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Research Note

A Comparison Between Single Point and Conventional Urban Diamond Interchanges Based on Estimated Delay

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In this paper, the results of a study comparing the performance of two types of interchange, i.e., Conventional Diamond Interchange and Single Point Urban Diamond Interchange, are reported. The analysis is based on simulation of a Single Point Urban Diamond Interchange. The real data that is collected from two different existing Conventional Diamond Interchanges during peak hours is used as input for analysis. Highway capacity software is used to analyze the performance of the Conventional Diamond Interchanges while a simulation model is used for analyzing the performance of the Single Point Urban Diamond Interchanges. The results indicate that the performance of the Single Point Urban Diamond Interchange is superior to that of the Conventional Diamond Interchange.

INTRODUCTION

Urban traffic congestion and delay are two of the biggest challenges faced by traffic engineers. Transportation professionals have tried several measures, both conventional and innovative, to reduce congestion and make traffic flow smoothly, safely and more efficiently. Two areas which directly affect traffic operations at signalized intersections are geometric and signal timing and traffic engineers have, thus, tried to make improvements in these two areas in an attempt to increase capacity and reduce delays.

The Single Point Urban Diamond Interchange (SPUDI) is a design concept developed in the 1960s to address congestion problems at interchange locations. It is a modification of the Conventional Diamond Interchange design (CDI), in which the two intersections on the arterial road are moved in closer, to form one large signalized intersection. Figure 1 shows the features of the two types of interchange. Because of its salient features, it is believed that the SPUDI has a great potential for reducing congestion and delays and, thereby, increasing capacity. Some of the salient features of SPUDI are as follows:

1. Because the number of intersections on the arterial road is reduced by 1 for every SPUDI (it combines two intersections into one), the probability of a vehicle having to stop along the arterial road is reduced. This will increase average speed along the arterial road, resulting in a better level of service and increased capacity;
2. Another advantage of reducing the number of intersections along the arterial road is that it is possible to achieve a better coordination of signals. It is a well-known fact that the larger the number of intersections, the more difficult it is to achieve coordination of signals to obtain progression in both directions. Note that if the progression is required in only one direction, then the number of intersections will not pose as much of a problem in obtaining a large bandwidth. Hence, SPUDI will help in obtaining better progression in both directions along the arterial road;
3. Because of the large turning radii for both left and right turns in SPUDI, the turning speeds will be higher, resulting in a higher saturation flow rate, higher capacity and lower delays, as compared to CDI. In fact, the left turning speeds are identical to through speeds because of the very large turning radii;
4. Because the two intersections are combined into one, the right of way required in the direction of the arterial road is greatly reduced for SPUDI compared to CDI. Also, in the case of SPUDI, the

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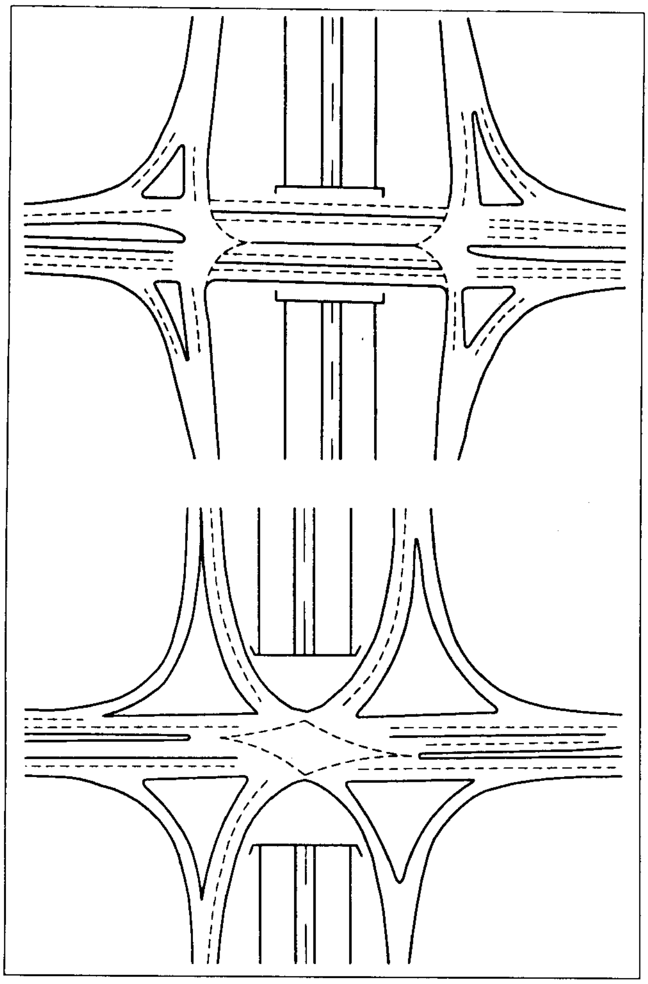

Figure 1. Geometric features of the SPUDI (top) and the CDI (bottom) [1].

opposing left turns on the arterial road may be placed facing each other. This will reduce the width of intersection, as compared to CDI with the same number of lanes and, hence, will also reduce the right of way required for SPUDI in the direction perpendicular to the arterial. If the arterial has dual left turn lanes, then the reduction in the right of way requirement is equal to the total width of the left turn lanes that can be placed facing each other. Reduction in the right of way requirement is a very significant advantage considering the fact that these interchanges are built in urban environments where cost of real estate is very high and sometimes the surrounding land is already highly developed.

Because of the great potential that SPUDIs possess, many states have chosen to construct SPUDIs and many other states are considering constructing SPUDIs in urban locations. However, only a handful of studies have been conducted to evaluate the performance of SPUDIs and, as will be mentioned in the next section, some of these studies have come to conflicting conclusions.

A fact that one can notice from literature reviews is that several different signalized intersection analysis software packages have been used to compare the two forms of interchange. These include Highway Capacity Software (HCS), TRANSYT 7F, TRAF-NETSIM and PASSER III 88. There is a very good possibility that the different models and their usage may have contributed to the conflicting results of different studies. Hence, it is desirable to study and understand what goes on in these signal analysis models and see whether they are capable of analyzing or simulating traffic operations at SPUDIs. SPUDIs are the main concern because they are a relatively new design concept which none of these signal analysis packages were designed to handle. These packages were developed for analyzing regular intersections in CBDs, where the typical curb radius is 20 feet, there is no median, turning speeds are very low and intersection areas are very small (75-100 feet wide). Although SPUDI may be considered as a signalized single intersection, its geometric features are very unique and very different from regular CBD intersections. Shafahi et al. [2] developed and tested a special micro simulation model for analyzing traffic operation at SPUDI with actuated signals.

This paper reports the results of a study comparing the performance of CDI and SPUDI using real data that has been collected from two different CDI during the morning and afternoon peak hours. For analyzing the performance of the SPUDI, the simulation model developed by Shafahi et al. [2] was used, while HCS was used to analyze the performance of the CDI. The reason for using the HCS for analyzing CDI is that CDI can be considered as two urban intersections on an arterial road and the HCS is the most widely used software package for analyzing signalized intersections.

LITERATURE REVIEW

Since SPUDIs are a recent development, it is understandable that only a handful of studies have been conducted to evaluate their performance. The few studies that were done have come to conflicting conclusions.

Leisch et al. [1] studied 5 locations with a wide variety of traffic volumes and patterns. The TRANSYT 7F model was used to analyze the two forms of interchange, SPUDI and CDI, that could have been built at these 5 locations. Measures of effectiveness obtained were the total system delay (veh-hr/hr) and average intersection system delay (sec/veh). This study found that the total and average delay were higher for the SPUDI option at 4 out of 5 locations. The only location where the SPUDI was found to be more efficient had heavy left turns on all approaches, resulting in a shorter headway and higher saturation flow rate.

In another study, Malek [3] compared operations at an existing SPUDI at Norcross, Georgia, with an imaginary CDI having similar lane configurations. He obtained the optimum cycle length for the CDI, using the PASSER III 88 model, and used the existing signal timing parameters for the SPUDI. The TRAF-NETSIM model was used to simulate operations at SPUDI and CDI, assuming pretimed signal operations.

Based on this study, it was concluded that the CDI can handle a variety of traffic volumes and patterns more efficiently than the SPUDI. The point to be noted here is that CDI with optimum signal timing was compared with SPUDI with existing signal timing. The existing signal timing of SPUDI may or may not have been optimum. For the comparison to be valid, both facilities should have been analyzed with optimum signal timing.

Izadi [4] compared the operations at a SPUDI in Orange county, California, with a CDI that would have had similar lane configurations. Signals were assumed to be actuated. He used HCS to estimate the delay and level of service of the intersections and the arterial road. He also compared the total system delay, which is the sum of the intersection and arterial delay for the two types of interchange. According to this study, the SPUDI looks more efficient compared to CDI.

One of the most recent and comprehensive of SPUDI studies was undertaken by Messer et al. [5] in an NCHRP project in 1989-1991. This study has resulted in outlining broad guidelines for the design of an ideal SPUDI. These guidelines cover geometric features, traffic signal control and other traffic control devices, pedestrian considerations and safety experience, roadway lighting, bridge design guidelines, sight distance, left turn path geometry, bridge length and cost effectiveness analysis. These guidelines are based on the operational experience of 27 SPUDIs that were visited during the field survey part of this project. For the purpose of economic analysis, this study has compared a SPUDI with an At-Grade-Intersection (AGI) using cost-benefit methodology. For the comparison to be more meaningful, the SPUDI should have been compared with interchange forms other than an AGI. When grade separation is the only alternative left to improve the situation at an AGI, the two viable options are SPUDI and CDI because of the limitations on the right of way in an urban environment. An economic analysis comparing these two interchange forms would have provided very useful information for an engineer faced with the problem of selecting the type of interchange to be constructed in an urban background.

Since SPUDIs are relatively new, the accident experience with SPUDIs is very limited. However, the limited information available on accidents at SPUDIs is encouraging. Cheng [6] has compared accident records at 3 SPUDIs in Utah with 3 CDIs. This study found that the accident rates at SPUDIs are lower and the accidents are less severe compared to accidents at CDIs. This study also concluded that: 1) Driver unfamiliarity with the new design is not a major factor in accident occurrence in the SPUDI area and 2) Factors of severity, weather, road surface, lighting, older drivers, pedestrians and trucks do not present any major problems through the SPUDI area. It should, however, be noted that two of the SPUDIs had accident data for only one year. Hence, accident data needs to be collected at further locations for a few more years before any definitive conclusions can be drawn regarding the accident experience at SPUDIs.

Brown and Walters [7] have given a general description of operational characteristics, right of way requirements, access to adjacent parcels, pedestrian and transit considerations and cost. No mathematical models, quantitative analysis or specific conclusions are given in the paper.

METHODOLOGY

Total delay is chosen as a criterion for comparison between the CDI and the SPUDI. Since the comparison is based on benefits that may be accrued, it is desirable to minimize the cost of each alternative, based on savings obtained from a reduction in average stopped vehicle delay. In this comparison, it will be assumed that, in general, if the delay at a signalized intersection is reduced, then a cost saving will result, due to a reduction of overall costs associated with pollution, fuel consumption, time and vehicle maintenance.

Highway Capacity Manual Delay Estimates

In order to obtain delay estimates for the CDI, techniques from the 1994 Highway Capacity Manual (HCM) [8] were used in the form of the Highway Capacity Software (HCS) package. The method employed by the 1994 HCM describes a series of ideal conditions that include:

- Lane widths of 12 ft,
- Level grade,
- No curb parking,
- All passenger cars in traffic stream,
- No bus stoppage,
- No turns from through lanes,
- Not located in CBD,
- 100% green time.

These factors are then modified according to the actual condition of the intersection. Once these parameters have been identified, the delay estimate equations are applied to the intersection in order to compute the average stopped delay for each vehicle.

ASPUDI Simulation Program

The computer program used to simulate the SPUDI interchanges is the Actuated Single Point Urban Diamond Interchange (ASPUDI) program that has been developed for analysis of SPUDIs [2]. ASPUDI is a time based GPSS-H simulation program at micro level for analyzing the traffic operation at actuated SPUDIs. Features of the model include actuated signal operation, protected and permitted left and right turn, storage lanes, car following, lane changing, gap acceptance behavior, primary and secondary queue formation and dissipation. The reliability of the model has been tested successfully by collecting delay and queue length data in the files and comparing them with the results predicted by the model.

The ASPUDI can be used to design or analyze or, to some extent, even optimize SPUDIs. This model is easy to use with user-friendly data input programs and a simple and straightforward output which is easy to understand and use. Specific applications include, but are not limited to, the following:

1. The model can be used to analyze and design the geometric configuration of a SPUDI for existing or predicted future traffic volumes by varying the number of lanes, curb radius, length of storage lanes, and other geometric features in each simulation run. Different signal timing plans can be tested to obtain an acceptable level of service in terms of stopped delay, total delay, average speed and maximum queue length;
2. The model can be used to design the length of storage lanes. Once the traffic volume, number of lanes and signal timings have been decided, the system can be analyzed by assuming full length lanes. Maximum queue length for the turning lanes, as given by the model, can be used to determine the length of the storage lanes;
3. The model can also be used to check the adequacy of the length of existing storage lanes. It may so happen that the turning volumes have increased over the years and the capacity of the storage lanes may often be exceeded. If this model is used to simulate traffic for existing geometric, traffic and signal data, it will issue a warning whenever a storage lane is full and a turning vehicle blocks a through lane;
4. The model can be used to analyze a SPUDI for future traffic volume and make decisions if the SPUDI needs improvements in the near future;
5. The model can be used to find the capacity of an existing SPUDI by making simulation runs with increasing input traffic volume in each run for the same geometric and signal data until the throughput is more or less constant;
6. It can be used to evaluate different signal phases and cycle lengths for given geometric and traffic data;
7. It can be used to evaluate and compare the left turn movement characteristics with left turn controls, such as permitted, protected and permitted/protected phasings;
8. The model can be used to evaluate and compare the right turn movement characteristics with and without right-turn-on-red control.

The data needed for the analysis of these intersections is divided into three sections: Geometric data, traffic data and signal data. This data is provided by the user in three user-friendly interactive sub-programs: GEODATA, TRAFDATA and ASIGDATA. These three programs create the necessary input files required by the ASPUDI program.

Geometric Data

GEODATA is the program used to create the input file that contains the geometric data for the ASPUDI. The geometric features that are input for the ASPUDI include:

- Number of legs at the intersection,
- Number of inbound and outbound lanes on each leg,
- Lane types,
- Lane configurations,
- Left and right exclusive lane storage lengths,
- Number of lanes,
- Percent grade,
- Curve radii,
- Median widths.

Traffic Data

TRAFDATA is the program used to create the traffic data input file for the ASPUDI. This program asks the user to input the total hourly volume and percent of heavy vehicles for each traffic movement allowed in the geometric configuration, including all left, right and through movements for each leg. In addition, the program requires an estimate of approach speed and minimum headway.

Signal Data

ASIGDATA is the program that creates the required input file that identifies the signal parameters for the ASPUDI program. Signal input data include:

- Number of phases,
- Movements allowed,
- Minimum and maximum green time,
- Absolute maximum green time,
- Yellow plus all red time,
- Extended time for each phase.

ASPUDI Main Program

ASPUDI is the main program which reads the input files created using the above mentioned input file programs and uses that data to simulate the traffic conditions at SPUDIs with actuated signal operations. The vehicle generation method, car following, lane changing module, queue formation/dissipation and left and right turn logic in the ASPUDI attempt to predict the operating conditions of the intersection. The program also uses a revised version of the advanced design controller method described in the Manual of Traffic Signal Design [9] to simulate actuated signal timing procedures. In this procedure, a minimum green time for each phase is assigned. As long as there is a call on a conflicting phase and there is no vehicle on the green phase to extend the green, the green time would equal the minimum green interval. At the end of the minimum green time, the controller checks whether a vehicle is detected during the minimum green time. If the time between the last vehicles detection and the end of the minimum green time is less than the extended time, the minimum green time is extended by the difference between these two times. If other vehicles are detected during the extended time interval, the timer is reset and timing of a new extended interval begins. This process is repeated for each new detection until a preset maximum is reached after a vehicle places a call for another phase or until the gap between detections is greater than the extended interval. To avoid such hazards as a system malfunction, an absolute maximum green time is considered and the total green time for each phase in each period cannot exceed this amount.

In this study, the CDI intersections data is used as inputs to the ASPUDI program. The results are then compared with the HCM delay estimates to determine which configuration would produce less average delay per vehicle.

DATA COLLECTION

In order to have a real world base for comparison between CDI and SPUDI, two existing CDIs were chosen. The idea was to find out what would happen in terms of the total delay under the current traffic conditions if there were SPUDIs in those locations instead of CDIs. To perform this analysis, data was collected at each location. This included traffic volume, turning movements, signal timing data and geometric data. Traffic volume and turning movement data were collected in the field by four groups of students from the University of Maryland using Jamar traffic collection devices. The data was collected during both A.M. and P.M. peak hours during a weekday, for the I-270 & Old Georgetown Road, and the I-495 & Old Georgetown Road intersections that are both CDIs in Montgomery County, Maryland. The Aggregated morning and afternoon volume data for the I-270 & OGR and the I-495 & OGR intersections are shown in Tables 1 to 4, respectively.

The Maryland State Highway Administration (SHA) was contacted, in order to provide the signal timing and geometric data needed for each intersection. The geometric data was obtained from as-built

Table 1. Traffic data for the CDI at the Old Georgetown Road and I-495 during the A.M. peak.

Leg	**Volume (VPH)**			**Heavy Vehicle (%)**		
	Left	**Through**	**Right**	**Left**	**Through**	**Right**
1	265	1	441	5	-	5
2	352	1375	472	5	5	5
3	271	1	362	5	-	5
4	183	560	140	5	5	5

Table 2. Traffic data for the CDI at the Old Georgetown Road and I-495 during the P.M. peak.

Leg	**Volume (VPH)**			**Heavy Vehicle (%)**		
	Left	**Through**	**Right**	**Left**	**Through**	**Right**
1	256	1	159	5	-	5
2	465	990	260	5	5	5
3	135	1	215	5	-	5
4	307	1709	192	5	5	5

Table 3. Traffic data for the CDI at the Old Georgetown Road and I-270 during the A.M. peak.

Leg	**Volume (VPH)**			**Heavy Vehicle (%)**		
	Left	**Through**	**Right**	**Left**	**Through**	**Right**
1	128	0	768	5	-	5
2	487	2439	211	5	5	5
3	483	0	445	5	-	5
4	325	1111	169	5	5	5

Table 4. Traffic data for the CDI at the Old Georgetown Road and I-270 during the P.M. peak.

Leg	**Volume (VPH)**			**Heavy Vehicle (%)**		
	Left	**Through**	**Right**	**Left**	**Through**	**Right**
1	243	4	437	5	-	5
2	759	737	258	5	5	5
3	303	0	251	5	-	5
4	688	2083	493	5	5	5

Table 5. Geometric data for the CDI at the Old Georgetown Road and I-495.

Leg	Inbound Lanes		Outbound Lanes	% Grad.	Median Width	Left Turn Pocket		Right Turn Pocket	
	Number	Type				Length	No. of Lanes	Length	No. of Lanes
1	3	L,LT,R	1	-2.5	-	0	0	0	0
2	4	T,T,T,R	3	0.0	16	0	0	0	0
3	3	L,LT,R	1	-4.0	-	0	0	0	0
4	4	T,T,T,R	3	0.0	16	0	0	0	0
5	4	L,T,T,T	3	0.0	4	200	1	0	0
6	4	L,T,T,T	3	0.0	4	200	1	0	0

1- Curb radius for curb 1 to 8 are: 250, 5, 30, 171, 30, 170, 284, 2 ft.
2- Length of the connecting link is: 550 ft.
3- L = left turn, T = through, LT = left and through, R = right turn

Table 6. Geometric data for the CDI at the Old Georgetown Road and I-270.

Leg	Inbound Lanes		Outbound Lanes	% Grad.	Median Width	Left Turn Pocket		Right Turn Pocket	
	Number	Type				Length	No. of Lanes	Length	No. of Lanes
1	2	LT,R	1	1.5	-	0	0	0	0
2	4	T,T,T,R	3	0.0	16	0	0	0	0
3	2	LT,R	1	9.5	-	0	0	0	0
4	4	T,T,T,R	3	0.0	16	0	0	0	0
5	4	L,T,T,T	3	0.0	4	150	1	0	0
6	4	L,T,T,T	3	0.0	4	150	1	0	0

1- Curb radius for curb 1 to 8 are: 170, 8, 7, 170, 13, 170, 170, 8 ft.
2- Length of the connecting link is: 375 ft.
3- L = left turn, T = through, LT = left and through, R = right turn.

drawings dated 1992 (I-270 & Old Georgetown Road) and 1965 (I-495 & Old Georgetown Road) and provided the required interchange configurations, curve radii and slope information. The geometric data is summarized in Tables 5 and 6 and Figure 2. Montgomery County Department of Transportation was also contacted to provide the necessary signal timing data.

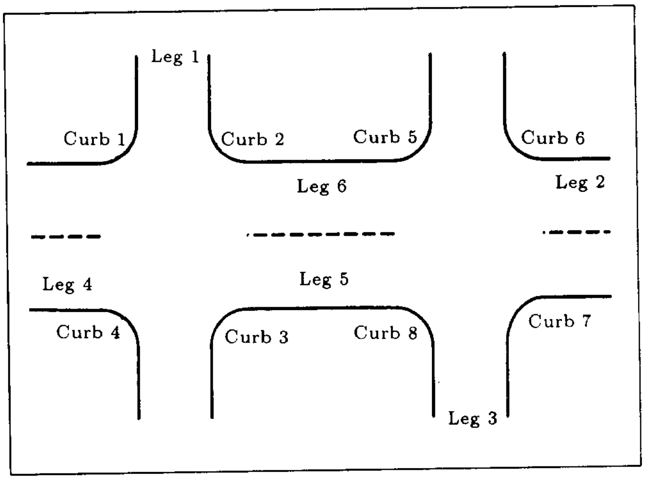

Figure 2. Intersection naming conventions for geometric data.

ANALYSIS AND CONCLUSION

Table 7 compares the results of estimated delay for the SPUDI and CDI, based on the real data. The table clearly shows that the SPUDI interchange produces a delay saving over the CDI design. In this instance, the total delay avoided by the SPUDI is approximately one-half of the delay produced by the CDI. In addition, if one examines the individual delay estimates obtained from the HCS software, it is noticed that the estimate given for the I-270 & Old Georgetown interchange during the AM peak period is considerably higher than any other delay estimate at any peak period. This may be due to limitations of the functions and mathematical relations that are provided and used by the HCS. These functions and mathematical relations may work well in a particular range, however, they may be susceptible to breakdown near the boundaries of these ranges and, hence, under or over estimation may occur.

Even if one considers that this is an unusually high estimate and removes this interchange data during the A.M. peak, a total delay of 92.37 hours and 95.42 hours is obtained from the ASPUDI simulation and

Table 7. Comparison of the results of the delay estimates for the SPUDI and the CDI.

Intersection	ASPUDI Delay Estimate for SPUDI (Hours)	HCS Delay Estimate for CDI (Hours)
I495 & Old Georgetown RD - A.M.	16.81	31.16
I495 & Old Georgetown RD - P.M.	26.40	25.31
I270 & Old Georgetown RD - A.M.	29.42	145.50
I270 & Old Georgetown RD - P.M.	49.16	38.95
Total Delay	121.79	240.92

the HCS estimation, respectively. This is still a total delay saving of more than three hours if the SPUDI configuration is used (a saving of about 3.21%). For this reason and because of other savings (e.g. in land), it is concluded that the SPUDI interchange is a more cost effective design. Furthermore, it is very likely that an even greater amount of delay saving would be attained in the real world, since many of the parameters used for the ASPUDI program were based upon the configuration of the CDI interchange and do not actually reflect an optimal design for a SPUDI. This lack of optimal parameters is most evident in the parameters that reflect signal timing, interchange geometrics and traffic flow conditions at SPUDI intersections.

Signal timing plans for the SPUDI intersections are based on the actual timing plans that were in effect for the CDI intersection at the time of data collection. These timing plans were assumed to be optimal for the CDI. Due to the difference in operating conditions between the two intersections, it is very likely that an optimum timing plan for a SPUDI intersection under the same traffic volume conditions would be different from that of the CDI. Since the average vehicle delay at a signalized intersection is a function of its signal timing (in addition to other parameters), it is very likely that an increase in delay saving would be obtained from a completely new timing plan that reflects an optimum cycle length and phasing for the SPUDI interchange. Even though the timing for each phase from the CDI was used in the development of a timing plan for the SPUDI, a different phase sequence was used for the SPUDI timing plan to reflect the difference in the geometrics and the controller types from the CDI to the SPUDI. In reality, a new timing plan, including phase sequence, should be developed for the SPUDI intersection that is completely independent of the initial CDI.

Intersection geometrics present in a CDI are significantly different from the geometrics of a SPUDI interchange. In order to avoid the complete re-design of a CDI into a SPUDI intersection, several geometric factors from the CDI were used to simulate the SPUDI. These included the curve radii, left turn bay conditions and the geometric configurations of the approach lanes that needed to be modified in order to reflect the consolidation of two intersections from the CDI into a single point for the SPUDI. In reality, the optimal geometric configuration for a SPUDI interchange is vastly different from the optimum geometrics found at a CDI.

The last primary difference in optimal design between these two types of intersection reflects how a vehicle is permitted to maneuver, based upon the physical design of each roadway. For example, field observations of traffic flow showed that a small number of vehicles operating within the constraints of the CDI intersection were able to perform U-turns and even through movements from one off-ramp to an adjoining on-ramp in the same direction. These movements are not possible (or extremely unlikely) to occur given the optimal geometric configuration of a SPUDI interchange. For this reason, these types of movement were omitted from the ASPUDI simulation program.

Even given the above limitations to the ASPUDI program, the SPUDI configuration is still considered to be the most economical arrangement, in terms of total delay. In fact, the very lack of optimum conditions for the SPUDI intersection adds to its credibility in providing an interchange that produces less total vehicle delay. More data from several other intersections are needed to conclude whether or not the differences in total delay between the CDI and the SPUDI are statistically significant. This is left for future research. However, the ASPUDI simulation model was run for several examples. Comparing the results of the simulation with corresponding values obtained from analyzing CDI with HCS, it was found that in 80% of the examples, the ASPUDI produced a delay saving over the CDI design. In particular, in the examples where there is high traffic flow near the capacities, ASPUDI is always superior, in terms of shorter delay time.

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