A Multi-stage System for Planning Analysis and Signal Design of Diverging Diamond Interchange

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ABSTRACT

As one of the most popular unconventional interchange designs, diverging diamond intersection (DDI) has received increasing attention over the past decade. This study, responding to the needs, has produced a reliable and convenient system for traffic engineers to perform operational analysis of DDI. The entire system comprises three modules for planning analysis, signal optimization, and operational evaluation. At the planning stage, this system presents a set of empirical equations for engineers to compute the overall interchange delay and identify the potential queue spillback locations in a DDI design. The second module aims to provide the optimal signal plans to prevent the potential queue blockage. This module is unique in its consideration of the interdependent relations between queues at a DDI's closely-spaced intersections, and the impacts by both geometrical constraints and traffic volumes. Given the traffic volumes, geometrical features, and signal timings, the system's third module provides users to link a VISSIM-based simulation model to estimate the resulting traffic queues and interchange delays. Numerical analysis with four real-world DDI designs has revealed the effectiveness of the proposed system.

INTRODUCTION

Diverging Diamond Interchange (or DDI), one of the new unconventional intersection designs, has received increasing attention in recent years due to its cost-effectiveness over a traditional diamond interchange design. The key logic of DDI is to provide efficient navigation for both left-turn and through movements between highway ramps, and to accommodate left-turning movements onto the arterial without using a left-turn bay. As shown in FIGURE 1, the reverse operations of the through traffic between two ramp terminals in a DDI design allow its left-turn traffic flows from the freeway off-ramps to the opposing flows at each subintersection *(1)*. Its right-turn movements from the cross street to the ramps take place at these two ramp terminal intersections. With such an assignment of flow movements, a DDI design can significantly reduce the number of traffic conflict points, and thus provide a safe and cost-economic environment.

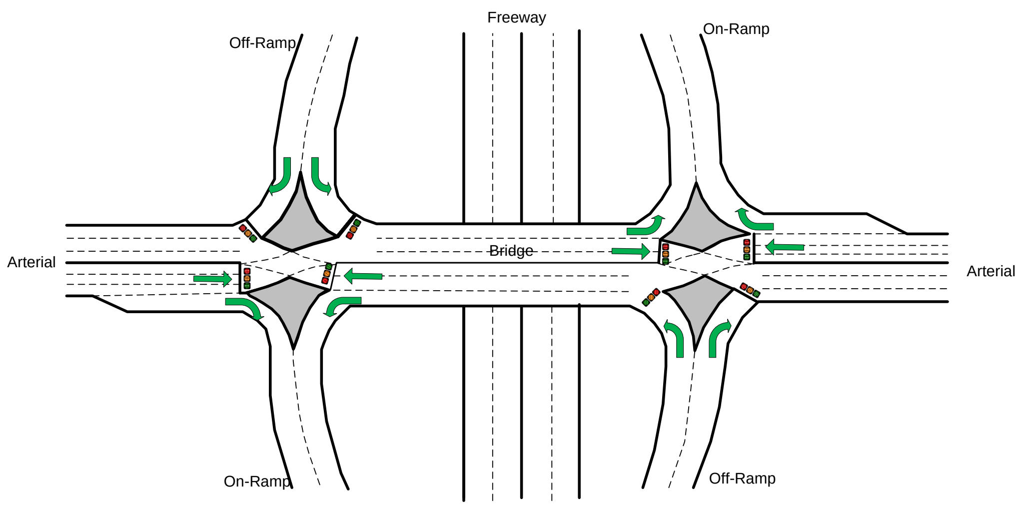

FIGURE 1 Geometric Layout of DDI design

Over the past decade, some DDIs have been implemented in US, and traffic community has increased its interest in investigating the strengths and deficiencies of such designs over a conventional interchange *(2,3)*. For instance, Chlewicki *(4)* used Synchro and SimTraffic to analyze the delays in a DDI design and compared its performance to a conventional interchange under various demand levels. Using the conventional diamond interchange as the basis for comparison, his study concluded that a properly designed DDI can reduce about 60 percent of the total intersection delay, and 50 percent of total number of stops. Applying the same simulation tools, Speth *(5)* conducted a similar analysis of DDI and conventional diamond interchanges and also reached the same conclusions, especially regarding the average delay and average number of stops per vehicle. Bared, et al. *(6)* extensively investigated the performance of DDI at five volume levels and under two geometric conditions. Their research results, based on simulation experiments, indicated that a DDI can outperform a conventional diamond interchange, particularly at a high volume level. The general conclusion is that, a DDI design can accommodate higher volumes for all movements, especially for left-turn flows, than a conventional diamond interchange. They also concluded that converting an interchange into a six-lane DDI is financially more beneficial than a design of widening the bridge. Considering the possible internal queue spillback, Xu *(7)* developed a method to calculate the control delay of DDI, using an analytical model on both internal movements and external movements.

Note that despite the increasing interest, existing DDI studies are quite limited and focus mainly on exploring its benefits using microscopic traffic simulations. Some critical issues for DDI proponents to address include: a) development of a convenient and effective tool for evaluating the performance of a preliminary DDI design, such as identifying potential queue spillback locations and their impacts on the overall delay; b) design of a systemic procedure to optimize a DDI's geometric parameters based on different demand patterns; and c) optimization of all signal plans, including their timings and offsets. This study, despite its exploratory in nature, intends to address all these critical issues, and to provide some preliminary results for the community for application and future extension.

This paper is organized as follows: Section 2 presents the methodology of the proposed models and introduced the flow chart of the multi-stage system. Section 3 discusses the geometric features of DDI and illustrates a set of empirical equations for planning applications. Sections 4 and 5 detail the development of a signal optimization model and its evaluation with field data. Some key findings and conclusions are summarized in the last section.

A MULTI-STAGE SYSTEM FOR DESIGN OF DDI

The MUID (Maryland Unconventional Intersection Design) system, jointly sponsored by the Maryland State Highway Administration and University of Maryland, aims to provide a comprehensive tool for engineers to design unconventional intersections and perform necessary evaluation. FIGURE 2 illustrates the primary functions for a three-stage DDI design:

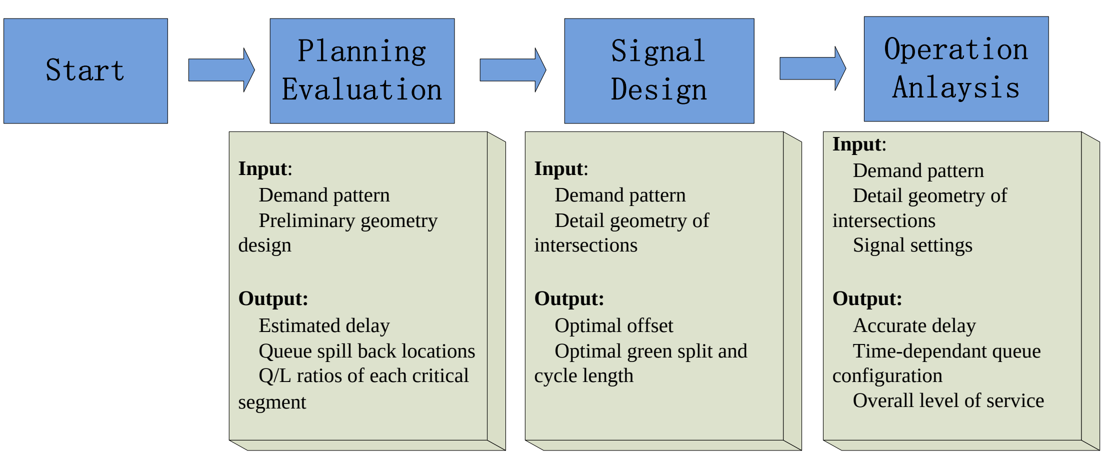

FIGURE 2 Flowchart of the MUID system

At the planning stage, this system offers a set of empirical equations for engineers to compute the overall interchange delay and to identify potential queue spillback locations in a DDI design. Some recommendations on revising the initial design would be provided at the end of this stage. Those empirical equations are built with regression models, based on extensive simulation experiments generated by VISSIM with its key parameters calibrated by field data. Note that due to the interdependent relations between queues at DDI's closely-spaced intersections, the impact of both geometrical constraints and traffic volume need to be incorporated in the signal optimization process. Thus, the focus of the proposed system at its second stage is to help traffic professionals develop the optimal signal timing plan, so as to synchronize traffic flows at those two intersections and to prevent any potential queue blockage. Based on the recommended geometric features and signal plans, the system subsequently offers a function to employ a VISSIM-based simulation model for users to assess the resulting queues and delays. The key logic and mathematical models embedded in each module are presented in the remaining sections.

DEVELOPMENT OF PLANNING MODEL

To perform a preliminary geometry design of DDI, traffic engineers first need to decide the length of each link and turning bay. Insufficient length for any link or bay will cause spillback, and consequently increase the overall delay. Therefore, our planning module aims to estimate the potential queue length at each link/bay and to identify potential bottlenecks. The procedures used to develop essential empirical models include the following steps:

Step 1: Identifying all factors contributing to the total DDI delay, including external factors such as traffic demand, and internal factors such as intersection geometric features;
Step 2: Generating a comprehensive set of data set with all identified factors for simulating analysis;
Step 3: Deriving the quantitative relationships between intersection delays and contributing factors;
Step 4: Estimating the impact of queues on the overall intersection performance and developing a set of statistical models for queues length prediction at each critical location within a DDI.

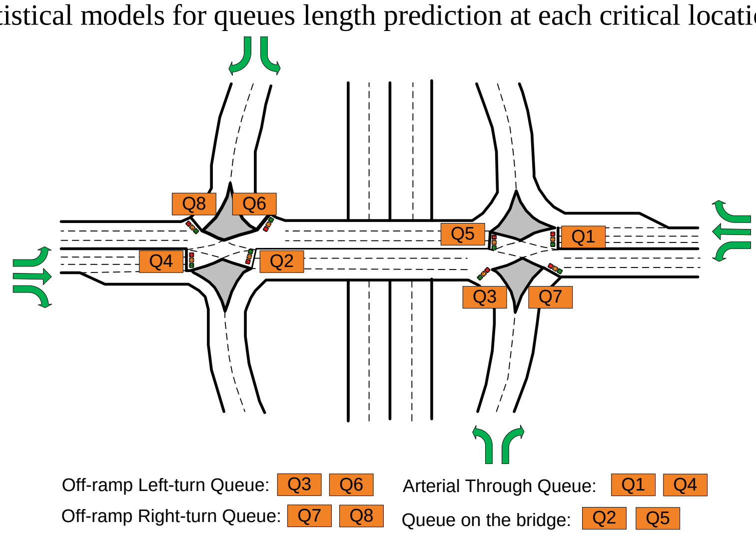

FIGURE 3 Spatial distributions of all potential queues at DDI

FIGURE 3 shows the spatial distribution of traffic queues in a DDI design. Due to the interdependent nature of traffic queues between those links, any spillback at one location may propagate the congestion to neighboring locations and thereby degrading the available interchange capacity. Hence, understanding the relationship between the queue development in each bay and its contributing factors is essential in evaluating the performance of a DDI design.

After using one set of field data to calibrate the driver behavior parameters in VISSIM, this study employed simulation experiments to generate the queue and delay data for a DDI under various demand conditions. Experimental scenarios are generated randomly with respect to changes in demand patterns, link lengths, and number of lanes per link. Our extensive exploratory analyses have revealed that overall congestion level and the distribution of queues are the two most critical factors. Hence, to condense the model formulation, we select the CLV (Critical Lane Volume) to represent the intersection congestion level and **QL ratio** (queue length / link length) to reflect the link queue level. Equation (1) is the proposed DDI delay model that reflects the collective impact of the volume at each subintersection and the queue length occurred in each link.

$$
\begin{aligned}
& Log(delay) = 2.549 + 0.514\frac{X_w}{1-X_w} + 0.149\frac{X_e}{1-X_e} + 0.206\rho_1 + 0.213\rho_2 \\
& \text{t value: } (3.87) \quad (13.98) \quad (21.37) \quad (16.58) \quad (3.96) \\
& \quad + 0.253\rho_3 + 0.212\rho_4 + 0.197\rho_5 + 0.251\rho_6 + 0.131\rho_5 + 0.127\rho_6 \qquad (1) \\
& \quad (5.78) \quad (54.31) \quad (42.64) \quad (5.98) \quad (14.31) \quad (7.45) \\
& R^2 = 0.947, \text{ sample size N: 1200}
\end{aligned}
$$

where, X_w (X_e) is the degree of saturation of the west (east) Intersection and can be approximated with the ratio of CLV and its saturation flow rate; ρ_i denotes the QL ratio at each critical location, as shown in FIGURE 3.

Note that equation (1) implies that a DDI will experience a large delay if any of its subintersections' CLV approaches its maximum level. Also, those regression coefficients indicate that the QL ratio on the bridge contributes most to the average DDI delay.

In view of the high correlation between QL ratios and average intersection delay, this study further calibrates a set of queue models to estimate critical link queue length. With these estimation models, one can evaluate the preliminary geometry design of a DDI without using time-consuming and complex simulation tools, and to identify if those designed links are sufficient to store potential traffic queues.

The results of extensive simulations experiments indicate that the following factors may significantly impact the formation and dissipation of traffic queues: the approaching demand to the target approach, the green time ratio, and the intersection congested level measured with the critical lane volume (CLV). At the planning stage, for convenience, the green ratio is determined with the target flow and its opposing flows. Hence, this study has calibrated the following queue models:

Type 1 Queue Model $(\mathbf{Q_1}, \mathbf{Q_4})$

$$\text{Queue}=0.64\frac{D_t(1-G_t)s}{s-D_t}+5.14\left(\frac{D_t}{s\text{-}CV}\right)^2+1.72e^{4\rho_d} \tag{2}$$

t value: (37.9) (76.5) (13.7)

$R^2=0.892$, Sample size N: 1200

where,

D_t: Approaching volume (through and left-turn volume from the upstream arterial);
G_t: Estimated green time ratio for the target movements;
CV: The critical lane volume at the target intersection;
ρ_d: QL ratio at the downstream link;
s: The critical lane capacity.

Type 2 Queue Model $(\mathbf{Q_3}, \mathbf{Q_6}, \mathbf{Q_7}, \mathbf{Q_8})$

$$\text{Queue}=0.73\frac{D_t(1-G_t)s}{s-D_t}+5.54\left(\frac{D_t}{s\text{-}CV}\right)^2 \tag{3}$$

t value: (31.2) (14.7)

$R^2=0.871$, Sample size N: 1200

where,

D_t: Approaching volume (left-turn or right-turn off-ramp volume);
G_t: Estimated green time ratio for the target movements;
CV: The critical lane volume at the target intersection;
s: The critical lane capacity.

Type 3 Queue Model $(\mathbf{Q_2}, \mathbf{Q_5})$

$$\text{Queue}=0.61\gamma\frac{(D_t+D_l)(1-G_t)s}{s-(D_t+D_l)}+5.62\left(\frac{D_t+D_l}{s\text{-}CV_n}\right)^2 \tag{4}$$

t value: (47.8) (53.1)

$R^2=0.831$, Sample size N: 1200

where,

D_t: Approaching through volume from the upstream arterial;
D_l: Approaching left-turn off-ramp volume;
G_t: Estimated green time ratio for the target movements;
CV: The critical lane volume at the target intersection;
s: The critical lane capacity.

FORMULATION OF SIGNAL OPTIMIZATION MODEL

Due to the unique geometry features, a DDI typically has two signalized intersections, controlled with two-phase signals. Compared with conventional interchanges, a DDI allows for a relatively shorter cycle length at its intersections. Left-turn and right-turn volumes from the off-ramps are preferably operated under signal control due to the sharp turning. FIGURE 3 illustrates an example of phasing schemes for a typical DDI design.

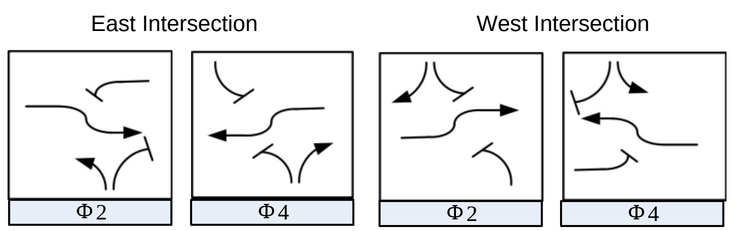

FIGURE 4 Signal phasing for DDI operating under separate controllers

To optimize the signal design for a DDI, one shall concurrently addresses the following three issues: green split at each intersection, cycle length, and offset. This study proposes two optimization models for such need. The first one is used for optimal green split, whereas the second one will yield the optimal offset and cycle length.

Green Split Optimization

One important issue for signal design is to maximize the capacity of an intersection given the geometric layout *(8,9)*. Based on the assumption that traffic demand matrix can be multiplied with a common flow multiplier μ to represent the maximum amount of increased volume that would still allow the intersection to perform reasonably well *(10)*, the optimization problem can be converted as an issue of determining the maximum multiplier μ_{max}.

With the increased demand, the flow conservation constraints could be set as:

$$q_j = \sum_i \mu \beta_{ij} Q_i \qquad \forall i, j \tag{5}$$

where $Q = \{Q_i,\ i \in N_T\}$ denotes the traffic demand to the entire DDI; q_j *is* the assigned traffic flow (multiplied by μ) on lane group j; a set of binary variables $\{\beta_{ij}\}$ are used to indicate the resulting traffic assignment:

$$\beta_{ij} = \begin{cases} 1 & \text{if flow } i \text{ is assigned to } j \\ 0 & \text{otherwise} \end{cases}$$

Note that due to the unique geometry features of DDI, the left-turn on-ramp volume may be allowed to move continuously on the bridge without any signal delay, as shown in FIGURE 5(A). However, for those DDIs with no "left-turn only lane", the through traffic queue may block the entry of the on-ramp vehicles, as indicated in FIGURE 5(B).

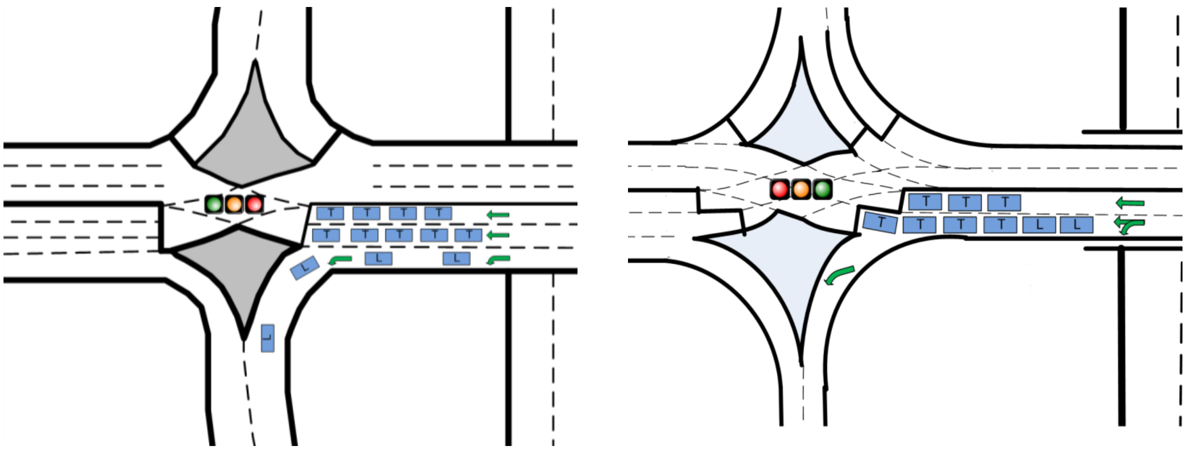

FIGURE 5(A) DDI with "left-turn only lane" **FIGURE 5(B) DDI without "left-turn only lane"**

FIGURE 5 Illustration of DDIs with different geometry design

To account for those DDIs without a "left-turn only lane", the left-turn volume is multiplied by a parameter γ_i and equivalently converted to through volume during the optimization process. The value of γ_i is determined by the congestion level of the intersection.

Based on the same assumption as mentioned above, the following constraints should be satisfied to ensure that the degree of saturation in each movement is below the maximum acceptable limit.

$$q_j \le s_j \sum_m \sum_n \alpha_{mnj} g_{mn} \quad \forall j$$

(6)

where, s_j is the saturation flow rate at lane group j and g_{mn} denotes the assigned g/c ratio for phase *m* at intersection *n* while vehicles in lane group j have the right of way. The parameter $\{\alpha_{mnj}\}$, is adopted to represent the phase plan:

$$\alpha_{mnj} = \begin{cases} 1 & if\ j\ obtains\ its\ right\ of\ way\ in\ phase\ m\ at\ intersection\ n \\ 0 & otherwise \end{cases}$$

The green duration for each traffic group is subjected to a minimum value, and these constraints are set as follows:

$$g_{\min} \le g_{mn} \le 1 \quad \forall m, n \tag{7}$$

Also, for each intersection,

$$\sum_{m} g_{mn} = 1 \quad \forall n \tag{8}$$

Thus, one can present the optimization model as follows:

Maximize μ (9)

Subject to: constraints in (5) - (8).

This LP optimization model could be solved efficiently with most existing algorithms.

Note that the entire interchange is under an over-saturation traffic condition if the optimal result indicates $\mu_{max} < 1$.

Synchronization of Intersections

In addition to optimizing of the green ratios, another issue for the DDI signal design is how to determine the offset between two subintersections. The synchronization of intersections has been discussed extensively in the literature, and the MAXBAND (9~10) model is one of the most efficient one to coordinate the signals along an arterial. Hence, this study employs the core logic of MAXBAND to model the signal coordination, but focus on facilitating the heavy left-turn flows.

More specifically, instead of considering the green band of those arterials through volumes only, all movements in a DDI would be taken into account in this model. The green band of each movement is shown in FIGURE 6:

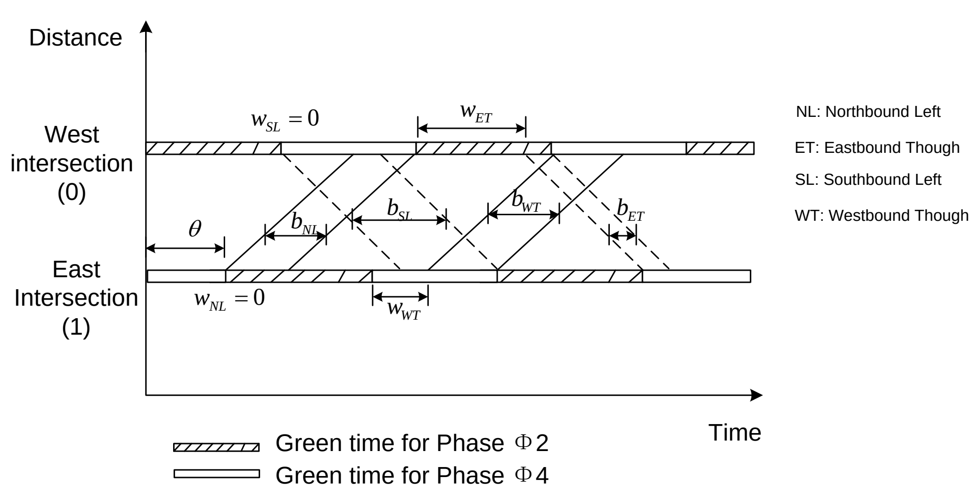

FIGURE 6 The green band of each movement in DDI

where, θ is the offset; b is the bandwidth, t_{in} (t_{out}) is the travel time between subsections, and ξ is the reciprocal of cycle length, and $\xi = 1/C$.

For the inbound direction (East to West), the relationship between travel time and bandwidth is given by:

$$\theta + w_{NL} + b_{NL} + t_{in} \leq N \tag{10}$$
$$\theta + g_E + w_{WT} + b_{WT} + t_{in}\xi \leq (N+1) \tag{11}$$
$$w_{NL} + b_{NL} \leq g_E \tag{12}$$
$$w_{WT} + b_{WT} \leq 1 - g_E \tag{13}$$

For the outbound direction (West to East), the constraints are given by:

$$w_{ET} + b_{ET} + t_{out} \leq \theta + N \tag{14}$$
$$\theta + g_W + w_{SL} + b_{SL} + t_{out}\xi \leq \theta + N \tag{15}$$
$$w_{ET} + b_{ET} \leq g_W \tag{16}$$
$$w_{SL} + b_{SL} \leq 1 - g_W \tag{17}$$

Note that cycle length should also be determined by the intersection's congestion level which is neglected in the MAXBAND model. To minimize the delay, Webster *(13)* formulated an equation for cycle length selection. In this study, we set a constraint for the cycle length optimization as follows:

$$C_{webster} \leq C \leq C_{webster} + \Delta C \tag{18}$$

where, ΔC is a given parameter.

Thus, the objective function is:

$$Max: \sum_{i \in V} \varphi_i b_i \tag{19}$$

Subject to: constraints (10)~(18).

where, $\sum_{i \in V} \varphi_i = 1$ and φ_i is the weight factor. In this study, φ_i is proportional to the demand levels.

The above optimization model with the logic of MAXBAND is a LP problem and could be solved efficiently with existing methods.

CASE STUDY

Due to both operational efficiency and potential safety improvements that a DDI can offered, highway agencies are increasingly interested in constructing such interchanges. Some of those have been successfully operated in the USA. This section, presents the application of our developed multi-stage design system at the following DDI locations:

Case 1: National Ave @ Springfield, MO; Case 2: Bessemer St. @ US 129 Alcoa, TN
Case 3: Dorsett Road @ MD heights, MO; Case 4: MO 13 @ I-44, Springfield, MO

The bird view of each DDI design is represented in FIGURE 7.

FIGURE 7(A): National Ave @ Springfield, MO

FIGURE 7(B): Bessemer St. @ US 129 Alcoa, TN

FIGURE 7(C): Dorsett Road @ MD heights, MO

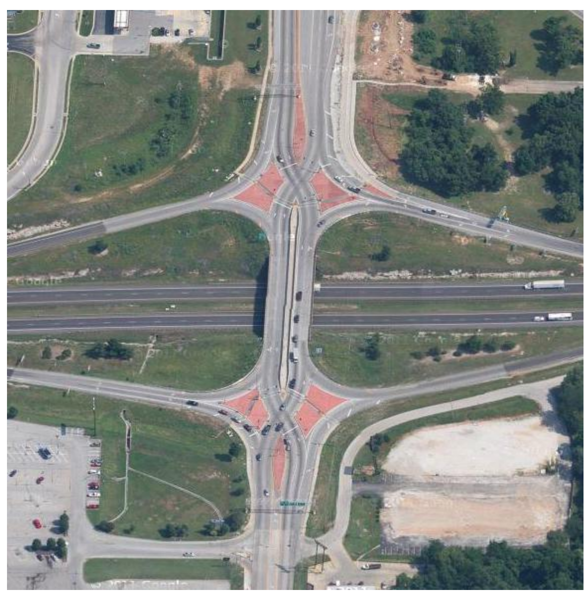

FIGURE 7(D): MO 13 @ I-44, Springfield, MO

FIGURE 7 Bird view of the four constructed DDI in USA

Notably, a total of eight critical locations can be identified in one DDI design, including two off-ramp left-turn links, two off-ramp right turn links, two arterial through links, and two bridge links. For convenience of discussion, each critical link is numbered in FIGURE 8, and the corresponding geometric parameters of each DDI case are summarized in TABLE 1.

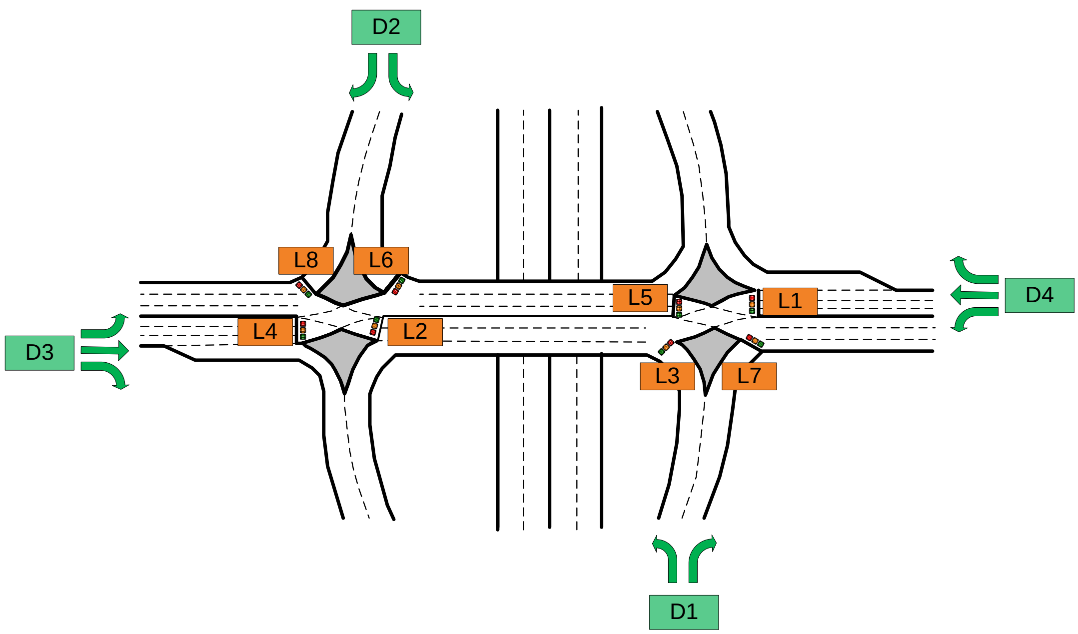

FIGURE 8 Geometric parameter index of DDI

TABLE 1 Geometric Parameters of the Four DDI Cases

Case		L1	L2	L3	L4	L5	L6	L7	L8
1	Link length (ft)	400	636	900	475	636	790	900	790
	# of Lanes	3	3	2	3	3	1	1	2
	Left-turn Only Lane	-	No	-	-	No	-	-	-
2	Link length (ft)	1200	620	350	1050	620	1400	350	1400
	# of Lanes	2	1	1	1	1	1	1	1
	Left-turn Only Lane	-	Yes	-	-	No	-	-	-
3	Link length (ft)	650	450	510	1500	450	400	510	400
	# of Lanes	3	2	2	3	2	2	2	2
	Left-turn Only Lane	-	Yes	-	-	Yes	-	-	-
4	Link length (ft)	260	450	430	600	450	460	430	460
	# of Lanes	2	2	1	2	2	1	1	1
	Left-turn Only Lane	-	No	-	-	No	-	-	-

The volume data from field survey was provided by the US Federal Highway Administration. In each case, AM peak hour and PM peak hour demands are represented in the TABLE 2.

TABLE 2 Collected Volume Data of the Four DDI Cases (veh/hr)

Case	Time of Day	D1		D2		D3		D4	
		L	R	L	R	L	T	L	T
1	AM-Peak	1031	374	198	602	153	303	126	995
	PM-Peak	247	297	214	244	372	951	296	574
2	AM-Peak	42	293	28	108	108	173	105	105
	PM-Peak	54	351	82	64	215	173	400	226
3	AM-Peak	1091	269	164	785	420	194	570	386
	PM-Peak	333	521	249	347	441	960	285	458
4	AM-Peak	160	270	375	335	415	970	95	430
	PM-Peak	165	180	290	335	250	845	120	1070

Based on the provided geometric parameters and traffic demand patterns, the proposed planning models have been applied to evaluate the geometry design of each constructed DDI. The results are represented in TABLE 3.

TABLE 3 Geometry Design Evaluation by the Planning Model

Case	TOD		L1	L2	L3	L4	L5	L6	L7	L8
1		Link Length (ft)	400	636	900	475	636	790	900	790
	AM	Max Queue (ft)	179	324	229	74	94	51	204	186
		QL Ratio	0.45	0.51	0.25	0.16	0.15	0.06	0.23	0.24
	PM	Max Queue (ft)	131	145	41	149	199	110	153	37
		QL Ratio	0.33	0.23	0.05	0.31	0.31	0.14	0.17	0.05
2		Link Length (ft)	1200	620	350	1050	620	1400	350	1400
	AM	Max Queue (ft)	29	40	19	74	83	14	105	29
		Queue/Link Ratio	0.02	0.06	0.05	0.07	0.13	0.01	0.30	0.02
	PM	Max Queue (ft)	104	66	24	136	106	37	136	20
		QL Ratio	0.09	0.11	0.07	0.13	0.17	0.03	0.39	0.01
3		Link Length (ft)	650	450	510	1500	450	400	510	400
	AM	Max Queue (ft)	159	286	219	98	45	22	67	246
		QL Ratio	0.24	0.64	0.43	0.07	0.10	0.05	0.13	0.61
	PM	Max Queue (ft)	130	171	41	191	175	53	157	65
		Queue/Link Ratio	0.20	0.38	0.08	0.13	0.39	0.13	0.31	0.16
4		Link Length (ft)	260	450	430	600	450	460	430	460
	AM	Max Queue (ft)	153	192	30	269	258	230	181	109
		QL Ratio	0.59	0.43	0.07	0.45	0.57	0.50	0.42	0.24
	PM	Max Queue (ft)	319	342	63	287	366	118	79	168
		QL Ratio	**1.23**	0.76	0.15	0.48	**0.81**	0.26	0.18	0.37

Based on the results in TABLE 3, most designed links are sufficient to storage traffic queues during both AM and PM peak hours. However, one can still observe some insufficient links in Case 4. For example, the QL ratio of link L1is over 1.0 in Case 4, which indicates queue spillback, and blockage to the downstream intersection. Also, the QL ratio (0.81) of L5 is close to 1.0 and blockages may occur at this location, due to the traffic fluctuation in real-world. To avoid such potential queue blockages, one simple way is to revise the signal settings by assigning additional green time to those congested movements. However, doing so may lead to congestion at other critical locations. Another potential remedy is to increase the number of lanes at those congested links with additional construction costs. Therefore, a rigorous cost/benefit analysis is essential to determine the best way.

The second stage of a DDI design is to optimize the signal settings for both subintersections. Some key parameters for signal optimization of those designs are given bellow:

- The free-flow speeds are set to be 40 mph;
- The lost time per cycle is given by 12s;
- ΔC is set to be 20s;
- The multiplier γ_i is 0.2 for Case 1,2 and 0.6 for Case 4.
- The minimal green time for each phase is 7s;
- Yellow time and all-red time are fixed to be 3s and 2s; and
- Saturation flow rate s is 1700 veh/h/lane for all traffic movements.

TABLE 4 Signal Optimization Result from Proposed Model and TRANSYT 14

Scenarios		Intersection	Cycle Length	Offset	φ2 Green	All-red	Yellow
Case 1	AM	East intersection	70[a] (65[b])	–	35[a](33[b])	2	3
		West intersection		45[a] (13[b])	18[a](22[b])	2	3
	PM	East intersection	55[a] (55[b])	–	27[a](25[b])	2	3
		West intersection		37[a] (6[b])	28[a](30[b])	2	3
Case 2	AM	East intersection	45[a] (55[b])	–	14[a](16[b])	2	3
		West intersection		14[a] (8[b])	24[a](34[b])	2	3
	PM	East intersection	50[a] (55[b])	–	17[a](17[b])	2	3
		West intersection		10 [a] (7[b])	24[a](29[b])	2	3
Case 3	AM	East intersection	80[a] (85[b])	–	45[a](46[b])	2	3
		West intersection		62[a] (65[b])	24[a](37[b])	2	3
	PM	East intersection	60[a] (80[b])	–	36[a](46[b])	2	3
		West intersection		45[a] (9[b])	27[a](48[b])	2	3
Case 4	AM	East intersection	75[a] (90[b])	–	51[a](61[b])	2	3
		West intersection		2[a] (4[b])	43[a](63[b])	2	3
	PM	East intersection	100[a] (110[b])	–	48[a](49[b])	2	3
		West intersection		13[a] (10[b])	41[a](39[b])	2	3

[a] The proposed model

[b] TRANSYT-14

TABLE 4 presents the optimal signal settings for each DDI case, by applying the maximum capacity model and bandwidth model proposed above. For performance comparison, the signal plans generate from TRANSYT 14 are also presented in this Table.

Also, to compare our signal plan with the one from TRANSYT-14, VISSIM is applied as an unbiased evaluator. The simulation results are represented in TABLE 5, including the MOEs of the entire intersection for each case. Note that the average results are computed over 10 simulation runs to overcome the stochastic nature of a microscopic simulation system.

TABLE 5 Operational Analysis Result of the Four DDI Cases

Scenarios	MOEs	Simulation results from VISSIM		Improvement (%)	
		AM	PM	AM	PM
Case 1	Ave. Delay (s)	19.91 [a] (22.67 [b])	20.13 [a] (21.90)	13.86%	8.79%
	Ave. number of stops	0.84 [a] (0.93 [b])	0.87 [a] (0.90[b])	10.71%	3.45%
Case 2	Ave. Delay (s)	11.86 [a] (11.93 [b])	15.92[a] (15.73[b])	0.59%	-1.19%
	Ave. number of stops	0.63 [a] (0.64[b])	0.66 [a] (0.65 [b])	1.59%	-1.52%
Case 3	Ave. Delay (s)	25.00 [a] (28.01 [b])	19.73[a] (20.37[b])	12.04%	3.24%
	Ave. number of stops	0.768[a] (0.84 [b])	0.73 [a] (0.76 [b])	8.98%	4.11%
Case 4	Ave. Delay (s)	25.43 [a] (26.78 [b])	34.69[a] (35.91[b])	5.31%	3.52%
	Ave. number of stops	0.90 [a] (0.97 [b])	0.95 [a] (1.03 [b])	7.78%	8.42%

[a] The proposed model

[b] TRANSYT-14

Based on the results in TABLE 5, for case 1, case 3 and case 4, the proposed optimization model outperforms TRANSYT-14 with respect to both average delay and average number of stops. These two models generated similar signal plans for Case 2, and the yielded indifferent MOEs. By examining the results in TABLEs 2, 4 and 5, it is interesting to note that:

- Case 1 and Case 3 received a heavy off-ramp left-turn volume during the AM peak hours, and our optimized signal offers a better intersection performance than TRANSYT-14. Specifically, our model can efficiently reduce the average number of stops, reflecting a more effective signal progression.
- For those congested scenarios, such as Case 3 and Case 4, our optimization models also outperform TRANSYT-14 with respect to both delay and number of stops.
- These two optimization models produce comparable traffic performance at uncongested scenarios such as in the AM-peak and PM-peak of Case 2.

Based on the preliminary comparison results, we can conclude that our proposed optimization model can effectively deal with those scenarios of having heavy off-ramp left-turn volumes. This is due to the fact that, our model is able to provide signal coordination to the heavy left-turn flows instead of the arterial through movements. Besides, for those designs under congested traffic conditions, our model also offers a better optimization plan than the existing software. One possible reason is that our cycle length optimization process considered both green band maximization and delay minimization objectives, which is more appropriate for the two-phase intersections such as DDI.

CONCLUSION

This study proposed a multi-Stage system for planning analysis and design of signal plans for Diverging Diamond Interchanges. Three key modules are integrated in this system: planning model, signal optimization model, and operation model. The planning model allows traffic engineers to approximate the delay of the entire DDI design, and compute the queue length at each critical location. For evaluation of the geometric design, our proposed model specifically includes QL ratio as a key variable, offering an effective and convenient way for users to identify potential queue spillback locations. The signal optimization module includes a capacity maximization model to optimize the green split, and a MAXBAND model to best select the offset and cycle length. Compared with TRANSYT 14, our signal model is more effective in dealing with congested scenarios, especially for those with a heavy left-turn volume. The proposed system also offers a convenient tool for users to use simulation tools to perform detailed analysis of a designed DDI based on various MOEs.

Despite the progress made in this study, we fully recognize that several key issues remain to be discussed. For instance, most existing studies report the operational benefits of a DDI, but a rigorous yet efficient model for cost-benefit assessment is not available for engineers to justify the construction of a DDI. In addition, safety issues in a DDI design in comparison with a conventional interchange also needs to be further investigated.

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