Development of Performance Evaluation Models for a Continuous Flow Intersection

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Abstract

Continuous Flow Intersection, known as CFI, is a type of unconventional design which receives increasing concern among transportation professionals. Many studies in recent years reported the advantage of the CFI design in terms of reducing delay and increasing capacity. However, the lack of systematical design guidelines and tools for evaluation and analysis has emerged as the main obstacle in promoting such an innovative design. This paper presents a multi-stage design framework for traffic engineers to develop a CFI intersection that can offer better level of service over the conventional intersection with acceptable cost. Since the relation between various queue types and their turning bay lengths is the primary factor that determines the effectiveness of a CFI. This study presents a set of procedures and models for traffic professionals to evaluate the V/C ratio of each critical movement in a CFI, and to identify the potential bottlenecks for design improvement. Our proposed models, developed with queuing theory and simulation experiments, also offers the basis for engineers to design the bay length for various queue types in CFI, based on the target type of V/C ratio. Both simulation experiments and a real-world case study have indicated the effectiveness and applicability of our proposed models that is the first step forward the development of a complete set of guidelines for evaluation and design of CFI.

1. **Background**

Continuous flow intersection, named CFI, is an unconventional design that provides a larger capacity

over a conventional four-leg intersection. The main feature of CFI is that left-turn vehicles begin their

turn movement several hundred feet prior to the main intersection and then move to a separated lane

located at the right of the opposing through traffic. Figure 1 shows the geometric features of a full-CFI

design where all four legs contain a displaced left-turn crossover. Figure 2 compares CFI's the trajectory

and conflicting points with a conventional intersection, reflecting its strength on improving safety by

reducing the number of conflicting point.
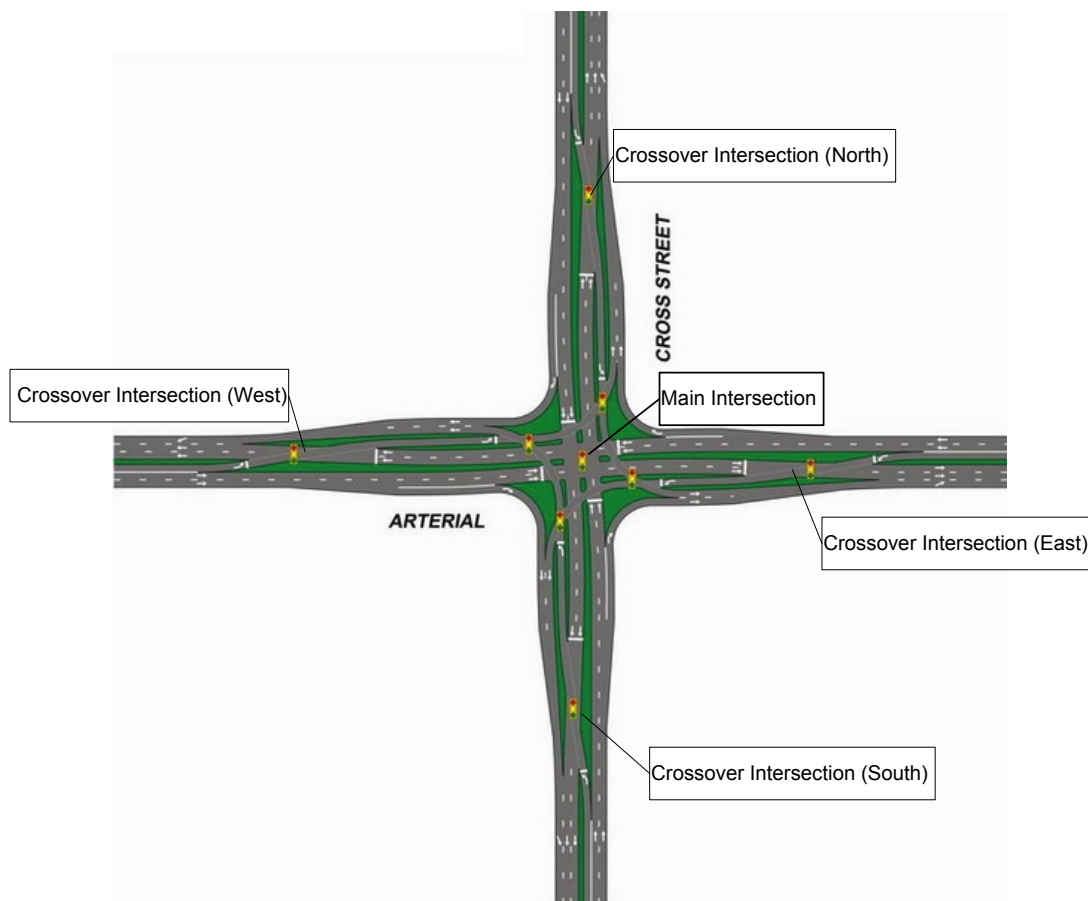
Figure 1 Full-CFI Geometry

Conventional Intersection
(16 conflicting points)

CFI Intersection
(12 conflicting points)
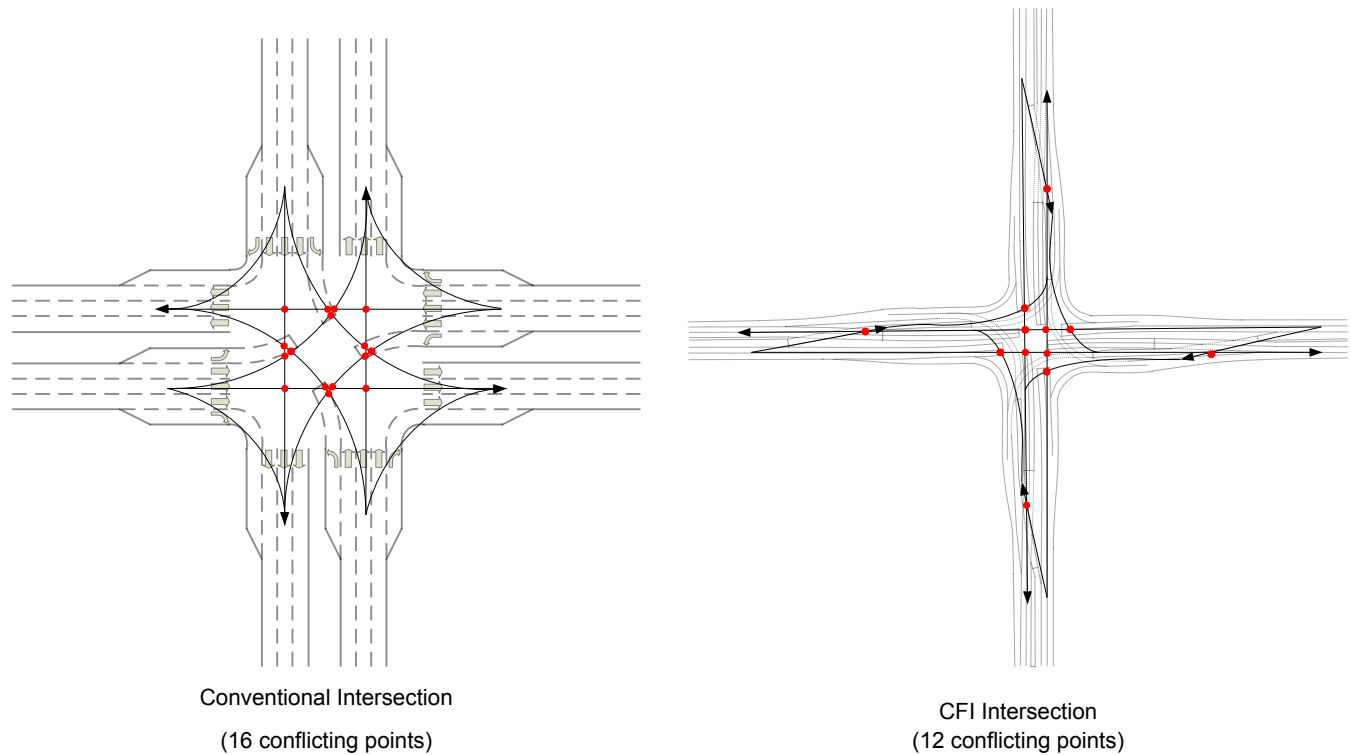
Figure 2 Movement Trajectory and Conflict Points of CFI and Conventional Intersection

Depending on the volume distribution, one can design CFI as a hybrid form between a full CFI and a conventional intersection. Figure 3 illustrates the three most commonly seen partial CFI designs:

- CFI-T intersection: A T intersection that contains one CFI leg.

- Two-Leg CFI (Type A): An intersection contains two displaced left-turn legs in two opposite directions. The other two legs have the same geometric feature as with a traditional intersection.

- Two-leg CFI (Type B): An intersection contains two displaced left-turn legs in two perpendicular directions. The other two legs have the same geometry as with a traditional intersection.

| T-CFI | T-CFI-A | T-CFI-B |
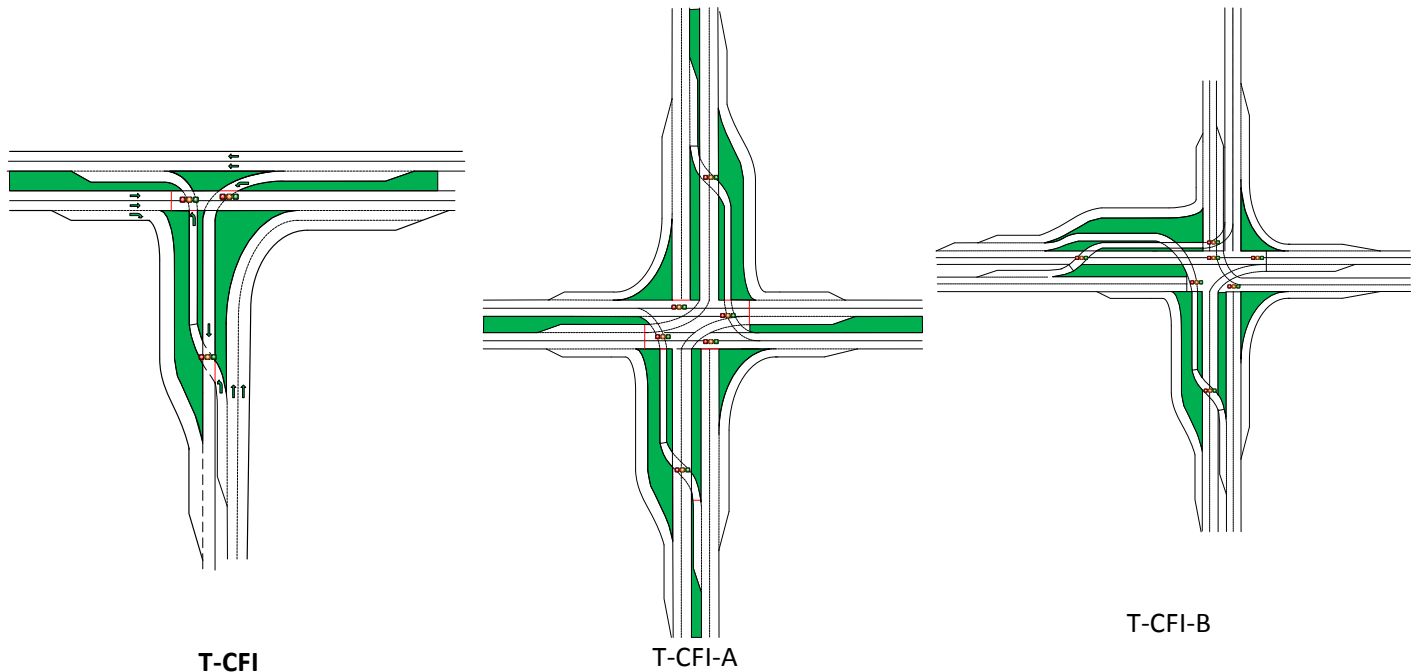
Figure 3 Geometry Layout of CFI Family

Comparing with a conventional intersection, the CFI design could offer the following potential benefits:

- Reducing the number of conflicting points;

- Reducing the number of signal phases to mostly two phases, thus increasing the intersection capacity;

- Incurring less overall traffic delay;

- Improving traffic safety by separating the left-turn movement.

CFI has becoming increasingly popular among traffic agencies if the right-of-way is insufficient for constructing a grade-separated interchange. Table 1 below summarizes these pioneering CFI designs constructed in different states.

Table 1 List of Examples of CFI Designs in US

Year	Location	Description
1995	Shirley, NY	A CFI-T intersection at the intersection of William Floyd Parkway and the entrance of Dowling College National Aviation
2001	Accokeek, MD	A CFI-T intersection at the intersection of Indian Head Highway (Route 210) and Berry Road (Route 228)
2006	Baton Rouge, LA	A partial CFI at the four legged intersection of US 61 at Seigen Lane
2007	Salt Lake City, UT	A partial CFI at the intersection of 3500 South and Bangerter Highway
2007	Fenton, MO	A partial CFI at US Route 30 and summit drive

1.1. Literature Review

In early 1990s, Goldblatt and Mier conducted a research on unconventional intersections and stated that the advantage of CFI is most significant when the demand approaches or exceeds the capacity of conventional intersection (1). In the same year, Hutchinson also concluded that the CFI design could be greatly superior over the conventional intersection when conflicting flows are very heavy. In 1996, FHWA published the Traffic Control Systems Handbook by FHWA indicated that the capacity of an intersection could be increased by nearly 60% if with a CFI design (4).

KLD Associates in their study reported that CFI could provide a comparable capacity at a fraction of the cost of a grade separation, and it can increases intersection capacity without compromising safety (5). Chick stated in his paper that CFI can improve the overall junction capacity through the removal of conflicts at the center of the intersection (7).

Based on a case study in Maryland, Pitaksringkarn suggested that CFI can significantly reduce the overall delay and queue lengths, respectively by 64% and 61%, during the PM peak hour (9). Using VISSIM simulations, Jagannathan and Bared developed some statistical models for the practitioner to assess the average delay and average queue length for three types of CFI intersections (10). Recently, FHWA

published a report (11), covering salient geometric design features, operational and safety issues, access management, costs and construction sequencing. Based on the simulation results of four CFI cases, the report indicates that a reduction in signal phases for CFI intersection significantly reduced vehicle delay.

Despite the increasing research efforts on CFI, many critical issues associated with CFI applications remain to be addressed. For example, how to evaluate a preliminary CFI design and identify potential service deficiency such as inadequate bay length? How to finalize a CFI design based on the projected traffic volume and the target level of service (or volume to capacity ratio)? How to design a series of coordinated signals that can take full advantage of the increased capacity of CFI intersection? This paper intends to address the first issue, which is to develop a set of models to quantitatively evaluate the service efficiency of a preliminary CFI design.

1.2. Research Scope and Objective

Figure 4 illustrates the typical four-stage process for CFI-design, including preliminary design phase, planning evaluation phase, geometry optimization phase, and signal design phase.

1.Preliminary Design
- Input
  - Demand
- Output
  - Type of unconventional treatment
  - Number of lanes
  - Distances between signals
  - Storage bay length

2.Planning Evaluation
- Input
  - Demand
  - Preliminary geometry design
- Output
  - Queue spillback locations
  - V/C ratio of each critical segment

3.Geometry Optimization
- Input
  - Demand
  - Queue spillback locations
  - V/C ratio of each critical segment
- Output
  - Optimum distance between signals
  - Optimum storage bay length

4.Signal Design
- Input
  - Demand
  - Detail geometry of intersection
- Output
  - Signal configuration
  - Estimated delay for operation
  - Estimated queue for operation
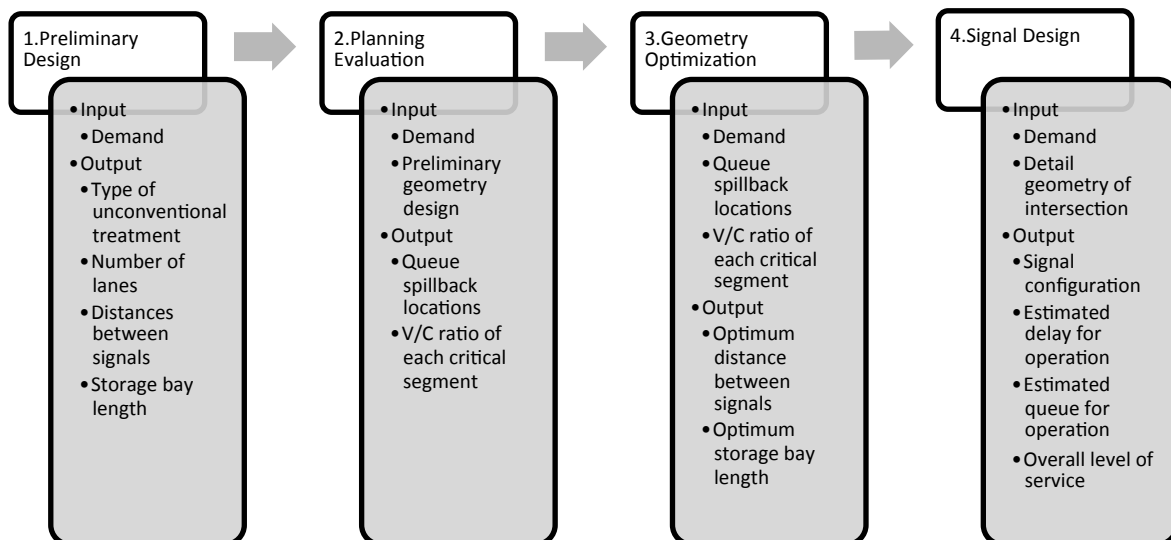 - Overall level of service

Figure 4 Design Process of CFI

At the preliminary design stage, it is essential for a responsible engineer to estimate the overall volume to the capacity ratio for the entire intersection and for each critical movement. The information will enable the engineer to assess the adequacy and efficiency of the designed geometric features. Note that given the complexity of unconventional design and the lack of effective guidelines, a carefully designed CFI may still fail to accommodate the distributed volumes due likely to insufficient bay length at some critical locations. Since CFI is composed of several sub-intersections, the capacity of an intersection will be affected by not only the distance between satellite signals but also the length of storage bay for turning movements. The primary objective of this study is to provide both procedures and necessary tools for engineers to perform the following tasks at the planning evaluation stage.

- Estimating the maximum queue length at each turning bay based on the given demand
- Identifying the volume/capacity ratio for each critical movement
- Computing the volume/capacity ratio for the overall intersection
- Redesign the bay length to meet the target volume/capacity ratio

Methodology for Developing the Evaluation Models

1.3. Basic Concepts

1.Identify the critical movement and potential queue spillback location

2.Categorize the critical movements according to its geometric character; For each category, define the type of the queue

3.By studying the mechanism of each queue, establishing evaluation formula and associated parameters	4.Build simulation model for each queue type; establish dataset for regression analysis

5.Perform regression analysis to obtain the maximum queue estimation formula. Test model accuracy and stability through validation process

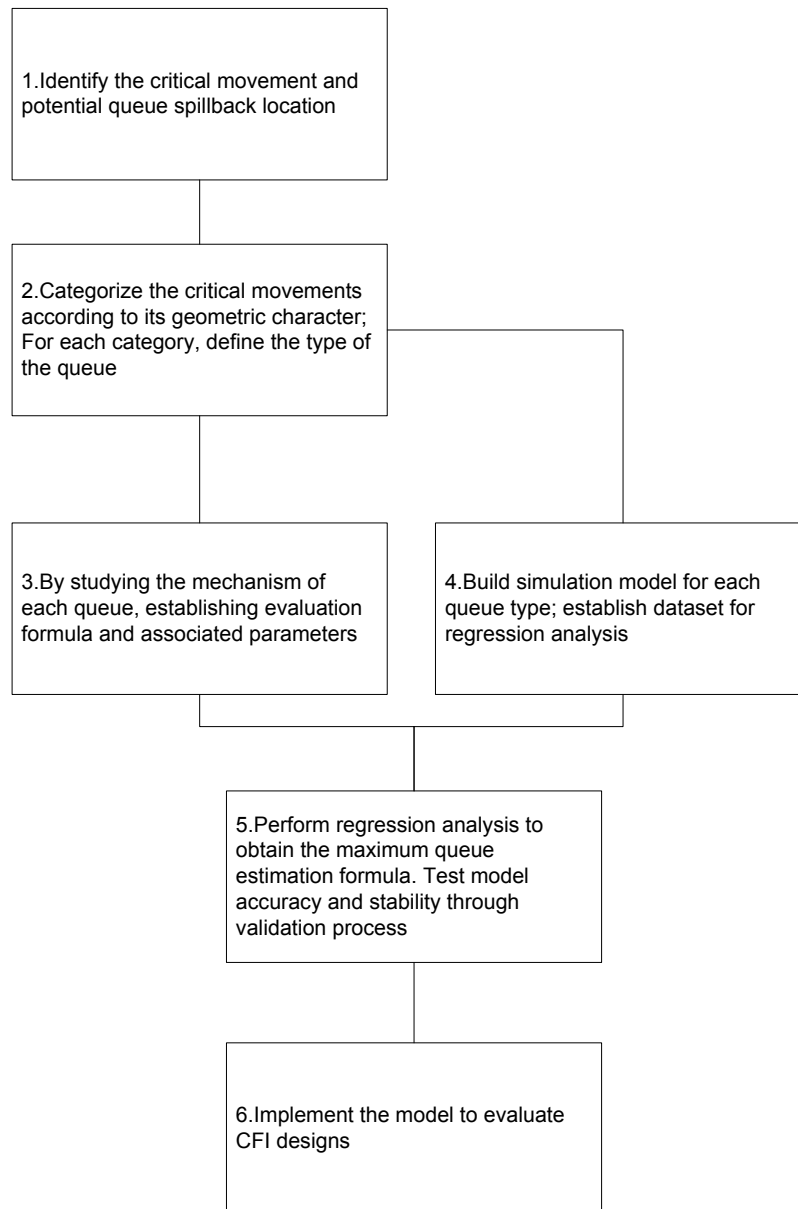| 6.Implement the model to evaluate CFI designs |
| --- |

Figure 5 Research Flow Chart

Figure 5 illustrates the key research steps for this study. We divided all the critical movements of CFI into

four categories, and identified four different queue models for their evaluation. Figure 6 shows the

categorization of traffic queue incurred at each segment of a CFI intersection, and Table 2 summarizes

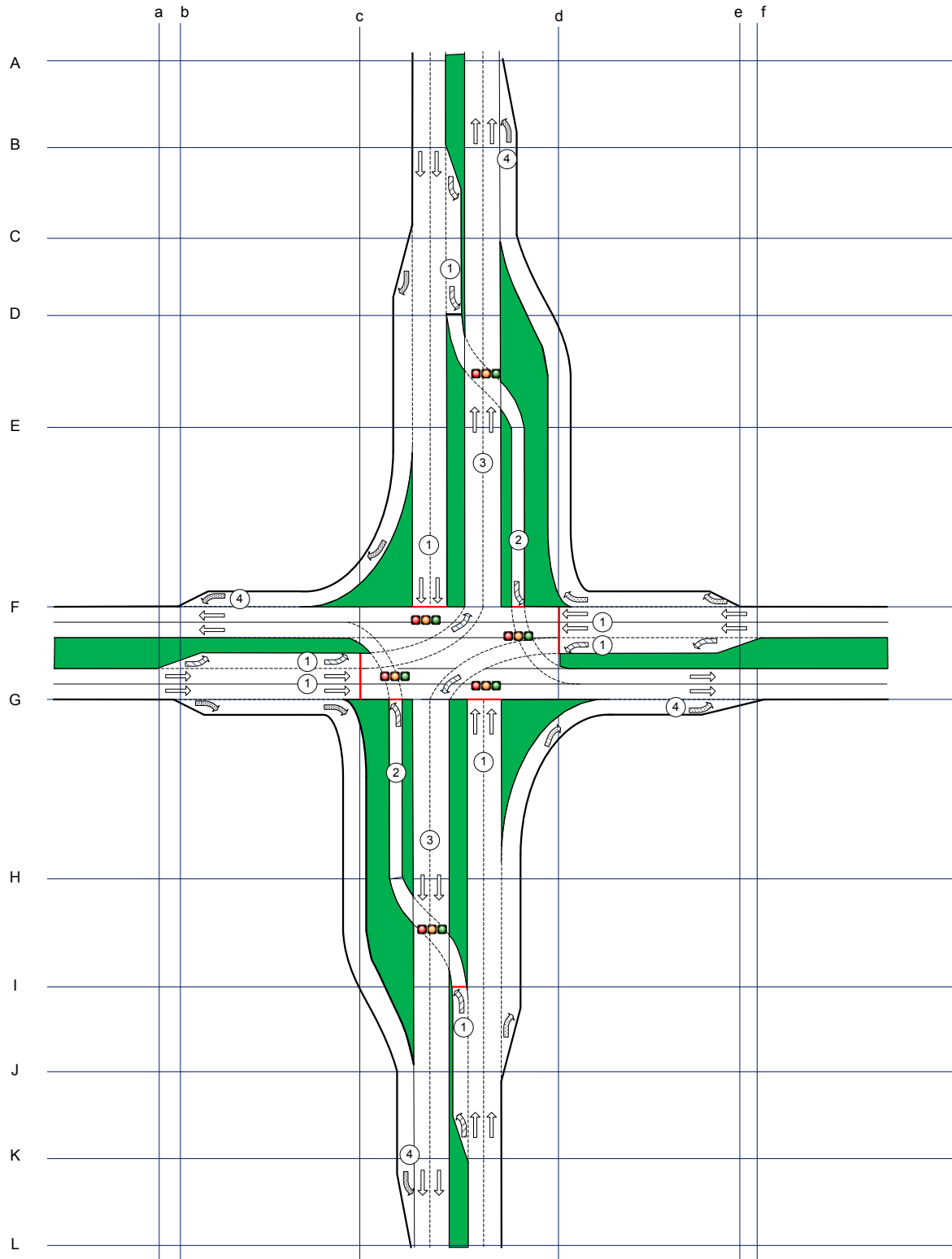associated information and justification of each queue type.

Figure 6 Categorization of different queue types incurred at a T-CFI-A intersection

<div align="center">**Table 2 Geometric Features of Different Queue Types**</div>

Type	Location	Description
1	Northbound through (G –K) Southbound through (B-F) Eastbound through (a-c) Westbound through (d-f) Northbound left (I-K) Southbound left (B-D) Eastbound left (a-c) Westbound left (d-f)	A signal is used to control the right of way of a single movement. The queue will form in front of the stop line.
2	Northbound left (G-H) Southbound left (E-F)	Two sequential signals are used to control the right of way of a single movement. The queue forms in front of the stop line of the downstream queue. The two intersections are coordinated and share the same cycle length.
3	Northbound through (E-F) Southbound through (G-H)	Three signals are used to control the right of way of two movements. Three signals are coordinated and share the same cycle length.
4	Northbound right (J-f) Southbound right (b-C) Eastbound right (b-L) Westbound right (A-e)	The merging flow attempts to merge into the main street while yielding to the traffic. The queue forms in front of the stop line of yield sign.

A typical CFI traffic queue could accumulate over time due to the capacity reduction at its downstream.

Assuming that the maximum queue length is a monotonic function of demand and other factors, one can represent this relationship as follows:

$$Q_{max} = f(D, x^*) \tag{1}$$

Where D is demand and x^* is a vector which contains all other factors that may affect Q_{max}

The inverse function of f can be used to compute the maximum demand level that does not cause a spillback, and we define the maximum demand as the capacity of the target segment.

$$C = f^{-1}(\text{B}, \text{x}^*) \tag{2}$$

Where B is bay length. Then one can express the volume to capacity ratio of the segment as follows:

$$V/_C = D/C \tag{3}$$

Where D is the projected demand and C is the capacity obtained from Equation 2.

In the remaining analysis, we have applied Equation 1 to 3 to each movement segment of CFI intersection to give an overall evaluation of the intersection.

1.4. Development of Queue Models

The procedure for developing those four queue models for a CFI intersection contains the following three steps:

1. Identifying all geometric and demand factors and their interactions that may contribute to the formation of traffic queue

2. Applying traffic engineering knowledge and queuing theory to capture the systematic relationship between the given type of queue length and all contributing factors

3. Applying simulation experiments and regression analysis to capture some random impacts on the resulting traffic queue

Note that at the simulation experiment study, we employ VISSIM to generate the maximum length for each type of queue. To explore the interrelationship between the max queue and the demand, we have generated a total of 350 experimental scenarios for each queue type, where 300 samples are used for fitting the regression and 50 samples were used to perform the evaluation of each of these four queue models. MSE (Mean square error) and MAPE (Mean absolute percentage error) were computed for each queue model. Each simulation run covers two hours and ten replications with different random seeds.
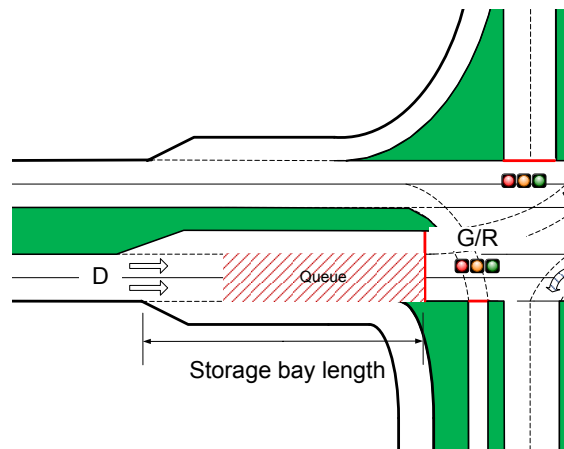
- **Type-1 Queue**

Figure 7 Geometric Feature of Queue Model 1

Figure 7 illustrates the formation pattern which often incurs Type-1 queue when a stream of traffic flow encounters a signal. Table 2 summarized 8 movement segments in a CFI design that could experience Type-1 queue scenario. Here the arrival of the demand is assumed to be random. The green (red) time can be estimated based on the ratio between demand flow and the total conflicting flow of the intersection. To simplify the potential application, we propose a concise form for Type-1 queue estimation and the result is:

$$Q_{max} = 32.78 + 0.01312D * R + 0.000394(D * X)^2 \tag{4}$$

$$t - value:\quad (6.380) \qquad (14.220) \qquad (9.071)$$

$$R^2 = 0.908 \qquad Root\ MSE = 116.1 \qquad MAPE = 11.8\%$$

Where

$Q_{max} - The\ maximum\ length\ of\ the\ queue$

$D - Demand\ flow\ in\ vehicle\ per\ hour\ per\ lane$

$R - Estimated\ red\ phase\ duration$

$X - Degree\ of\ saturation, X$

$$= \frac{D}{s * g}\ in\ which\ s\ is\ saturation\ flow\ rate, g\ is\ estimated\ green\ time\ ratio$$

As commonly used in traffic signal literature, the first term of Equation 4 represents the uniform queue term which captures the queue under uniform arrival, and the second term, called random queue term, is to catch the additional part of the queue caused by the randomness of the arriving and discharging flows. Uniform queue term is linearly correlated to the number of vehicles arriving during the red phase.
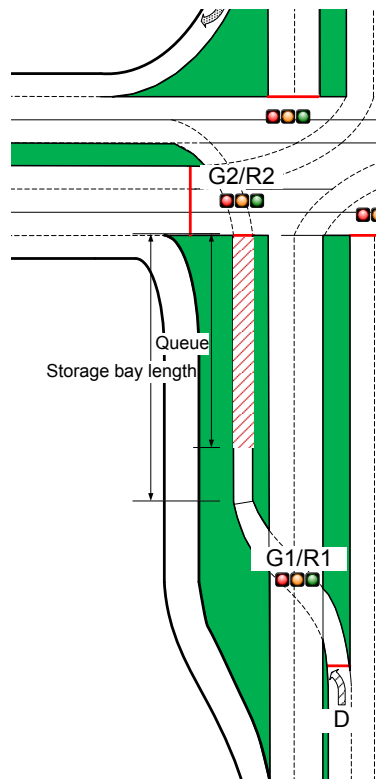
- **Type-2 Queue**

The second queue model is used to predict the queue length between two consecutive signals (see

Figure 8). For Type-2 queue, the target traffic flow is constrained between the upstream and

downstream signal, which are assumed to be well coordinated. Hence, the offset between the two

signals is assumed to be optimized to maximize the bandwidth of the incoming traffic flow. The short

distance between the two signals (usually less than 600 feet) makes the downstream signal often

encounters heavy platoon arrivals. Therefore, the random arrival assumption is not justifiable under this

condition.

Notably, Type-2 queue is mainly the residual queue which exists when the green time of the upstream

signal is longer than that of the downstream signal (G1>G2). Otherwise, the arrival and departure curves

at downstream will merge together and incur no residual queue. Based on such relations, we define the following linear queue predictor Z for Type-2 queue:

$$Z = \begin{cases} \dfrac{(G_1 - G_2)D}{3600} & if \; G_2 < G_1 \\ 0 \; otherwise \end{cases}$$

We propose the following simple form for Type-2 queue:

$$Q_{max} = 0.856 + 45.71Z \tag{5}$$
$$t-value: \quad (3.350) \quad (46.399)$$
$$R^2 = 0.888 \qquad Root\; MSE = 104.2 \qquad MAPE = 12.1\%$$

Where

$Z -$ *The linear predictor of queue under the optimal coordination state*

$D -$ *Demand per lane in vehicle per hour*

$G_1 -$ *The estimated green time at the upper stream signal*

$G_2 -$ *The estimated green time at the down stream signal*
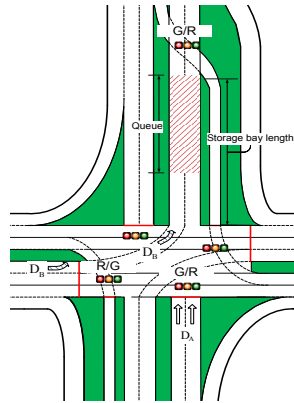
- **Type-3 Queue**

Figure 9 Geometric Feature of Queue Model 3

The Type-3 queue model is used to estimate the queue length between two signals under two incoming traffic flows, where two movements (Demand A and B in Figure 9) share one storage bay when they wait for the red light at downstream signal. The right of way of the two movements is controlled by the

upstream signal controller. We assume that two signals have the same cycle length and the offset is optimized to maximize the bandwidth of mainline demand (Demand A).

Considering all vehicle flows contributing to the queue and the potential non-linear impact, we proposed the following form for Type-3 queue:

$$Q_{max} = 6.208 + 0.01005D_B * R + 0.000103D_B^2 + 0.004997D_A * G \qquad (6)$$

$$t - value: \quad (5.760) \qquad (27.308) \qquad (5.661) \qquad (21.616)$$

$$R^2 = 0.897 \qquad Root\ MSE = 114.0 \qquad MAPE = 20.0\%$$

Where

Q_{max} − The maximum back of the queue

D_A − Demand A volume in vehicle per lane per hour

D_B − Demand B volume in vehicle per lane per hour

R − Estimated red time of movement A at upstream signal

G − Estimated green time of movement A at upstream signal

Equation 6 contains three terms: the first term captures the uniform queue incurred by demand B; the second term is random queue term; and the last term accounts for the additional queue caused by demand A during the queue clearance time during the green phase.
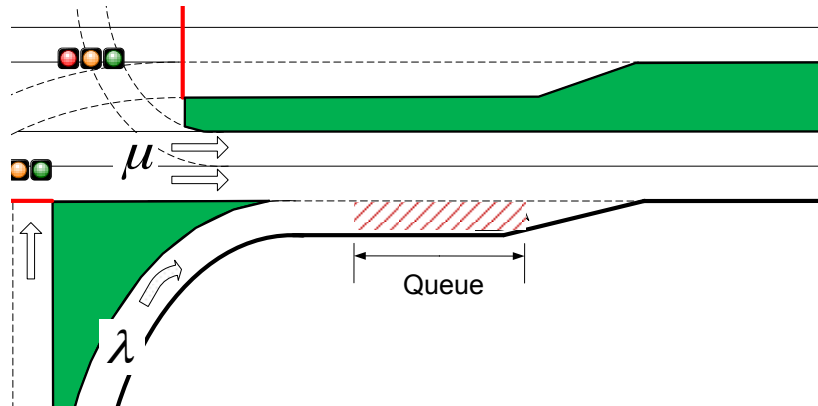
- **Type-4 Queue**

Figure 10 Geometric Feature of Queue Model 4

Type-4 queue model is used to predict the queue length in a merging area (see Figure 10). The merging

flow intends to merge into the mainline during acceptable gaps. Such a relationship can be described

with a classical M/G/1 model which stands for random arrival/single server/single queue. The service

time, which is equivalent to the merge time, is assumed to be a general distribution (i.e, not the typical

exponential distribution). Thus, the expected waiting time is:

$$E(S) = \frac{\lambda}{\mu}[e^{t_{nr}\mu} - (1 + t_{nr}\mu)] \tag{7}$$

Where

$E(S) -$ *The expected waitingtime for a successful merging*

$t_{nr} -$ *The gap time required for a safe merge*

$\lambda -$ *The arrival rate of merging flow, in vehicle per hour*

$\mu -$ *The arrival rate of the mainline flow, in vehicle per hour*

A detail description of such a M/G/1 model is available from the literature (12). According to Little's law,

the average number of waiting vehicles is the product of arrival rate and the expected waiting time.

$$\rho = \lambda E(S)$$

A strong correlation between the maximum queue and the average number of waiting vehicles is

confirmed by our simulation result. Based on queuing theory and simulation data, we proposed the

following formulation to for Type-4 queue:

$$Q_{max} = 5.23 + 66.22\rho^2 + 1007.83\lambda^2 + 328.66\mu^2 \tag{8}$$

$$t - value: \quad (16.79) \quad (40.71) \quad (11.37) \quad (11.98)$$

$$R^2 = 0.962 \quad Root\ MSE = 11.8 \quad MAPE = 9.2\%$$

To capture the randomness of maximum queue at the high demand range, the second and third term is

added to account for the impact of demand from both directions.

2. Model Application

This section illustrates the model application with the intersection at MD 228 and MD 210 in Maryland, a CFI-T intersection. Figure 11 illustrates the geometric features of the example CFI-T intersection and its peak hour demand volumes.

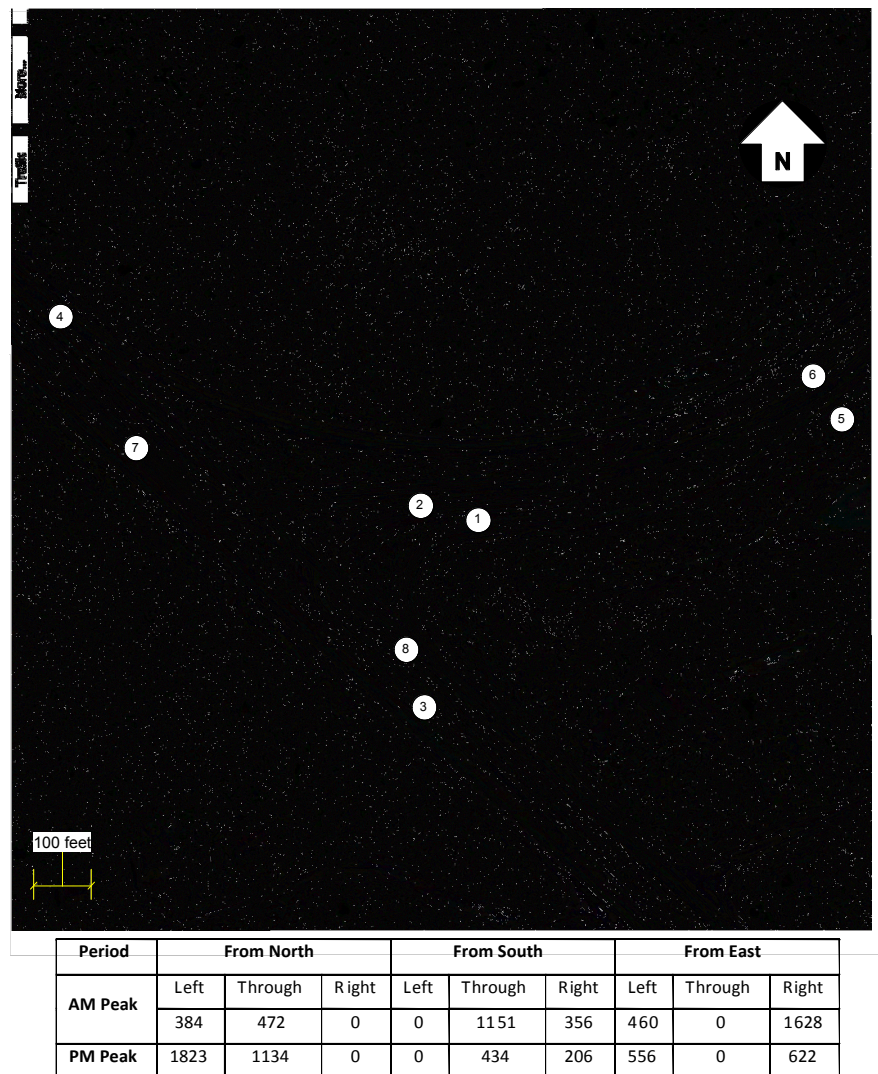| Period	From North			From South			From East		
	Left	Through	Right	Left	Through	Right	Left	Through	Right
AM Peak	384	472	0	0	1151	356	460	0	1628
PM Peak	1823	1134	0	0	434	206	556	0	622

Figure 11 Picture of MD 210 & MD 228 and Peak Hour Demand

Note that this CFI-T design contains two sub intersections and six movement flows. The east leg is a CFI design. All those storage bays are marked with a number. Using the proposed evaluation model, we summarized the maximum queue and V/C ratio in Table 3.

Table 3 Planning Evaluation Result of Original Design

Segment	Bay Length (feet)	Queue Type	AM				PM			
			V/C Ratio	Estimated Maximum Queue	Simulated maximum Queue (feet)	Queue Spillback	V/C Ratio	Estimated Maximum Queue	Simulated maximum Queue (feet)	Queue Spillback
1	400	model 2	0.4	159	163	No	0.46	183	170	No
2	430	model 2	0	0	7	No	0	0	8	No
3	500	model 4	0.27	23	35	No	0.33	53	59	No
4	1500	model 4	0.34	50	38	No	0.13	12	28	No
5	300	model 4	0.21	17	32	No	0.12	17	26	No
6	500	model 1	0.51	203	219	No	*1.09*	*553*	*567*	*Yes*
7	1200	model 1	0.18	173	159	No	0.3	197	203	No
8	400	model 1	*1.18*	*490*	*478*	*Yes*	0.72	292	281	No

The result indicates that queue spillback occurs at segment 6 during PM peak hours and at segment 8 during AM peak hours. The actual maximum queue obtained from simulation also confirmed such results. In order to eliminate the queue spillback scenario, one can either extend the storage bay length or increase the number of lanes to reduce the demand per lane. Based on the estimated maximum queue length from Table 3, we proposed two possible revisions:

- Increase the bay length of segment 8 from 400 feet to 500 feet and segment 6 from 500 feet to 600 feet, respectively
- Increase one additional lane for left-turn movement from east bound

The performance of the intersection before and after the improvement is compared in Table 4.

Table 4 Planning Evaluation of Modified Design

Solution	Segment	AM				PM			
		V/C Before	V/C After	Max Queue Before	Max Queue After	V/C Before	V/C After	Max Queue Before	Max Queue After
1 (Increasing Bay Length)	1	0.4	0.4	159	159	0.46	0.46	183	183
	2	0	0	0	0	0	0	0	0
	3	0.27	0.27	23	23	0.33	0.33	53	53
	4	0.34	0.34	50	50	0.13	0.13	12	12
	5	0.21	0.21	17	17	0.12	0.12	17	17
	6	*0.51*	*0.42*	*203*	*203*	*1.09*	*0.91*	*553*	*553*
	7	0.18	0.18	173	173	0.3	0.3	197	197
	8	*1.18*	*0.98*	*490*	*490*	*0.72*	*0.58*	*292*	*292*
2 (Add Additional Lane)	1	0.4	0.19	159	0	0.46	0.16	183	22
	2	0	0.28	0	18	0	0	0	0
	3	0.27	0.14	23	11	0.33	0.16	53	33
	4	0.34	0.34	50	50	0.13	0.13	12	12
	5	0.21	0.21	17	17	0.12	0.12	17	17
	6	*0.51*	0.33	203	165	*1.09*	*0.7*	*553*	*345*
	7	0.18	0.23	173	213	0.33	0.33	197	197
	8	*1.18*	*0.85*	*490*	*324*	0.72	0.72	292	292

As shown in the highlighted numbers, both alternatives can bring down the V/C ratio below 1 for those two congested segments. However, the second alternative also concurrently decreases the V/C ratio of several other segments. This is due to the strong correlations between different movements and segments in CFI intersection. Therefore, improvements on one location or movement can sometimes increase the overall performance of the intersection. There are two plausible reasons for having such improvements. First, the change in geometric feature can affect the ratio of lane volume at each intersection which will in turn change the green time ratio and queue length of different movements. Secondly, a queue spillback at downstream segment can significantly decrease the capacity of its upstream segment. Hence, any improvement in the downstream intersection could also alleviate the side effect at its upstream intersection caused by the queue spillback. The first type of interaction has been considered in our evaluation model. Nevertheless, the second factor has yet to be developed in the future study.

3. Conclusion

This paper proposed an evaluation method for CFI intersections, based on in-depth analysis of the interrelationships between demand distribution and its geometric features, and simulation experiments. The primary task of planning model is to identify the insufficient storage bays which will limit the capacity of a CFI intersection. Based on the output from our model, we can estimate the volume to capacity ratio for all critical movements, and such information can be used for indicate the overall efficiency of the CFI design. This study has also used a real world example to demonstrate the application of our proposed models.

As we have mentioned previously, the complete process for design of unconventional intersections includes four stages. This paper has addressed the modeling work for use at the planning evaluation stage. This is the first step towards establishing a complete set of procedures and tools for analysis and design of unconventional intersections. Along this line, we fully recognize that many issues remain to be

addressed in the future study. For example, how to design an optimal set of signal configuration for CFI, given the geometry parameters? How to evaluate the overall level of service of an intersection? How to perform a comprehensive benefit/cost analysis for CFI, compared with conventional solutions?

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