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Conventional Diamond, Diverging Diamond, and Single Point Diamond Interchanges: A Comparative Operational Performance Evaluation in the Era of Connected Vehicles

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A B S T R A C T

This study evaluates the mobility impact of implementing Connected Vehicle (CV) technology at the Conventional Diamond Interchange (CDI), Diverging Diamond Interchange (DDI), and Single Point Diamond Interchange (SPDI). The evaluation is based on a microsimulation environment created in VISSIM combined with ASC/3 Software-in-the-Loop signal controllers and Python-programmed Vehicle to Infrastructure (V2I) communication algorithms. The effect of varying CV-Market Penetration Rates (CV-MPRs) on traffic operations is taken into consideration. The study shows that the interchange design has a higher impact on traffic operations than does the CV-MPR. Particularly, a 100% CV-MPR has led to 6.4% reductions in delays compared to the 0% CV-MPR, without considering the effect of interchange design. Contrarily, the CDI would increase delays as high as 24.0% as opposed to the SPDI, without considering the effect of CVs. Similarly, the DDI would reduce delays by up to 60.6% compared to the SPDI, without considering the effect of CVs.

1. Introduction

Due to the increase in automobile ownership, vehicle-miles travelled, and the limited capacity of roadway infrastructure, roadway facilities have become incapable of coping with the increasing traffic demands. As a consequence, traffic congestion is becoming a serious problem in many urban areas around the globe. Therefore, traffic congestion mitigation measures are being explored and implemented widely to help alleviate congestion and reduce delays. Unconventional intersection and interchange designs are among the most effective mitigation measures in certain situations (Hughes et al., 2010). The Conventional Diamond Interchange (CDI) is the most implemented type of service interchanges in North America (Garber & Fontaine, 1999). However, the CDI is not the best interchange design for all traffic conditions. For instance, the CDI operates poorly in the event of high left-turn traffic volumes. Chlewicki (2003) developed the Diverging Diamond Interchange (DDI) as an alternative to the CDI that could accommodate larger left-turn traffic volumes while operating with one less signal phase, compared to the CDI. Therefore, there has been nearly 100 implementations of the DDI in the U.S. (Advanced Transportation Solutions/American, 2017). Another alternative to the CDI is the Single Point Diamond Interchange (SPDI) which was conceived and developed in the early 1970s to improve the operational performance (Hughes et al., 2010). According to the results of a survey (Qureshi et al., 2004), 14 states favor the SPDI over other designs dure to considerations that include the right-of-way, frontage roads, traffic volume, and coordination of intersections.

Recently, the introduction of intelligent transportation systems and Connected Vehicle (CV) technology became a promising solution in mitigating the global problem of traffic congestion. CVs use wireless communication technologies to communicate with each other, and the infrastructure through the Vehicle-to-Vehicle (V2V) communication, and the Vehicle-to-Infrastructure (V2I) communication, respectively. According to the U.S. Department of Transportation, CV implementation efforts have shown a potential for notable mobility, safety and environmental benefits (U.S. Department of Transportation, 2016).

However, studies on the coupled implementation of unconventional designs and CVs have been limited. Therefore, this paper aims to evaluate the mobility impact of implementing V2I communications at the CDI, DDI, and SPDI. This study takes into consideration the effects of varying Market Penetration Rates (MPRs) of CVs. VISSIM microsimulation software was used to create a replica of a real-world transportation network and evaluate its operational performance. The required V2I communication algorithms that help exchange the vehicle type, speed, acceleration, and accurate latitude/longitude coordinates are programmed in Python and embedded in the simulation networks through VISSIM's Component Object Model (COM). This study also takes into account the effects of varying market penetration rates of CVs.

This paper is organized as follows. The literature review is presented in the next section. The following section is dedicated to the research methodology. The results and discussion are provided in section four. Lastly, the conclusions and recommendations are discussed in the final section.

2. Literature Review

2.1. Alternative Designs

Unconventional interchange designs date back to the early 1970s when the first SPDI was developed and implemented as an alternative to improve traffic operations at CDIs (Hughes et al., 2010). Since then, many researchers (Alzoubaidi et al., 2021; Alzoubaidi & Zlatkovic, 2022; Bared et al., 2005; Dorothy et al., 1997; Hughes et al., 2010; Hunter et al., 2019; Reid & Hummer, 1999; Speth & Siromaskul, 2008; Tabernero & Sayed, 2006; Zlatkovic, 2015; Zlatkovic & Kergaye, 2018) have been investigating the mobility and safety impacts of unconventional designs using microsimulation tools, among other methodologies. Dorothy et al. (1997) used TRAF-NETSIM to evaluate the operational characteristics of the CDI against the Michigan Urban Diamond Interchange (MUDI). The MUDI outperformed the CDI by reducing travel times by as high as 60%. Miller & Vargas (1999) studied the operational performance of each of the echelon, compressed diamond and SPDI using TRAF-NETSIM. The results of their study showed that none of the approaches of the echelon had a failing Level of Service (LOS). The compressed diamond, however, had a LOS of F for two of its approaches, while the SPDI had a LOS of F for three of the approaches. Reid & Hummer (1999) and Reid & Hummer (2001) used CORSIM and Synchro to compare the operational performance of various designs including the quadrant intersection, the Median U-Turn (MUT), jughandle, bowtie, superstreet, continuous flow intersection, and split intersection. The MUT and quadrant intersection generated the lowest travel times, consistently. Chlewicki (2003) developed and introduced the synchronized split-phasing intersection and the DDI as promising alternatives to their conventional counterparts, namely, the conventional signalized intersection and the CDI, respectively. Chlewicki (2003) found the two new designs to be superior to their conventional counterparts in terms of delays. According to Chlewicki (2003) the DDI reduced delays by approximately 67.5%. Bared et al. (2005) used VISSIM to analyze the operational performance of the DDI and the Double Crossover Intersection (DCI) under different traffic scenarios. The authors found that these two designs reduced delays by up to 60%, shortened queues, and led to a higher throughput when compared to their conventional counterparts. Chlewicki (2011) conducted a comparative operational analysis between the CDI, DDI and SPDI using the critical lane volume method. The author concluded that the DDI may not always perform the best. Khan & Anderson (2016) investigated whether the DDI could be applicable as an alternative to the CDI under various performance measures and scenarios. The study results concluded that the DDI is not always the best alternative to the CDI. Sannikov et al. (2018) compared the operational aspects of the CDI, three-level SPDI, and Diverging Windmill Interchange (DWI). The authors found that the DWI is the best design, in that it reduces delays by an average of 90 seconds and increases speeds by up to 33.3%. The SPDI, on the other hand, was found to provide the best performance overall among multiple interchanges, in a study conducted via VISSIM (Click et al., 2010).

However, the SPDI is known for its relatively large bridge structure which results in higher construction costs when compared to other designs (Hughes et al., 2010). In addition, longer yellow and all-red intervals are required for safe operations of the SPDI, due to their extreme widths (Hughes et al., 2010; Speth & Siromaskul, 2008). These longer yellow and all-red intervals have caused the SPDI to perform worse than the DDI, according to Speth & Siromaskul (2008). Alzoubaidi et al. (2021) employed VISSIM to evaluate the benefits of implementing the Super Diverging Diamond Interchange (SDDI) against those of implementing other interchange designs, including the CDI, DDI, Folded Diamond Interchange (FDI), Diverging Partial Cloverleaf Interchange (DPCI), and Ramp Crossover Interchange (RCI) designs.

The results showed that the FDI is superior in performance to all the other designs in most of the tested scenarios. Particularly, the FDI reduced travel times by a range of 2.6% to 35.0% compared to the other designs. Similar findings were also reported by Riniker (2009), who found that the FDI outperformed the SPDI, CDI, DDI among other configurations of interchange designs.

2.2. Connected Vehicles

Olia et al. (2016) evaluated the impact of varying Market Penetration Rates (MPRs) of CVs using PARAMICS. The results of the study showed that CVs could improve safety, reduce travel times and emissions by 45%, 37% and 30%, respectively. With the goal of optimizing traffic operations at signalized intersections, Sida & Shuai (2019) developed a dynamic control strategy and implemented it in the cellular automata models. The findings showed that delays were reduced by 15% as a result of the implemented control strategy. Lioris et al. (2017) used PointQ simulation software combined with CV algorithms to examine the benefits brought by platooning at signalized intersections. The authors concluded that the capacity of intersections increased due to platooning that is achieved by the implementation of CV technology. Guler et al. (2014) developed a CV algorithm that optimizes intersections' performance using data obtained from CVs. The results of the study indicated substantial delay reductions if the MPR of CVs was between 20% and 60%. Letter & Elefteriadou (2017) developed a freeway merging control algorithm of fully automated CVs. In their study, they simulated a hypothetical freeway merging segment and compared the performance of their algorithm, when applied to this segment, to the performance of conventional vehicles. They found that the algorithm increases travel speeds, reduces travel times, and provides safe merging operations during congested traffic scenarios. Cvijovic & Zlatkovic (2019) used VISSIM and V2I communication protocols to examine the benefits of CV-based Freight Signal Priority (FSP). The findings of the study showed nearly 35% to 48% reductions in delays for trucks equipped with the CV technology, compared to non-equipped trucks. Martin-Gasulla & Elefteriadou (2021) developed a roundabout manager that could provide for optimal coordination of CVs entering and exiting roundabouts with the goal of maximizing throughput and minimizing average control delay. The authors concluded that the capacity of roundabouts increased between 58% and 73% under fully CV environment. The authors also added that the average control delay was reduced by 80% to 97%, compared to non-CV environments. Amini et al. (2021) proposed mathematical models to optimize CV trajectories navigating weaving segments assuming a 100% CV MPR. The results suggest that their developed models increase the capacity and speed by up to 11% and 16%, respectively. Saldivar-Carranza et al. (2021) developed a new method to help in the assessment of DDI performance using CV trajectories. Zhong & Lee II (2019) used VISSIM to investigate the operational effects of CVs at CDIs, and DDIs. The study results showed that converting CDIs to DDIs would result in 20% increased throughput and implementing CV technologies at DDIs further increased the throughput by nearly 7%.

As seen from the literature presented, many studies have examined the operational performance of the SPDIs, CDIs, and DDIs when navigated by conventional vehicles. However, their operational performance in the era of CVs is still not well investigated. In fact, to the best of the authors' knowledge, a comparative operational analysis between the combined deployment of CVs at each of the CDIs, DDIs, and SPDIs has not been studied previously. Therefore, this paper aims to contribute to the literature by targeting this knowledge gap.

3. Research Methodology

3.1. Simulation Testbed and Data Collection

With more than 35% of Utah's total population, Salt Lake County ranks number one as the county with the highest population (U.S. Census Bureau, 2019), making it the county with highest traffic demands. Therefore, the interchange of I-15 and a high demand major urban arterial (12300 S) in Salt Lake County, was selected as the study area. It should be mentioned that the currently implemented interchange design at this location is a SPDI. This study also included the nearby intersections in the models as shown in Figure 1, since these intersections are likely to affect lane choices and arrival patterns. All required traffic data for the creation of simulation models were collected from the Utah Department of Transportation (UDOT) Traffic Operations Center, as well as UDOT's Automated Traffic Signal Performance Measures (ATSPMs) system. The data consisted of traffic signal timing data, traffic composition, traffic volumes and turning movement counts. Data analysis showed the peak hour to be from 4:45 to 5:45 P.M. during a typical work week.

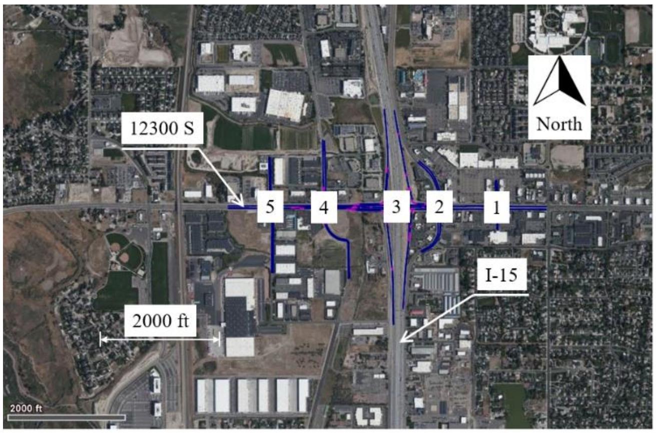

Figure 1. Testbed Network of I-15 and 12300 S, Salt Lake County, Utah.
1 – 210 East (conventional intersection);
2 – State St. (conventional intersection);
3 – I-15 (SPDI);
4 – Lone Peak Pkwy (conventional intersection);
5 – S 256 W (conventional intersection).
Source: (GoogleEarth).

3.2. Microsimulation Modeling Methodology

To ensure high quality recreation of the selected section of 12300 S in a simulation environment, VISSIM microsimulation software was used in conjunction with ASC/3 Software-in-the-Loop (SIL) signal controllers and Python-programmed V2I communication algorithms. The obtained data from UDOT along with VISSIM's high resolution Bing Maps served as the basis for creating the simulation models. This helped ensuring accurate modelling and representation of the geometric and traffic conditions of the study area. Since a variation in the CV MPR is expected to influence the operational performance of transportation networks, this study has considered implementing CV technology using three CV MPRs as follows: 0% CV MPR, 50% CV MPR and 100% CV MPR. Therefore, nine simulation models were created for the purposes of this study as shown in Table 1.

Table 1. Simulation Model Description

Model Number	Interchange Design	CV MPR	Description
1	SPDI	0%	Current Situation
2	SPDI	50%	Suggested Alternative
3	SPDI	100%	Suggested Alternative
4	CDI	0%	Suggested Alternative
5	CDI	50%	Suggested Alternative
6	CDI	100%	Suggested Alternative
7	DDI	0%	Suggested Alternative
8	DDI	50%	Suggested Alternative
9	DDI	100%	Suggested Alternative

The V2I communication algorithms used are a vital component of this study as they facilitate the two-way exchange of latitude/longitude coordinates between signal controllers and vehicles. Exchange of latitude/longitude coordinates requires the conversion of VISSIM coordinates to real-world coordinates which was made possible through the use of VISSIM's user-defined attributes. The V2I communication algorithm functionality requires distance calculation between vehicles and signal controllers in each simulation time step, this was achieved using the following formulae (Upadhyay, 2020):

$$d = R \times c \tag{1}$$

Where *R* represents the Earth's radius of 20,902,231 ft (~3,960 mi), and *c* is computed using the following equation.

$$c = 2 \times \operatorname{atan2}\left(\sqrt{a}\sqrt{1-a}\right) \tag{2}$$

The atan2 function yields the angle between the line projected by the input coordinates and the positive side of the abscissa. The outputted angle ranges between -*π* and *π* radians (Palm, 2005). The parameter, *a*, is computed using the Haversine formula expressed as follows (Upadhyay, 2020):

$$a = \sin^2\left(\frac{\Delta lat}{2}\right) + \cos(lat_{veh}) \times \cos(lat_{int}) \times \sin^2\left(\frac{\Delta long}{2}\right) \tag{3}$$

The terms, lat_{int} and lat_{veh}, are the latitudinal coordinate of the intersection's center the latitudinal coordinate of the vehicle, respectively. The term *Δlat* represents the difference between the latitudinal coordinates ($lat_{veh} - lat_{int}$). Similarly, *Δlong* represents the difference between the longitudinal coordinate of the vehicle, $long_{veh}$, and the longitudinal coordinate of the intersection's center, $long_{int}$, expressed as $long_{veh} - long_{int}$.

In the developed V2I algorithms, to ensure consistency with actual CV Dedicate Short-Range Communication (DSRC) technology, communication radii around intersections were set to 1,000 ft. As a result, in the simulation models, when vehicles enter the DSRC communication range of 1,000 ft the V2I algorithm would help change their behavior to CV behavior. It should be noted that, in this study, conventional vehicles and CVs that are out of the communication range behave in accordance with VISSIM's Wiedemann-74 for urban traffic driving behavior. Multiple studies (M. S. Rahman et al., 2019; S. Rahman & Abdel-Aty, 2018; Sinha et al., 2020; Yang et al., 2020) have successfully modelled CV behavior through modifications made to VISSIM Wiedemann's car following models. Therefore, in this study, a modified Wiedemann-74 that could simulate CV behavior has been coded based on recommendations from the mentioned studies. As a result, in simulations once CVs are within the communication range of 1,000 ft, they would behave according to the adjusted values of the Wiedemann-74 car following model presented in Table 2.

Table 2. Wiedemann-74 Car Following Model Parameters.

Wiedemann-74 Parameter	Default Value	CV Modified Value
Average standstill distance	2.00 m	1.50 m
Additive part of safety distance	2.00	1.25
Multiplicative part of safety distance	3.00	2.25

3.2. Calibration and Validation

The base simulation model, which represents the current situation of a SPDI, and 0% CV MPR was created using the existing field data from UDOT along with VISSIM's high resolution Bing Maps.

The current common practice of multiple agencies across the nation for the calibration and validation procedure suggests the use of the Geoffrey E. Heavers (GEH) statistic, along with linear regression in calibrating traffic simulation models (Florida Department of Transportation, 2014; Oregon Department of Transportation, 2011; Utah Department of Transportation, 2018; Washington State Department of Transportation, 2014). These practices also recommend the use of any other traffic element that can be measured in the field for validation purposes. It should be mentioned that a GEH value that is less than 5 implies a well calibrated simulation model (Florida Department of Transportation, 2014; Oregon Department of Transportation, 2011; Utah Department of Transportation, 2018; Washington State Department of Transportation, 2014). Therefore, in this study, the GEH statistic and linear regression were employed for calibration, whereas the linear regression was used for validation.

The calibration and validation results of the base model are shown in Figure 2. For the base model calibration, field-collected turning movement counts were compared to simulation-generated turning movement counts. As recommended, the GEH statistic was used along with the linear regression to assess the fidelity of the base model. As depicted in Figure 2, GEH and R2 values were 0.22 and close to 1.0, respectively, showing a strong correlation between the field-collected and simulation-generated traffic volumes. For base model validation, the phase green times that were obtained from UDOT's ATSPMs were plotted against the simulation-generated phase green times. The R2 value of 0.97 shows a good comparison between the simulation and field phase green times. Both results of the calibration and validation show a good fidelity of the base simulation model.

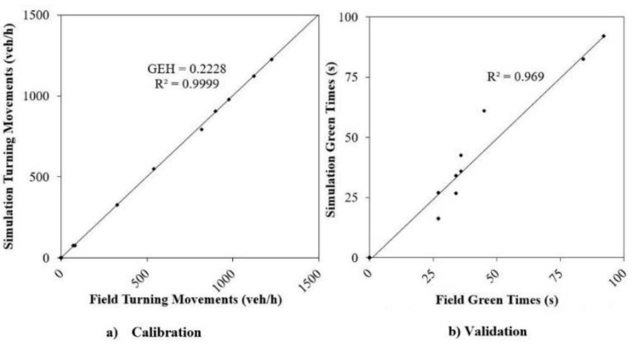

Figure 2. VISSIM model calibration and validation scatterplots.

4. Results and Discussion

Travel times, delays, queue lengths, and the number of stops were the performance metrics used in this study to evaluate the impact of CVs at the CDI, DDI, and SPDI. These metrics were extracted and averaged from 10 randomly seeded simulation runs.

Two-tailed t-tests were employed to evaluate the statistical significance of the results. Tables 3 and 4, show the performance metrics results for different interchanges and same CV MPR. Tables 5 and 6, show the performance metrics results for different CV MPRs and same interchange design.

Table 3. Mean Vehicular Travel Times (s) for Different Interchanges and Same CV MPR.

Direction (From – To)[c]	Mean Vehicular Travel Times (s)[a,b]					
	0% MPR			50% MPR		
	SPDI	CDI	DDI	SPDI	CDI	DDI
WBT (2 – 4)	62.12	71.32	81.04	61.59	69.52	80.52
P-value		<0.01	<0.01		<0.01	<0.01
EBT (4 – 2)	64.29	79.69	105.95	61.69	76.67	105.00
P-value		<0.01	<0.01		<0.01	<0.01
NBR (I-15 – 2)	96.03	133.95	80.44	95.82	130.16	79.38
P-value		<0.01	<0.01		<0.01	<0.01
NBL (I-15 – 4)	125.69	196.07	135.69	126.68	195.27	134.31
P-value		<0.01	<0.01		<0.01	<0.01
SBL (I-15 – 2)	113.95	157.88	122.60	113.07	154.08	121.33
P-value		<0.01	<0.01		<0.01	<0.01
WBL (2 – I-15)	132.84	158.54	80.08	131.98	157.70	80.01
P-value		<0.01	<0.01		<0.01	<0.01
EBL (4 – I-15)	112.23	188.89	84.06	105.92	182.94	81.59
P-value		<0.01	<0.01		<0.01	<0.01

Direction (From – To)[c]	100% MPR		
	SPDI	CDI	DDI
WBT (2 – 4)	60.79	67.95	80.11
P-value		<0.01	<0.01
EBT (4 – 2)	59.37	74.10	103.89
P-value		<0.01	<0.01
NBR (I-15 – 2)	96.43	125.63	78.17
P-value		<0.01	<0.01
NBL (I-15 – 4)	128.32	193.01	134.38
P-value		<0.01	<0.01
SBL (I-15 – 2)	113.23	152.52	120.05
P-value		<0.01	<0.01
WBL (2 – I-15)	134.38	157.64	79.62
P-value		<0.01	<0.01
EBL (4 – I-15)	101.46	179.95	80.02
P-value		<0.01	<0.01

[a]: Southbound Right (SBR) results are not included in the table since UDOT has not provided traffic volumes for the SBR movement.
[b]: WBT is Westbound Through; EBT is Eastbound Through; NBR is Northbound Right; NBL is Northbound Left; SBL is Southbound Left; WBL is Westbound Left; EBL is Eastbound Left.
[c]: Refer to Figure 1.

Table 4. Mean Delays, Number of Stops, and Queue Lengths for Different Interchanges and Same CV MPR.

	Vehicle Delay (s)[a]		
	SPDI	CDI	DDI
0% MPR	24.57	30.24	9.67
P-value		<0.01	<0.01
50% MPR	23.59	29.24	9.48
P-value		<0.01	<0.01
100% MPR	23.00	28.46	9.38
P-value		<0.01	<0.01
	Number of Stops[a]		
	SPDI	**CDI**	**DDI**
0% MPR	0.42	0.51	0.23
P-value		<0.01	<0.01
50% MPR	0.40	0.49	0.23
P-value		<0.01	<0.01
100% MPR	0.39	0.48	0.23
P-value		<0.01	<0.01
	Queue Length (m)[a]		
	SPDI	**CDI**	**DDI**
0% MPR	14.10	25.90	7.20
P-value		<0.01	<0.01
50% MPR	13.05	24.00	6.71
P-value		<0.01	<0.01
100% MPR	12.44	22.65	6.42
P-value		<0.01	<0.01

a: These performance metrics are measured at the interchange level (i.e., the results are for the entire interchange).

Except for the EBT and WBT movements, Table 3 shows that the CDI has the longest travel times for all movements among all the studied designs and tested scenarios of this study. This is consistent with previous studies, since the DDI was created to accommodate larger left-turn demands than the CDI (Chlewicki, 2003). However, in higher through traffic demands, the CDI would outperform the DDI (Chlewicki, 2011; Khan & Anderson, 2016). Similarly, Table 4 shows that the CDI has the longest delays, queue lengths and highest number of stops among all the studied designs and tested scenarios. This not uncommon, since each of SPDI and DDI were created to improve the operational performance of CDIs. A new finding, however, is that CDI would still perform the worst of all the three designs even under varying CV MPRs. Analyzing the effect of alternative interchange designs on traffic operations within the same CV MPR shows that the interchange design has a considerable impact on the operational performance. For instance, converting the SPDI to a CDI increased travel times between 11.8% and 77.3%, depending on the traffic movement while keeping the CV MPR at the same level. In a similar manner, when the CV MPR is at 100% converting the SPDI to a DDI, would increase the NBL (from I-15 to intersection 4) travel time by 4.7% and the EBT (from intersection 4 to intersection 2) travel time by nearly 75.0%. When the CV MPR is at 0% converting the SPDI to a DDI would decrease the NBR (from I-15 to intersection 2) travel time by 16.2%. Additionally, at the 100% CV MPR converting the SPDI to a DDI would reduce the WBL (from intersection 2 to I-15) travel time by 40.8%. At the same CV MPR, substituting the SPDI with a CDI would increase delays by up to 24.0%, while substituting the SPDI with a DDI would lead to reductions in delays of up to 60.6%. Analysis of the number of stops results at the same CV MPR shows that a CDI would increase the number of stops by up to 23.1%, while a DDI would decrease the number of stops by as high as 45.2%, when both designs are compared to the SPDI. As for the queue lengths, considering the same CV MPR, the DDI implementation reduced queue lengths by up to 48.4%, while the CDI implementation increased queue lengths by up to 82.1%.

Table 5. Mean Vehicular Travel Times (s) for Different CV MPRs and Same Interchange Design.

Direction (From – To)[c]	Mean Vehicular Travel Times (s) [a,b]					
	SPDI			CDI		
	0% MPR	50% MPR	100% MPR	0% MPR	50% MPR	100% MPR
WBT (2 – 4)	62.12	61.59	60.79	71.32	69.52	67.95
P-value		<0.01	<0.01		<0.01	<0.01
EBT (4 – 2)	64.29	61.69	59.37	79.69	76.67	74.10
P-value		<0.01	<0.01		<0.01	<0.01
NBR (I-15 – 2)	96.03	95.82#	96.43#	133.95	130.15	125.63
P-value		0.72#	0.69#		<0.01	<0.01
NBL (I-15 – 4)	125.69	126.68#	128.33	196.07	195.27#	193.01
P-value		0.09#	<0.01		0.32#	<0.01
SBL (I-15-2)	113.95	113.07	113.23	157.88	154.08	152.52
P-value		0.02	<0.01		<0.01	<0.01
WBL (2-I-15)	132.84	131.98#	134.38#	158.54	157.70#	157.64#
P-value		0.48#	0.24#		0.14#	0.17#
EBL (4 – I-15)	112.23	105.92	101.46	188.89	182.94	179.95
P-value		<0.01	<0.01		<0.01	<0.01

Direction (From – To)[c]	DDI		
	0% MPR	50% MPR	100% MPR
WBT (2 – 4)	81.04	80.52	80.11
P-value		<0.01	<0.01
EBT (4 – 2)	105.95	105.00#	103.89
P-value		0.07#	<0.01
NBR (I-15 – 2)	80.44	79.38	78.17
P-value		0.06	<0.01
NBL (I-15 – 4)	135.69	134.31#	134.38#
P-value		0.07#	0.16#
SBL (I-15 – 2)	122.61	121.33	120.05
P-value		<0.01	<0.01
WBL (2 – I-15)	80.08	80.01#	79.62#
P-value		0.83#	0.25#
EBL (4 – I-15)	84.06	81.59	80.02
P-value		<0.01	<0.01

a: Southbound Right (SBR) results are not included in the table since UDOT has not provided traffic volumes for the SBR movement.
b: WBT is Westbound Through; EBT is Eastbound Through; NBR is Northbound Right; NBL is Northbound Left; SBL is Southbound Left; WBL is Westbound Left; EBL is Eastbound Left.
c: Refer to Figure 1.
#: The mean difference is statistically insignificant at the 95% confidence level when compared to the 0% MPR.

Table 6. Mean Delays, Number of Stops, and Queue Lengths for Different CV MPRs and Same Interchange Design.

	Vehicle Delay (s)[a]		
	0% MPR	50% MPR	100% MPR
SPDI	24.57	23.59	23.00
P-value		<0.01	<0.01
CDI	30.24	29.24	28.46
P-value		<0.01	<0.01
DDI	9.67	9.48	9.38
P-value		<0.01	<0.01
	Number of Stops[a]		
	0% MPR	50% MPR	100% MPR
SPDI	0.42	0.40	0.39
P-value		<0.01	<0.01
CDI	0.51	0.49	0.48
P-value		<0.01	<0.01
DDI	0.23	0.23#	0.23#
P-value		0.19#	0.68#
	Queue Length (ft)[a]		
	0% MPR	50% MPR	100% MPR
SPDI	14.10	13.05	12.44
P-value		<0.01	<0.01
CDI	25.90	24.00	22.65
P-value		<0.01	<0.01
DDI	7.20	6.71	6.42
P-value		<0.01	<0.01

[a]: These performance metrics are measured at the interchange level (i.e., the results are for the entire interchange).

#: The mean difference is statistically insignificant at the 95% confidence level when compared to the 0% MPR.

Table 5 shows that the deployment of CV technology would lead to savings in travel times of 0.6% to 9.6% depending on CV MPR, with more than half the travel time reductions being statistically significant. Table 5 also shows that CV deployments have increased travel times for some traffic movements by 0.4% to 2.1%. Most of the travel time increases due to CV deployments, however, were statistically insignificant. According to the results of Table 6, deploying CV technology at any MPR has notably reduced the vehicular delays, and shortened the queue lengths at all interchange designs. The number of stops were also reduced as the CV MPR increased for all designs except for the DDI. Particularly, CV deployments have reduced delays by 4.0% to 6.4%, 3.3% to 5.9%, and 2.0% to 3.0%, at the SPDI, CDI, and DDI, respectively, depending on CV MPR. Similarly, CV deployments have reduced the number of stops by 4.8% to 7.1%, and 3.9% to 5.9%, at the SPDI, and CDI, respectively, depending on CV MPR. Furthermore, CV implementations have shortened the queue lengths by 7.4% to 11.8%, 7.3% to 12.5%, and 6.8% to 10.8%, at the SPDI, CDI, and DDI, respectively, depending on CV MPR.

5. Conclusion

The goal of this paper was to study the mobility impacts of implementing CV technology at the CDI, DDI, and SPDI. This goal was achieved through the use of VISSIM microsimulation software in conjunction with ASC/3 SIL and Python-programmed V2I communication algorithms. In this simulation environment, a replica of a real-world transportation network was created to evaluate its operational performance in the presence of CVs. This study considered the effects of varying CV MPRs on traffic operations.

Based on the results, the interchange design has a higher impact on traffic operations than does the CV MPR. For instance, at the same CV MPR, converting the SPDI to a CDI would increase travel times, delays, the number of stops, and queue lengths by up to 77.3%, 24.0%, 23.1%, and 82.1%, respectively. In a similar manner, when the CV MPR is at 100% converting the SPDI to a DDI, would increase the NBL (from I-15 to intersection 4) travel time by 4.7% and the EBT (from intersection 4 to intersection 2) travel time by nearly 75.0%. This high increase in travel time is attributed merely to the interchange design since the CV MPR is at 100% in both cases. When the CV MPR is at 0% converting the SPDI to a DDI would decrease the NBR (from I-15 to intersection 2) travel time by 16.2%. Additionally, at the 100% CV MPR converting the SPDI to a DDI would reduce the WBL (from intersection 2 to I-15) by 40.8%. Moreover, by keeping the CV MPR at the same level and converting the SPDI to a DDI the delays, number of stops, and queue lengths would be reduced by up to 60.6%, 45.2%, and 48.4%, respectively.

On the other hand, the impact of CV deployments on travel time savings was between 0.6% and 9.6%, at the 50% and 100% CV MPRs, respectively. While CV deployments have led to reductions in the delays, number of stops, and queue lengths, their impact was less considerable than the impact of the interchange design. Specifically, CV deployments have reduced delays by 4.0% to 6.4%, 3.3% to 5.9%, and 2.0% to 3.0%, at the SPDI, CDI, and DDI, respectively, depending on CV MPR. Similarly, CV deployments have reduced the number of stops by 4.8% to 7.1%, and 3.9% to 5.9%, at the SPDI, and CDI, respectively, depending on CV MPR. Furthermore, CV implementations have shortened the queue lengths by 7.4% to 11.8%, 7.3% to 12.5%, and 6.8% to 10.8%, at the SPDI, CDI, and DDI, respectively, depending on the CV MPR.

While this paper focused mainly on the operational impact of CVs at each interchange design, current work is underway for further investigating the safety impact. An interesting area for future research might be looking into the benefits and effects of implementing V2V communication algorithms at similar interchange configurations. Additionally, it is recommended that a similar experiment be conducted in a driving simulator environment to explore any benefits the CVs would bring to alleviate the driver confusion these unconventional designs may induce.

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