacent intersections. It should be noted that CDI and DDI had a higher rate of completed tests considering no adjacent signals in the network. According to Table 3.1, there is higher traffic demand (about 25%) in the I-25 and 120th Ave interchange compared with the I-225 and Mississippi Ave interchange. This should be the possible reason for the superior performance of the Super DDI in both networks with and without adjacent intersections. Also, based on Table 3.1, traffic demand had a more balanced distribution in all approaches at the I-225 and Mississippi Ave interchange; however, WB and (off-ramp) NB have considerably higher demands compared to EB and SB at the I-25 and 120th interchange. It means that a higher capacity is needed at the eastern traffic signal of the interchange to respond well to the demand of the entire network.

Travel time performance at the I-25 and Hamden Ave network without adjacent intersections showed that DDI had the lowest travel time, attributing 34% and 9% (on average) lower as compared with Super DDI-1 and Super DDI-2, respectively. However, both the Super DDI designs showed a higher rate of completed tests (as an indicator of capacity) than the DDI. The mean travel time difference was also identified to be statistically insignificant between the DDI and Super DDI-2. Based on Table 3.1, right-turn traffic demand is considerably higher at I-25 and Hamden Ave compared with the other interchanges considered in this study. For example, during AM peak hour, an average of over 1,000 vehicles are turning right from the arterial in this location. Therefore, the DDI could provide a big advantage for those vehicles due to its free-flow right-turn routes. Also, as a possible reason for the unsatisfying performance of Super DDI-1 at the I-25 and Hamden Ave, the left-turn demand from the off-ramps averaged over 800 vehicles per hour, while Super DDI-1 had only one lane for such a significant left-turn volume. On the other hand, Super DDI-2 could perform well (and similar to the DDI) due to its dual left-turn lanes.

As a summary of the travel time analysis mentioned in Tables 4.1 – 4.3, both Super DDI designs should perform better than the other designs when considering adjacent signals. DDI would perform apparently similar to or insignificantly better compared with Super DDI designs if its capacity could respond well to the demand and if no adjacent intersections are located in the vicinity. ANOVA analysis also indicated significant improvement of vehicle travel time performance of Super DDI designs for the presence of adjacent intersections and at the locations where CDI and DDI designs cannot perform well due to lack of capacity.

Table 4.1 Mean travel time performance at I-225 and Mississippi Ave

Interchange Type		Travel Time (sec)							Completed Tests (%)
		2020			2030			Average	
		AM	Noon	PM	AM	Noon	PM		
Without Adjacent Intersections	CDI	65	63	65	**94**	**71**	**<u>107</u>**	**<u>78</u>**	100
	DDI	57	54	60	67	<u>59</u>	<u>94</u>	**<u>65</u>**	100
	Super DDI-1	53	51	56	88	73	113	72	100
	Super DDI-2	52	51	56	84	62	99	67	100
With Adjacent Intersections	CDI	164	128	145	211	162	269	180	83
	DDI	145	124	131	**<u>177</u>**	155	223	**<u>159</u>**	67
	Super DDI-1	121	109	116	193	132	173	141	100
	Super DDI-2	120	112	115	183	126	171	138	100

Note: Bold represents the insignificant difference between Super DDI-1 and the other design while underline represents the insignificant difference between Super DDI-2 and the other design, at 0.05 significance level.

Table 4.2 Mean travel time performance at I-25 and 120th Ave

Interchange Type		Travel Time (sec)							Completed Tests (%)
		2020			2030			Average	
		AM	Noon	PM	AM	Noon	PM		
Without Adjacent Intersections	CDI	118	107	138	138	150	149	133	83
	DDI	108	**61**	123	**113**	**93**	123	104	83
	Super DDI-1	90	60	69	103	78	98	83	100
	Super DDI-2	102	59	68	86	69	109	82	100
With Adjacent Intersections	CDI	260	146	226	296	228	321	246	67
	DDI	215	138	209	240	192	291	214	33
	Super DDI-1	159	125	177	195	153	236	174	100
	Super DDI-2	193	128	167	219	141	224	179	100

Note: Bold represents the insignificant difference between Super DDI-1 and the other design while underline represents the insignificant difference between Super DDI-2 and the other design, at 0.05 significance level.

Table 4.3 Mean Travel Time performance at I-25 and Hampden Ave

Interchange Type		Travel Time (sec)							Completed Tests (%)
		2020			2030			Average	
		AM	Noon	PM	AM	Noon	PM		
Without Adjacent Intersections	CDI	114	69	133	153	144	171	131	67
	DDI	49	51	60	64	56	91	62	83
	Super DDI-1	75	74	103	86	92	135	94	100
	Super DDI-2	66	51	67	73	57	94	68	100

Note: Bold represents the insignificant difference between Super DDI-1 and the other design while underline represents the insignificant difference between Super DDI-2 and the other design, at 0.05 significance level.

4.2 Maximum Queue

The second performance measure investigated was the maximum queue produced in each design, as shown in Tables 4.4 – 4.6 for the three locations. Similar to travel time evaluation, the queue length generated from the completed tests was only included in the analysis to make a fair comparison. The queue lengths, shown in Tables 4.4 – 4.6, were provided for each approach (i.e., EB, WB, NB, and SB), including the approach length to identify the potentials for spillback concern. Also, adjacent intersections were not considered in this analysis because the maximum queue would not change significantly for those intersections in different interchange designs. Also, the maximum queue of through traffic movement was negligible in Super DDI designs due to the perfect two-way progression system.

In general, DDI performed fairly well except in some cases where through traffic demand is very high (e.g., AM and PM through traffic at I-25 and 120th Ave). In a comparison between DDI and Super DDI designs, Super DDI experienced shorter queues on the arterial, while it had longer queues on the off-ramps compared with the DDI. The shorter queues on the arterial should be due to the shorter cycle lengths of the Super DDI designs. DDI encounters a relatively high traffic entry in each node, including entire through traffic volume (on EB and WB), both left-turn traffic from the freeway and one left-turn traffic from the arterial, which ultimately resulted in higher cycle lengths with long queues. For instance, the average cycle length for DDI was 116 seconds in this study, which is more than 1.5 times higher compared with Super DDI (76 seconds). On the other hand, DDI mostly resulted in shorter queues on the off-ramps. All three DDI designs had dual left-turn lanes on the off-ramps, while Super DDI-1 had only one left-turn lane on the off-ramps. Moreover, the off-ramp traffic demands could experience a higher ratio of the green interval over cycle length (g/c) because they received a green indication at the same time with one of the through traffic movements on the arterial. However, in Super DDI designs, traffic of the off-ramp would not share a signal phase with any of the through traffic movements on the arterial (note that g/c ratio would be more in signal phases with a higher demand). It should also be mentioned that incomplete tests are excluded in Tables 4.4 – 4.6, while DDI could have a longer queue in those tests.

As a summary of the queue length analysis mentioned in Tables 4.4 – 4.6, Super DDIs performed better in minimizing the potential of spillback on the arterial due to shorter cycle lengths compared with the other designs. On the other hand, DDI could reduce the risk of spillback in locations with short off-ramps, possibly due to the higher g/c ratio of the off-ramp traffic movements compared with Super DDI designs.

Table 4.4 Maximum queue length at I-225 and Mississippi Ave

Scenario	Approach	Approach Length (ft)	Without Adjacent Intersections			
			CDI	DDI	Super DDI-1	Super DDI-2
2020 AM	EB	1000	157	163	121	147
	WB	800	234	256	207	244
	NB	1100	185	121	201	144
	SB	1500	294	330	342	244
2020 Noon	EB	1000	141	190	122	177
	WB	800	121	158	135	137
	NB	1100	179	139	227	157
	SB	1500	267	273	294	204
2020 PM	EB	1000	302	243	192	217
	WB	800	216	213	182	235
	NB	1100	171	144	314	193
	SB	1500	245	246	346	191
2030 AM	EB	1000	339	238	157	473
	WB	800	428	438	383	579
	NB	1100	277	218	520	186
	SB	1500	722	528	636	391
2030 Noon	EB	1000	297	271	199	342
	WB	800	181	235	222	264
	NB	1100	204	186	404	230
	SB	1500	344	331	894	279
2030 PM	EB	1000	624	588	368	467
	WB	800	484	486	355	437
	NB	1100	281	320	796	339
	SB	1500	474	447	1251	305
Overall	EB	1000	310	282	193	304
	WB	800	277	298	247	316
	NB	1100	216	188	410	208
	SB	1500	391	359	627	269

Note: Bold represents the queues that exceeded the approach lengths.

Table 4.5 Maximum queue length at I-25 and 120th Ave

Scenario	Approach	Approach Length (ft)	Without Adjacent Intersections			
			CDI	DDI	Super DDI-1	Super DDI-2
2020 AM	EB	1500	442	253	264	510
	WB	900	856	**1195**	509	734
	NB	1500	531	588	816	815
	SB	1200	190	63	301	263
2020 Noon	EB	1500	591	507	199	372
	WB	900	539	161	194	356
	NB	1500	285	169	403	244
	SB	1200	160	43	196	134
2020 PM	EB	1500	1432	**1658**	520	920
	WB	900	341	232	260	758
	NB	1500	286	169	400	284
	SB	1200	284	132	291	197
2030 AM	EB	1500	746	1073	362	463
	WB	900	**1083**	**1189**	**1116**	**1164**
	NB	1500	746	693	815	815
	SB	1200	216	78	306	306
2030 Noon	EB	1500	1145	1137	303	582
	WB	900	**1062**	350	272	829
	NB	1500	488	291	815	357
	SB	1200	244	130	234	159
2030 PM	EB	1500	1428	**1658**	1117	**1674**
	WB	900	474	359	281	**1116**
	NB	1500	668	263	814	403
	SB	1200	324	171	359	234
Overall	EB	1500	964	1048	461	754
	WB	900	726	581	439	826
	NB	1500	501	362	677	486
	SB	1200	236	103	281	216

Note: Bold represents the queues that exceeded the approach lengths.

Table 4.6 Maximum queue length at I-225 and Hampden Ave

Scenario	Approach	Approach Length (ft)	Without Adjacent Intersections			
			CDI	DDI	Super DDI-1	Super DDI-2
2020 AM	EB	1000	177	208	79	249
	WB	800	437	241	269	**803**
	NB	1000	409	104	984	201
	SB	1500	249	86	247	149
2020 Noon	EB	1000	124	119	81	106
	WB	800	273	142	150	233
	NB	1000	383	113	973	221
	SB	1500	323	125	467	228
2020 PM	EB	1000	342	178	100	203
	WB	800	484	335	250	363
	NB	1000	974	297	**1546**	902
	SB	1500	236	163	984	320
2030 AM	EB	1000	209	304	81	145
	WB	800	422	469	297	**868**
	NB	1000	765	217	984	448
	SB	1500	557	110	330	172
2030 Noon	EB	1000	237	139	77	105
	WB	800	489	290	253	339
	NB	1000	970	140	**1546**	452
	SB	1500	561	162	984	314
2030 PM	EB	1000	268	247	71	99
	WB	800	417	469	364	436
	NB	1000	977	440	**1546**	**1542**
	SB	1500	751	251	984	984
Overall	EB	1000	226	199	81	151
	WB	800	420	324	264	507
	NB	1000	746	219	1263	628
	SB	1500	446	150	666	361

Note: Bold represents the queues that exceeded the approach lengths.

5. RESULTS AND DISCUSSIONS ON SAFETY EVALUATION

The following paragraphs demonstrate the results found from the safety analysis. As mentioned earlier, the main performance measures investigated for assessing traffic safety include frequency and type of vehicular conflicts; maximum speed, average TTC, and PET value of conflicting vehicles; and the average number of vehicle stops.

5.1 The Comparison of Conflicting Interactions

Table 5.1 shows overall results from SSAM, while Table 5.2 and Table 5.3 present the number of conflict results from SSAM broken down by conflict type for each design considering with and without adjacent intersections, respectively. Table 5.4 and Table 5.5 provide comparisons of the proposed designs with CDI and DDI in terms of the number of conflicts for different scenarios and also indicate whether the differences were statistically significant at the 0.05 level based on ANOVA. It is worth mentioning that while running each simulation scenario five times, some of the CDI and DDI models were not able to complete at least 90% of the entry traffic volume because of their limited capacity, especially in high demand conditions of the year 2030 tests. Those incomplete tests were discarded from the analysis, and the results in this paper only represent the completed simulation tests to make a fair comparison (with similar entry traffic volumes).

The frequency of the simulated conflicts can be illustrated as the probability of crashes, while the maximum speed, average TTC, and PET at the moment of conflict can be interpreted as the severity of crashes. Overall, the results indicate superior performance of the Super DDI designs in terms of minimizing the total number of conflicts. The only exception was found for Super DDI-1 at Hampden Ave considering no adjacent intersections. There should be a safety concern for Super DDI-1 at Hampden Ave without adjacent signals. The possible explanation could be because the ratio of turning traffic over through traffic is higher at I-25 and Hamden Ave compared with the other interchanges based on Table 3.1. For example, during AM peak hour, an average of over 1,000 vehicles are turning right from the arterial in this location. Therefore, the DDI could provide a big advantage for those vehicles due to its free-flow right-turn routes. Also, as another possible reason for the unsatisfying performance of Super DDI-1 at I-25 and Hamden Ave, the left-turn demand from the off-ramps was averaging about 1,000 vehicles per hour, while Super DDI-1 had only one lane for such a significant left-turn volume. On the other hand, Super DDI-2 could perform well (and similar to the DDI) due to its dual left-turn lanes. Tables 5.2 and 5.3 highlight the relatively better performance of either Super DDI-1 or Super DDI-2 in reducing the number of conflicts obtained from crossing, rear end, and lane changing events.

Another notable point is that the difference of conflicting interactions between the proposed designs and other designs was found to be higher when there were adjacent signals. This may be because of the fact that through traffic could experience a perfect two-way progression system with no stop at the interchange. In fact, in reviewing simulation animations, it was found that the majority of through traffic demand could cross at least three (out of four) of the traffic signals in green light. As discussed earlier and shown in Figure 1.4, through traffic demand could cross both the half signals of the Super DDI in green. In addition, the spacing between two adjacent full signals on all the case study sites is about 0.5 miles as shown in Figure 1.2. This is an appropriate spacing for providing a good two-way progression, so the majority portion of the through traffic could also cross the second full signal of their route in a green interval, which ultimately resulted in less conflicting interactions. ANOVA analysis, shown in Tables 5.4 and 5.5, also indicated significant improvement of Super DDI designs in reducing traffic conflicts for the presence of adjacent intersections and at the locations where CDI and DDI designs cannot perform well due to lack of capacity.

Regarding the severity of conflicts, shown in Table 5.1, the DDI had the lowest speed of conflicting vehicles although the differences were likely to be negligible to other designs. This could be due to the presence of horizontal curves at the crossover in DDIs, which leads drivers to reduce their speeds. Also, the perfect progression system provided in Super DDI designs should increase vehicle speed. Therefore, the speed should be higher in vehicle-vehicle interactions. Despite having the lowest speeds, the DDI had the lowest values of mean TTC and mean PET among designs considered. On the other hand, the Super DDI-1 seemed to be one of the safest designs from the viewpoint of severity due to its relatively high values of TTC and PET.

Table 5.1 The comparison of overall frequency and severity of simulated conflicts based on SSAM (TTC threshold = 1.5 seconds)

Location		Interchange type	Overall conflicts	Speed (mph)	TTC (sec)	PET (sec)
Without adjacent intersections	I-225 and Mississippi Ave	CDI	1519	8.71	0.77	1.02
		DDI	1378	7.68	0.66	0.77
		Super DDI-1	1413	8.25	0.87	1.07
		Super DDI-2	1371	8.29	0.87	1.01
	I-25 and 120th Ave	CDI	3279	8.15	0.88	1.33
		DDI	3040	6.64	0.80	1.20
		Super DDI-1	2244	6.68	0.99	1.34
		Super DDI-2	2377	7.61	0.96	1.23
	I-25 and Hampden Ave	CDI	2117	7.46	0.76	1.11
		DDI	1803	7.12	0.76	1.04
		Super DDI-1	2643	6.04	0.94	1.47
		Super DDI-2	1296	6.54	0.88	1.14
With adjacent intersections	I-225 and Mississippi Ave	CDI	4429	8.27	0.78	0.94
		DDI	3798	7.11	0.68	0.93
		Super DDI-1	3224	8.07	0.96	1.30
		Super DDI-2	3022	8.15	0.79	0.95
	I-25 and 120th Ave	CDI	9287	6.67	0.84	1.20
		DDI	7739	6.66	0.83	1.19
		Super DDI-1	5498	7.14	0.85	1.30
		Super DDI-2	6543	7.16	0.80	1.19

Note: All the values are indicating the average of total scenarios.

Table 5.2 Type of simulated conflicts considering no adjacent intersections (TTC threshold = 1.5 seconds)

Interchange type			AM peak				Noon peak				PM peak			
			Total	Crossing	Rear end	Lane change	Total	Crossing	Rear end	Lane change	Total	Crossing	Rear end	Lane change
2020 traffic volume	I-225 and Mississippi Ave	CDI	898	2	668	228	776	3	576	196	1188	2	890	296
		DDI	762	1	537	224	651	0	512	139	841	0	623	218
		Super DDI-1	678	0	514	164	649	0	528	121	640	0	462	178
		Super DDI-2	720	0	520	200	678	0	534	144	876	0	647	229
	I-25 and 120th Ave	CDI	2239	0	1709	529	1381	1	1095	286	3543	1	2930	612
		DDI	2582	2	1974	605	864	2	625	236	3842	2	3181	660
		Super DDI-1	2093	0	1778	315	584	0	483	101	1215	0	1028	187
		Super DDI-2	2139	3	1729	406	697	0	539	158	921	1	726	194
	I-25 and Hampden Ave	CDI	1772	3	1285	484	996	0	761	235	2462	8	1752	702
		DDI	1036	1	783	251	1096	2	795	299	1318	4	918	396
		Super DDI-1	1629	3	1275	351	1832	1	1495	336	3341	1	2826	515
		Super DDI-2	1747	4	1403	340	555	1	406	147	838	0	632	206
2030 traffic volume	I-225 and Mississippi Ave	CDI	1591	2	1137	452	1302	0	998	304	3359	5	2384	970
		DDI	2057	4	1427	626	917	1	663	254	3041	10	2186	845
		Super DDI-1	2532	11	1755	767	1162	0	898	264	2816	5	2078	733
		Super DDI-2	2049	7	1470	572	903	0	654	249	2998	8	2117	873
	I-25 and 120th Ave	CDI	3794	0	2917	877	3965	0	3256	709	4749	2	3855	893
		DDI	4214	3	3184	1027	2335	4	1809	522	4402	3	3639	761
		Super DDI-1	3787	1	3204	583	1493	0	1244	249	4289	3	3424	862
		Super DDI-2	2989	1	2384	603	1233	1	989	243	6280	2	5102	1176
	I-25 and Hampden Ave	CDI	2116	6	1534	577	2762	13	1958	791	2592	9	1856	727
		DDI	2618	5	1873	740	1583	4	1132	448	3168	8	2200	960
		Super DDI-1	2179	3	1762	414	3005	2	2496	508	3872	8	3071	793
		Super DDI-2	2221	5	1743	473	667	2	475	190	1749	2	1405	342

Table 5.3 Type of simulated conflicts considering adjacent intersections (TTC threshold = 1.5 seconds)

Interchange type			AM peak				Noon peak				PM peak			
			Total	Crossing	Rear end	Lane change	Total	Crossing	Rear end	Lane change	Total	Crossing	Rear end	Lane change
2020 traffic volume	I-225 and Mississippi Ave	CDI	3826	2	2916	908	1955	2	1406	547	3082	2	2236	844
		DDI	2244	3	1752	489	2063	3	1642	418	2889	3	2320	565
		Super DDI-1	1817	1	1348	468	1606	2	1177	427	2309	2	1715	592
		Super DDI-2	1542	3	1140	399	1655	1	1239	415	2141	3	1578	561
	I-25 and 120th Ave	CDI	8858	10	7186	1662	2312	2	1716	594	8821	7	6866	1949
		DDI	5571	3	4288	1280	2223	1	1613	609	9032	6	7096	1930
		Super DDI-1	3127	1	2453	673	1334	2	1009	323	6310	6	4921	1383
		Super DDI-2	5742	2	4583	1157	1532	1	1144	388	5876	6	4541	1329
2030 traffic volume	I-225 and Mississippi Ave	CDI	5198	3	4050	1145	4127	2	3105	1019	8387	16	6578	1792
		DDI	4114	4	3411	699	4756	8	4028	720	6720	26	5762	933
		Super DDI-1	5115	7	4034	1074	3133	2	2293	839	5361	2	4113	1245
		Super DDI-2	4637	12	3565	1061	2740	1	1995	744	5415	5	4035	1375
	I-25 and 120th Ave	CDI	12617	17	10189	2412	7186	6	5524	1656	15926	20	12703	3203
		DDI	9359	7	7329	2023	4514	1	3305	1208	15736	18	12494	3224
		Super DDI-1	6571	8	5209	1354	3435	2	2625	808	12213	18	9738	2457
		Super DDI-2	10890	12	8882	1996	2706	2	2016	688	12509	13	10051	2445

Table 5.4 ANOVA with post hoc tests for simulated conflicts without adjacent intersections

Interchange type		Compares with	Mean difference (number of conflicts)						
			Overall	2020			2030		
				AM	Noon	PM	AM	Noon	PM
I-225 and Mississippi Ave	Super DDI-1	CDI	**-106**	-220	-127	-549	941	**-140**	**-543**
		DDI	**35**	-84	**-2**	-202	**475**	245	**-225**
	Super DDI-2	CDI	**-148**	-178	-98	-312	**458**	-399	**-361**
		DDI	**-8**	**-42**	**26**	**35**	**-7**	**-14**	**-43**
I-25 and 120th Ave	Super DDI-1	CDI	-1035	**-146**	-797	-2327	**-7**	-2472	**-460**
		DDI	**-796**	-489	**-280**	-2627	**-427**	**-842**	**-113**
	Super DDI-2	CDI	**-902**	**-100**	-684	-2622	**-806**	-2732	1531
		DDI	**-663**	-443	**-167**	-2921	-1226	**-1102**	1878
I-25 and Hampden Ave	Super DDI-1	CDI	**526**	**-143**	835	879	**63**	**243**	1281
		DDI	840	**594**	735	2023	**-439**	1422	704
	Super DDI-2	CDI	-821	**-25**	-442	-1624	**105**	-2096	-842
		DDI	**-507**	**712**	-542	-480	**-397**	-917	-1419

Note: Bold represents the insignificant differences at 0.05 level.

Table 5.5 ANOVA with post hoc tests for simulated conflicts with adjacent intersections

Interchange type		Compares with	Mean difference (number of conflicts)						
			Overall	2020			2030		
				AM	Noon	PM	AM	Noon	PM
I-225 and Mississippi Ave	Super DDI-1	CDI	-1205	-2009	-349	-773	**-82**	-994	-3026
		DDI	**-574**	**-427**	-458	-580	1002	-1623	-1360
	Super DDI-2	CDI	-1407	-2284	-300	-941	-560	-1387	-2971
		DDI	**-776**	-703	-408	-747	**524**	-2016	-1305
I-25 and 120th Ave	Super DDI-1	CDI	-3788	-5731	-978	-2512	-6045	-3751	-3713
		DDI	**-2241**	-2443	-889	-2722	-2788	**-1079**	-3524
	Super DDI-2	CDI	-2744	-3116	-779	-2945	-1727	-4480	-3417
		DDI	**-1197**	**171**	-691	-3155	1531	-1808	-3228

Note: Bold represents the insignificant differences at 0.05 level.

5.2 The Comparison of Number of Vehicle Stops

In the last step of the analysis, the average number of vehicle stops was extracted from VISSIM as another indicator of safety. The number of stops should be considered to affect driver comfort. For instance, when a driver experiences a high number of stops, it is possible to get frustrated and commit more driving violations. Tables 5.6 and 5.7 show the average number of stops for each design considering without and with adjacent signals, respectively. Based on the results considering no adjacent intersections shown in Table 5.6, DDI had apparently a similar or insignificantly lower (on average) number of stops compared with Super DDI designs. However, in the case of adjacent signals shown in Table 5.7, both Super DDIs, on average, outperformed the other designs in terms of minimizing the number of stops. Although the differences with DDI were not significant (on average), they were found significant as compared with CDI when there were adjacent signals.

Table 5.6 The average number of stops considering no adjacent intersections

Interchange type		Stops (no)						
		Overall	2020			2030		
			AM	Noon	PM	AM	Noon	PM
I-225 and Mississippi Ave	CDI	**<u>0.55</u>**	0.46	0.44	<u>0.46</u>	<u>0.67</u>	<u>0.52</u>	**<u>0.75</u>**
	DDI	**<u>0.53</u>**	0.43	0.40	0.47	<u>0.58</u>	<u>0.48</u>	**<u>0.85</u>**
	Super DDI-1	0.58	0.40	0.42	0.39	0.82	0.62	0.82
	Super DDI-2	0.52	0.40	0.42	0.46	0.59	0.50	0.74
I-25 and 120th Ave	CDI	**0.86**	0.77	0.83	0.77	0.96	0.89	0.95
	DDI	**<u>0.70</u>**	0.76	**<u>0.43</u>**	0.74	0.88	**<u>0.67</u>**	0.74
	Super DDI-1	0.76	0.89	0.43	0.61	1.10	0.67	0.87
	Super DDI-2	0.70	1.08	0.41	0.54	0.75	0.55	0.90
I-25 and Hampden Ave	CDI	1.09	0.84	**0.57**	1.40	1.01	1.34	1.40
	DDI	<u>0.49</u>	0.39	0.40	<u>0.44</u>	**0.62**	<u>0.45</u>	<u>0.66</u>
	Super DDI-1	0.70	0.58	0.52	0.61	0.65	0.59	0.99
	Super DDI-2	0.54	0.63	0.34	0.46	0.72	0.42	0.64

Note: Bold represents the insignificant difference between Super DDI-1 and the other design, while underline represents the insignificant difference between Super DDI-2 and the other design, at 0.05 significance level.

Table 5.7 The average number of stops considering adjacent intersections

Interchange type		Stops (no)						
		Overall	2020			2030		
			AM	Noon	PM	AM	Noon	PM
I-225 and Mississippi Ave	CDI	0.89	0.91	**<u>0.58</u>**	**0.67**	**<u>0.97</u>**	0.77	1.45
	DDI	**<u>0.80</u>**	**<u>0.68</u>**	0.65	**0.67**	0.82	0.80	**<u>1.16</u>**
	Super DDI-1	0.70	0.55	0.57	0.64	1.01	0.68	0.79
	Super DDI-2	0.71	0.56	0.59	0.62	0.95	0.65	0.91
I-25 and 120th Ave	CDI	1.27	1.41	<u>0.67</u>	1.01	1.72	1.19	1.64
	DDI	**<u>1.00</u>**	**<u>0.94</u>**	0.63	**0.94**	**0.95**	**<u>0.85</u>**	1.68
	Super DDI-1	0.90	0.89	0.59	0.85	1.04	0.83	1.23
	Super DDI-2	0.92	0.97	0.68	0.76	1.30	0.70	1.14

Note: Bold represents the insignificant difference between Super DDI-1 and the other design, while underline represents the insignificant difference between Super DDI-2 and the other design, at 0.05 significance level.

6. RESULTS AND DISCUSSIONS ON PEDESTRIAN PERFORMANCE

The following paragraphs summarize the performance of pedestrians in two versions of Super DDI in comparison with existing CDI and DDI designs. The analysis also demonstrates the impact of pedestrians on traffic operations. As the last part of the evaluation in this research, pedestrian safety has been analyzed based on the new design flags method.

6.1 Pedestrian Travel Time, Number of Vehicle Stops, and Waiting Time

The overall pedestrian performance in each design is provided in Table 6.1, while Table 6.2 presents the pairwise comparisons of the performance measures and also indicates whether the mean differences were statistically significant at the 0.05 level based on ANOVA. Pedestrian travel time and the average number of stops were obtained from VISSIM. The number of stops should be considered as one of the effective variables to examine pedestrian safety. Pedestrians tend to commit more violations as the number of stops increases. To elaborate on this matter, the waiting time was estimated by multiplying the number of stops by half of the red interval (shown in Table 3.4) for the pedestrians. Note that the number of stops determined from the simulation output was due to red lights since pedestrians had the right-of-way for crossing at any other conflict point with vehicles. Therefore, this parameter is used to identify the probability of facing a red interval in this study. The purpose of applying half red interval was to consider an average stop length for the pedestrians assuming random arrivals (*5, 21*). For example, the waiting time was estimated to be equal to 8 seconds for CDI at I-225 and Mississippi Ave, multiplying 0.40 (the number of stops) by 18.75 (half of the average red interval = 0.5 × $\frac{28\times47}{2}$).

Based on the results shown in Table 6.1, on average, the CDI appeared to be the best design in terms of travel time and the number of stops by a close margin over the Super DDI, while both Super DDI designs outperformed the other designs in minimizing pedestrian waiting times, indicating being less prone to jaywalking or violation related activities. On the other hand, DDI had the worst performance in terms of all MOEs.

Regarding pedestrian travel time shown in Table 6.1, the CDI was found to perform best, providing faster routes for pedestrians. The reason for the higher travel time performance of the conventional diamond design is the existence of only one signalized crossing for each route in the geometry, while the other crossing is a free-flow with the right-of-way for pedestrians based on the existing design. Moreover, CDI pedestrians experienced protected green lights simultaneously with the green lights of off-ramps (since no through traffic was designated on the off-ramps). Based on the ANOVA analysis shown in Table 6.2, the pedestrian performance was significantly better in CDI compared with other designs in terms of travel time and number of stops except for the stop evaluation at I-25 and Hampden Ave. As a possible reason for the greater number of CDI stops at the I-25 and Hamden Ave, the signal cycle length (shown in Table 3.4) is considerably longer compared with the other interchanges; also, pedestrians would experience lower ratios of the green interval over cycle length (g/c) due to the presence of relatively high turning traffic from the off-ramp. For example, the NB left-turn demand from the off-ramp was over 1,000 vehicles per hour at this location based on Table 3.1. Although the simulation outcomes show a relatively good CDI performance, the results could be different in the real-world scenario. For instance, vehicles often do not yield to pedestrians on the free-flowing entrance ramps regardless of having pedestrian right-of-way in that situation. Also, there is always a possibility of limited through traffic on the off-ramps, resulting in conflict with pedestrians.

Compared with DDI, both Super DDI designs performed significantly better based on the ANOVA results shown in Table 6.2. The only exception was found in stop evaluation at I-25 and Hampden Ave, where the mean differences were considered insignificant. After reviewing the results, it can be concluded that Super DDI is a more promising design in improving pedestrian performance than DDI. The possible reasons behind the relatively worse performance of DDI include longer pedestrian paths, more stops (due to facing more traffic signals), higher clearance time due to crossing longer crosswalks (especially in crossing the through traffic in the crossovers), and lower g/c ratios.

Table 6.1 Average pedestrian performance for each interchange designs based on VISSIM

Interchange Type	I-225 and Mississippi Ave			I-25 and 120th Ave			I-25 and Hampden Ave		
	Travel Time (sec)	Stops (no)	Waiting Time (sec)	Travel Time (sec)	Stops (no)	Waiting Time (sec)	Travel Time (sec)	Stops (no)	Waiting Time (sec)
CDI	124	0.40	8	140	0.42	10	145	0.86	19
DDI	145	0.54	11	200	0.62	24	180	0.57	19
Super DDI-1	128	0.50	7	150	0.48	8	155	0.54	9
Super DDI-2	130	0.48	7	150	0.49	9	151	0.54	8

Note: All the values are indicating the average of total scenarios.

Table 6.2 ANOVA with post hoc tests for pedestrian performance per interchange design

Interchange Type	Compares With	Mean Difference								
		I-225 and Mississippi Ave			I-25 and 120th Ave			I-25 and Hampden Ave		
		Travel Time (sec)	Stops (no)	Waiting Time (sec)	Travel Time (sec)	Stops (no)	Waiting Time (sec)	Travel Time (sec)	Stops (no)	Waiting Time (sec)
CDI	DDI	-20.78	-0.14	-3.51	-59.48	-0.20	-14.59	-35.09	0.29	**0.12**
	Super DDI-1	-4.23	-0.10	**0.73**	-9.13	-0.06	1.48	-10.10	0.32	10.48
	Super DDI-2	-5.85	-0.08	**0.76**	-9.55	-0.07	1.28	-5.35	0.32	11.01
DDI	CDI	20.78	0.14	3.51	59.48	0.20	14.59	35.09	-0.29	**-0.12**
	Super DDI-1	16.55	0.04	4.24	50.34	0.14	16.07	24.99	**0.03**	10.36
	Super DDI-2	14.93	0.06	4.27	49.93	0.13	15.87	29.74	**0.03**	10.89
Super DDI-1	CDI	4.23	0.10	-0.73	9.13	0.06	-1.48	10.10	-0.32	-10.48
	DDI	-16.55	-0.04	-4.24	-50.34	-0.14	-16.07	-24.99	**-0.03**	-10.36
	Super DDI-2	**-1.62**	**0.02**	**0.03**	**-0.42**	**-0.01**	**-0.21**	4.75	**0.00**	**0.53**
Super DDI-2	CDI	5.85	0.08	-0.76	9.55	0.07	-1.28	5.35	-0.32	-11.01
	DDI	-14.93	-0.06	-4.27	-49.93	-0.13	-15.87	-29.74	**-0.03**	-10.89
	Super DDI-1	**1.62**	**-0.02**	**-0.03**	**0.42**	**0.01**	**0.21**	-4.75	**0.00**	**-0.53**

Note: Bold represents the insignificant differences at 0.05 level.

6.2 Pedestrian Conflicts

Based on the previous studies (*41, 42, 47*), the type, frequency, and size (length) of conflict points with vehicles have notable impacts on pedestrian safety. The volume of conflicting traffic is another contributing parameter to pedestrian safety. The conflicts between pedestrians and vehicles are demonstrated in Table 6.3 for the designs tested. Note that only one pedestrian path (i.e., southwest to southeast) was considered to estimate the conflicting traffic volume using Table 3.1. The results indicate that DDI had the highest number of crossing lanes with highest conflicting volume. This is because DDI has the through arterial lanes which cross and re-cross each other, resulting in more and longer conflicting points with a significantly higher total of conflicting volume experienced by pedestrians. Table 6.3 also shows that the Super DDI eliminated all the free-flow crossings and reduced the number of crossing lanes by 40% as compared with DDI. On the other hand, the conflicting traffic volume in Super DDI was found to be reduced by approximately 70%, 50%, and 55% at the three locations, respectively, when compared with DDI.

Table 6.3 The comparison of vehicle-pedestrian conflicts per each design for the selected locations

Location	Route	Design	Free-Flow Crossing			Permissive Crossing			Protected Crossing			Total Crossing		
			N[a]	L[b]	V[c]	N	L	V	N	L	V	N	L	V
I-225 and Mississippi Ave	Southwest to Southeast (one-way)	CDI	0	0	0	2	2	894	2	5	843	4	7	1737
		DDI	2	2	894	0	0	0	2	8	4441	4	10	5335
		Super DDI-1	0	0	0	1	1	453	3	5	1284	4	6	1737
		Super DDI-2	0	0	0	1	1	453	3	5	1284	4	6	1737
I-25 and 120th Ave	Southwest to Southeast (one-way)	CDI	0	0	0	2	2	1877	2	5	1556	4	7	3433
		DDI	2	2	1877	0	0	0	2	8	5030	4	10	6907
		Super DDI-1	0	0	0	1	1	1097	3	5	2336	4	6	3433
		Super DDI-2	0	0	0	1	1	1097	3	5	2336	4	6	3433
I-25 and Hampden Ave	Southwest to Southeast (one-way)	CDI	0	0	0	2	2	1629	2	5	1692	4	7	3321
		DDI	2	2	1629	0	0	0	2	8	5780	4	10	7409
		Super DDI-1	0	0	0	1	1	1023	3	5	2298	4	6	3321
		Super DDI-2	0	0	0	1	1	1023	3	5	2298	4	6	3321

Note:
[a] Number of crossings
[b] Number of lanes crossed
[c] Conflicting traffic volume (veh/hr) calculated using Table 3.1

6.3 Impact of Pedestrians on Traffic Operation

The impact of pedestrians on vehicle travel time was analyzed and presented in Table 6.4. Note that the vehicle travel time was extracted from VISSIM for runs with 360 pedestrians per hour for four routes (each route included 90 pedestrians per hour) and for runs without pedestrians. According to an ANOVA conducted on the overall mean differences, both CDI and DDI were found to have significant differences at the 0.05 level between their results with and without pedestrians at the specified locations. On the other hand, no significant influence of pedestrians on vehicular travel time (overall) was found in the Super DDI designs, indicating lower impacts compared with CDI and DDI. Note that pedestrians had the right-of-way in all vehicle-pedestrian conflicts in all the networks.

Table 6.4 Vehicle travel time in various traffic conditions and pedestrian presence

Location	Interchange Type	2020 PM			2030 PM			Overall		
		With Ped	No Ped	Mean Diff	With Ped	No Ped	Mean Diff	With Ped	No Ped	Mean Diff
I-225 and Mississippi Ave	CDI	67	65	1.71	111	107	3.46	89	86	2.58
	DDI	61	60	0.57	99	94	4.75	80	77	2.66
	Super DDI-1	59	56	3.27	120	113	6.71	89	84	4.99
	Super DDI-2	57	56	1.10	100	98	1.44	79	77	1.27
I-25 and 120th Ave	CDI	146	138	8.33	156	149	6.32	151	144	7.32
	DDI	134	123	10.52	129	123	6.23	132	123	8.37
	Super DDI-1	73	69	3.77	104	98	5.50	88	84	4.64
	Super DDI-2	71	68	3.13	110	109	0.35	90	89	1.74
I-25 and Hampden Ave	CDI	140	133	7.40	188	170	17.16	164	151	12.28
	DDI	63	61	2.04	99	96	2.78	81	79	2.41
	Super DDI-1	127	124	3.26	142	140	2.31	135	132	2.78
	Super DDI-2	68	67	1.08	104	94	9.47	86	81	5.28

Note: Underline represents the significant differences at 0.05 level.

6.4 Design Flags Assessment for Pedestrians

As the last part of the pedestrian evaluation, design flags were assessed for the four possible pedestrian crossing movements (shown in Figure 3.5). According to the NCHRP-948 guideline (*50*), 13 out of 20 flags were investigated for pedestrian safety, which summed up a total of 52 potential flags (13 flags multiplied by four pedestrian flows) for each design. Tables 6.5 – 6.7 present the design flag computation procedure per each design alternative for the three specified locations, respectively, while Figure 6.1 summarizes all flags, including potential flag severity (yellow vs. red flag). The analysis indicates that DDI resulted in the highest percentage flagged ranging from 50% to 54%, whereas CDI and Super DDI had fewer design flags compared with DDI ranging from 38% to 40%. Super DDI was found to be the best alternative design for having the lowest number of red flags (16), which is about 10% and 20% lower than that of the CDI (18 red flags) and DDI (20 red flags), respectively. Although CDI had the lowest yellow flags (4% to 6%), Super DDI design outperformed conventional diamond because of the reduction in flag severity. Table 6.8 provides some potential mitigation strategies to address the design flags found for this study based on guidance from the NCHRP report (*50*).

Table 6.5 Design flag assessment for I-225 and Mississippi Ave

No.	Flag Description	CDI				DDI				Super DDI-1				Super DDI-2			
		E	W	N	S	E	W	N	S	E	W	N	S	E	W	N	S
1	Motor vehicle right turn	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R
2	Uncomfortable/tight walking environment																
3	Non-intuitive motor vehicle movement	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R
4	Crossing yield or uncontrolled vehicle paths	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R
5	Indirect paths					Y	Y		Y								
6	Executing unusual movements					Y	Y	Y	Y								
7	Multilane crossing	R	R	Y	Y	R	R	R	R	Y	Y	Y	Y	Y	Y	Y	Y
8	Long red times																
9	Undefined crossing at intersections																
10	Motor vehicle left turn	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R
11	Intersecting driveways and side streets																
12	Sight distance for gap acceptance																
13	Grade change																
Total Possible Flags		52				52				52				52			
Total Yellow Flags		2				7				4				4			
Total Red Flags		18				20				16				16			
Percentage of Yellow Flags		4%				13%				8%				8%			
Percentage of Red Flags		35%				38%				31%				31%			
Percentage Flagged		38%				52%				38%				38%			

Note: E = East, W = West, N = North, S = South, R = Red Flag, and Y = Yellow Flag.

Table 6.6 Design flag assessment for I-25 and 120th Ave

No.	Flag Description	CDI				DDI				Super DDI-1				Super DDI-2			
		E	W	N	S	E	W	N	S	E	W	N	S	E	W	N	S
1	Motor vehicle right turn	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R
2	Uncomfortable/tight walking environment																
3	Non-intuitive motor vehicle movement	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R
4	Crossing yield or uncontrolled vehicle paths	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R
5	Indirect paths					Y	Y										
6	Executing unusual movements					Y	Y	Y	Y								
7	Multilane crossing	R	R	Y	Y	R	R	R	R	Y	Y	Y	Y	Y	Y	Y	Y
8	Long red times																
9	Undefined crossing at intersections																
10	Motor vehicle left turn	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R
11	Intersecting driveways and side streets																
12	Sight distance for gap acceptance																
13	Grade change																
Total Possible Flags		52				52				52				52			
Total Yellow Flags		2				6				4				4			
Total Red Flags		18				20				16				16			
Percentage of Yellow Flags		4%				12%				8%				8%			
Percentage of Red Flags		35%				38%				31%				31%			
Percentage Flagged		38%				50%				38%				38%			

Note: E = East, W = West, N = North, S = South, R = Red Flag, and Y = Yellow Flag.

Table 6.7 Design flag assessment for I-25 and Hampden Ave

No.	Flag Description	CDI				DDI				Super DDI-1				Super DDI-2			
		E	W	N	S	E	W	N	S	E	W	N	S	E	W	N	S
1	Motor vehicle right turn	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R
2	Uncomfortable/tight walking environment																
3	Non-intuitive motor vehicle movement	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R
4	Crossing yield or uncontrolled vehicle paths	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R
5	Indirect paths					Y	Y										
6	Executing unusual movements					Y	Y	Y	Y								
7	Multilane crossing	R	R	Y	Y	R	R	R	R	Y	Y	Y	Y	Y	Y	Y	Y
8	Long red times	Y					Y		Y								
9	Undefined crossing at intersections																
10	Motor vehicle left turn	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R	R
11	Intersecting driveways and side streets																
12	Sight distance for gap acceptance																
13	Grade change																
Total Possible Flags		52				52				52				52			
Total Yellow Flags		3				8				4				4			
Total Red Flags		18				20				16				16			
Percentage of Yellow Flags		6%				15%				8%				8%			
Percentage of Red Flags		35%				38%				31%				31%			
Percentage Flagged		40%				54%				38%				38%			

Note: E = East, W = West, N = North, S = South, R = Red Flag, and Y = Yellow Flag.

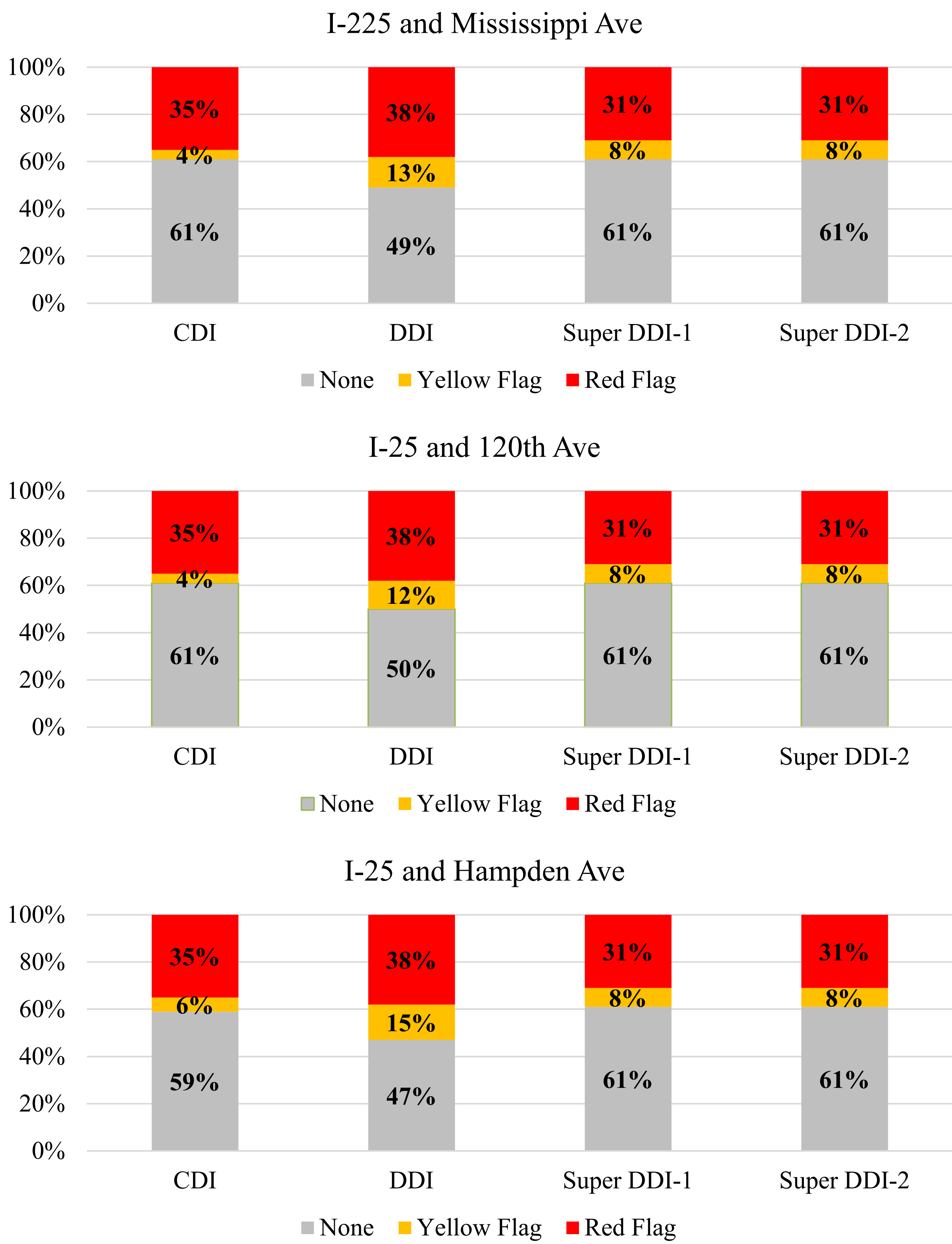


Figure 6.1 Summary of design flags for pedestrian assessment

Table 6.8 Mitigation strategies accounting for the design flags (*50*)

No.	Flag Description	Flag Treatments
1	Motor vehicle right turn	• Assigning a stop bar before the marked pedestrian crossing with adequate sight distance • Providing space for queue storage in-between the crossing and conflicting traffic flow • Limiting right-turns-on-red
3	Non-intuitive motor vehicle movement	• Providing adequate signage and marking that is viewable and understandable to all intended users • Including speech messages or audible information devices for the needs to assist disabled pedestrians
4	Crossing yield or uncontrolled vehicle paths	• Assigning signalized/stop-controlled crossings • Controlling vehicle speeds through curvatures and raised crosswalks
5	Indirect paths	• Providing grade-separated pedestrian facility depending on the context and the origin-destination patterns
6	Executing unusual movements	• Providing a dedicated pedestrian path and re-aligning the movement to make it more intuitive
7	Multilane crossing	• Designing two-stage signalized crossings to reduce the number of lanes crossed at one time • Installing raised crosswalks to control the vehicle speed
8	Long red times	• Modifying the overall cycle length to reduce the total crossing time
10	Motor vehicle left turn	• Transforming permissive left-turns into protected left turns with a dedicated signal phase • Ensuring queue storage for at least one vehicle between the pedestrian crossing and the end of the channelize turn lane to separate motorist decision points

7. CONCLUSIONS AND RECOMMENDATIONS

This study aims to evaluate the performance of two versions of the new Super DDI design in terms of traffic operation, safety, and pedestrian performance using field data. As part of a comprehensive research effort on improving the performance of failing service interchanges in the mountain-plains region, the research team identified three interchanges, Interstate 225 and Mississippi Ave, Interstate 25 and 120th Ave, and Interstate 25 and Hampden Ave, at Denver, Colorado, as the potential candidates to model for future retrofit. Four interchange designs (i.e., existing CDI, DDI, Super DDI-1, and Super DDI-2) were tested in this study. The analysis was conducted through the combination of VISSIM, Synchro, and SSAM analyzing tools. Several microsimulation models were created with three peak hours (AM, Noon, and PM) for the existing year (2020) and projected year (2030) traffic volumes. In addition, the study tested two simulation networks: (i) when no adjacent signal exists to determine how the four interchange designs would perform if there are no adjacent signals or they are far from the interchange, and (ii) when there are two adjacent signals to evaluate the performance of the four interchanges in a longer corridor with signal coordination needed. All the simulation scenarios were calibrated and validated until the satisfactory GEH statistics were met. The study considered travel time and maximum queue length as the primary measures for evaluating traffic operation. The performance measures investigated for assessing traffic safety include frequency and type of vehicular conflicts; maximum speed, average TTC, and PET value of conflicting vehicles; and the average number of vehicle stops. Pedestrian operation was evaluated in terms of travel time, number of stops, and waiting time while pedestrian safety was analyzed based on a surrogate performance measure called design flag, introduced by the new National Cooperative Highway Research Program (NCHRP-948) guideline (*50*). The study also examined the impact of pedestrians on vehicular operation.

Overall, Super DDI designs showed high potential in improving traffic operation based on the simulation tests considered in this study. While investigating travel time performance considering adjacent signals, Super DDI-1 (shown in Figure 6) had 28% and 12% (on average) lower travel times compared with the CDI and DDI, respectively; whereas Super DDI-2 (shown in Figure 7) was found to reduce travel times by 29% and 13% as compared with the CDI and DDI, respectively. In addition, both Super DDI designs showed a higher capacity compared with the other designs considered. On the other hand, DDI performed similarly or insignificantly better than Super DDI if no adjacent intersections were located in the vicinity and if the demand was lower than DDI's capacity. Therefore, it might be concluded that Super DDI designs should perform better in urban (or populated suburban) areas due to the higher traffic demand and a greater number of adjacent intersections compared with rural areas. The possible reasons behind the notable improvement in Super DDI traffic operation are attributed to the fact that the design experiences a lower demand on each traffic signal, and the signals could be coordinated to provide a perfect two-way progression system on the arterial.

As for the maximum queue length performance, Super DDI designs outperformed (on average) the other designs in terms of minimizing queue lengths on the arterials. However, shorter queues could be expected in the DDI than Super DDI designs on the off-ramps because vehicles coming from the freeway could receive a higher ratio of the green interval over cycle length (g/c) facing traffic signals. Therefore, the long queues on the off-ramps should be the main drawback of Super DDI designs based on traffic demands considered in this study. As a possible solution, Super DDIs could be constructed considering longer off-ramps. Also, dual left-turn lanes are essential at locations with a high demand coming from the freeways.

The safety analysis results indicate superior performance of the Super DDI designs in terms of minimizing the total number of simulated conflicts. Regarding the severity of conflicts, the DDI could be more vulnerable due to its lower values of TTC and PET. On the other hand, Super DDI-1 seemed to be one of the safest designs from the viewpoint of crash severity due to its relatively high values of TTC and

PET. As for the number of vehicle stops, DDI had an apparently similar or insignificantly lower (on average) number of stops compared with Super DDI designs when there were no adjacent intersections. However, in the case of adjacent signals, both Super DDIs, on average, outperformed the other designs in terms of minimizing the number of stops. As an important finding from the safety analysis, Super DDI designs outperformed DDI when considering adjacent signals, while DDI performed apparently similar or sometimes even insignificantly better compared with Super DDI if no adjacent intersections were located in the vicinity and if the demand was lower than DDI's capacity.

Based on the simulation outcomes of pedestrian analysis, the conventional diamond showed the best pedestrian operation in terms of travel time and the number of stops by a relatively small margin over the Super DDI. On the other hand, both Super DDI designs outperformed the other designs in minimizing pedestrian waiting times, indicating less tendency to jaywalking or violation-related activities. DDI had the worst performance in terms of all MOEs due to its longer pedestrian paths with higher clearance time of the crosswalk and lower g/c ratios. While analyzing vehicle-pedestrian conflicts, Super DDI appeared to offer relatively good pedestrian safety compared with other designs by eliminating all the free-flow crossings and reducing the number of crossing lanes and the conflicting traffic volume. The investigation of pedestrian impacts on vehicular travel time indicated that both CDI and DDI had significant differences between the vehicle travel times with and without pedestrians at the specified locations. In contrast, no significant influence of pedestrians on vehicular travel time was found in the Super DDI designs. From the assessment of design flag analysis, the Super DDI design is predicted to be safer for pedestrians compared with other designs due to its lowest number of red flags and the potential reduction in flag severity.

Future recommendations are for more studies using a driving simulator laboratory to evaluate drivers' behavior and driver expectation and reaction to pedestrians in Super DDI. Note that the wrong-way movement is one of the potential risks that might result in severe crashes in unconventional designs. This study considered a crossover angle of 45 degrees as recommended in the DDI guideline (*58*). However, future research should focus on estimating an appropriate angle at the Super DDI crossover. A crossover angle less than 45 degrees could be identified as safe since only the left-turn movements are redirected in the Super DDI (while through traffic movements are also redirected in a DDI design). Investigating the notable differences in signage and pavement markings between different alternative interchange designs could also be an interesting topic as a follow-up research study. Adaptive signal timing control can effectively improve traffic efficiency (*69-72*), and it is the trend of signal control in the future. Other potential topics related to this study could include developing safety models, conducting a cost-benefit analysis in locations with smaller bridge sizes, and incorporating a connected and autonomous vehicle (CAV) application into the simulation models. Findings from this study are expected to help transportation managers and policymakers take necessary actions and decide on management strategies for implementing appropriate alternative interchanges.

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