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DEVELOPMENT OF DESIGN AND TRAFFIC CONTROL FOR TIGHT URBAN DIAMOND INTERCHANGE

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Abstract: A new geometric design and two types of signal optimization models were proposed to cope with the problem for the left turn traffic from minor road to ramp in TUDI. One model is BMILP that optimizes all signal timings for the proposed design, the other is a combined model that optimizes green intervals by TRANSYT-7F subject to phase sequence and offset resulted from the BMILP. The proposed design and traffic control allows the left-turn traffic to make left-turn and enter the ramp at the upstream intersection. As the result, the left-turn traffic doesn't have to go to the downstream intersection and there are less traffic volumes than those of TUDI at the two intersections. The simulation results showed that the proposed design using the proposed models certainly are superior to TUDI using TRANSYT-7F in reducing delay, where the more traffic (especially the left turn traffic) the greater the effect. Combined model was superior to BMILP for low left turn volume, vice versa for high left turn volume.

Key Words: TUDI, internal link, external link, upstream intersection, downstream intersection

1. INTRODUCTION

Conventional diamond interchange (CDI) has the ability to accommodate higher volumes of traffic safely and efficiently through bottleneck intersections along the urban artery. Therefore CDI is the most widely used type of interchange at intersections of major roads where right-of-way is limited, turning volumes are low to moderate, and continuous frontage roads exist. There are two adjacent at-grade intersections on minor road in the tight urban diamond interchange (TUDI) as shown in Figure 1 and Figure 2. We denote the link between an upstream intersection and a downstream intersection as an internal link, and upstream feeding links to the internal link as external links as shown in Figure 2. The left-turn traffic flows (two bold lines in Figure 1) from the minor road to the ramp connecting to the major road have both to go through an upstream intersection and to make left-turn at a downstream intersection in each direction.

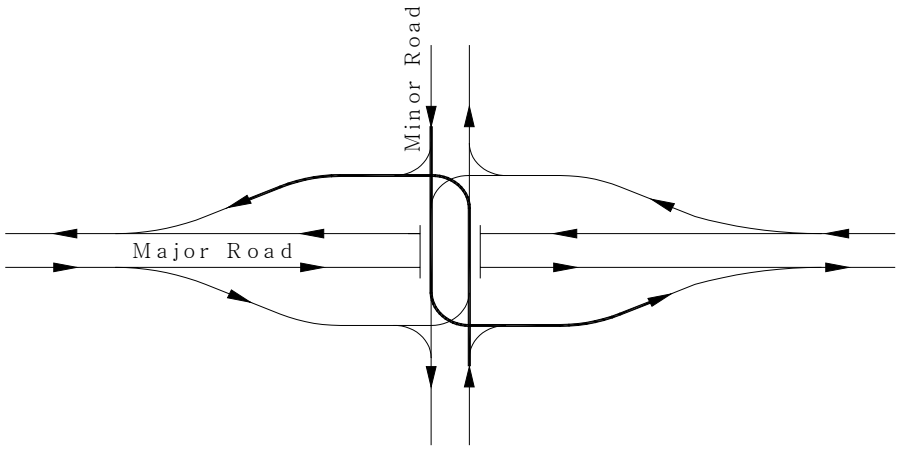

Figure 1. Traffic Flow Configuration in TUDI

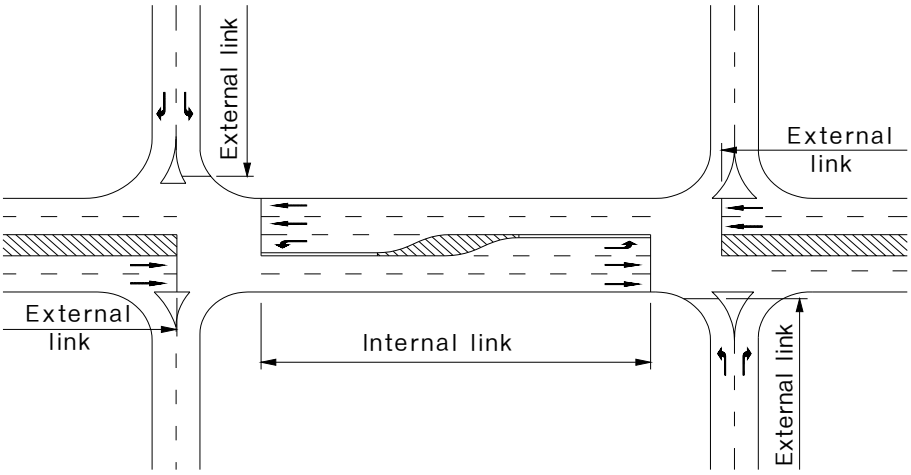

Figure 2. Geometry Layout of Minor Road in TUDI

Therefore it may be difficult that TUDI adequately handle heavy volume of the left-turn traffic from the minor road to the ramp at the downstream intersection, which is quite near to the upstream intersection. This deficiency depends mainly upon spillback resulting from the lack of queuing storage of the left-turn traffic and through traffic within the short internal link. When traffic operation problems are caused by the heavy left-turn traffic in TUDI, traditional remediation tactics include the use of special signal phasing and lane canalization. Other tactics include the use of traffic agents, widening the right-of-way and improving alternative routes.

Single point urban interchange (SPUI) appears as a viable alternative to TUDI control in certain situations is adaptable to locations where right-of-way is limited and can be operated efficiently under traditional NEMA 8-phase control. On the other hand, it does not appear to be well-suited to continuous frontage road situations because of capacity restrictions that resulted from an additional signal phase. When these or other tactics prove inadequate or infeasible in TUDI, a grade separation may be considered. Grade separation as a solution to traffic operation problems at TUDI has major drawbacks. The time and cost associated with construction of grade-separated structures such as three level diamond interchange make this solution impractical in many applications.

Therefore the purpose of this research is to develop a new geometric design and traffic control to cope with the aforementioned deficiency of TUDI.

2. PROPOSED DESIGN CONFIGURATION

The proposed design and traffic control allows the left-turn traffic, which moves from the minor road to the ramp at the downstream intersection in TUDI, to make left-turn and enter a divided exclusive lane connecting to the ramp at the upstream intersection as shown in Figure 3.

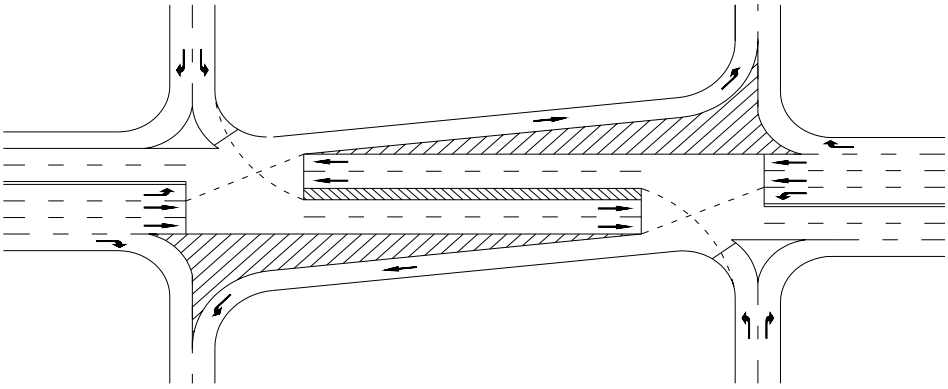

Figure 3. Geometry Layout of Minor Road for Proposed Design

Since the left-turn traffic does not have to go to the downstream intersection, therefore, there are less traffic volumes of the proposed design than those of TUDI at the two intersections. Since the left-turn traffic queues up for its right of phase at the upstream intersection with long external link of instead the downstream intersection with the short internal link, the proposed technique has no problem such as spillback related with the left-turn traffic in TUDI.

Compared to the TUDI, it is noticed that the technique has three advantages;
First, spillback seldom happen since the left-turn traffic from the minor road to the ramp queues up for their phase on left-turn auxiliary lane in long external links at the upstream intersections instead of the short internal links at the downstream intersections in TUDI. Second, since the left-turn traffic is eliminated from the internal link, the technique handles less traffic than that of TUDI at the intersections. Third, since there is straight-ahead traffic only in the internal links, it is easy to control traffic flow in the internal links.

As the results, levels of service of the link as well as the two intersections would be improved. The proposed technique can be also efficiently adaptable to the interchange of major roads where continuous frontage roads exist because it does not need additional phase for the frontage roads. The technique may be similar to the concept of continuous flow intersection (CFI). CFI requires an additional intersection on each upstream link for the left turn, but the proposed design with existing two intersections doesn't need any other intersection. Compared to the CFI, therefore, the restrictions for CFI are removed, and facility only is requested to separate the left-turning traffic and oncoming traffic.

3. TRAFFIC SIGNAL OPTIMIZATION MODEL FOR PROPOSED TUDI

3.1 Constraints

Under the geometric scheme for the proposed design, we developed two models to optimize an integrated signal timing plans (cycle length, offset, phase sequences and green intervals) between the two intersections. One is a combined model with TRANSYT-7F, the other is Binary-Mixed-Integer-Linear-Programming (BMILP) which are solvable by any standard branch and bound routine. BMILP optimizes all aforementioned signal timings in a model. Using the phase sequence and offset of BMILP, however, the combined model optimizes green intervals by TRANSYT-7F. The offset of each phase is prescribed as 0 for a signal operation in BMILP. In other words, as shown in figure 5, the starting time and the duration of each phase of the two intersections should be sequentially same each other. The phase sequence enables all traffic movements, which are given green interval per phase, of the two intersections to be comparable in the integrated signal control. The traffic movements are the finest elements for phase sequence, and are defined as shown in figure 4.

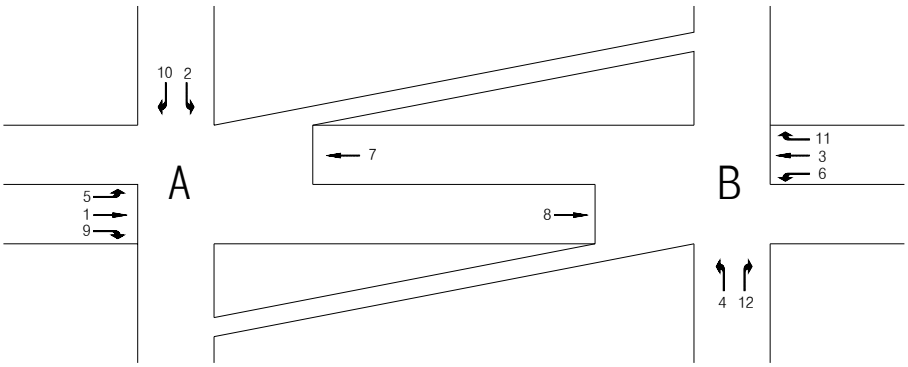

Figure 4. Numbering Convention for Traffic Movements

Since signal control for the two intersections should be operated as that for an intersection, the length of the internal link may be an important factor for the proposed design. For the effective operation of the short internal link in the proposed TUDI, the phase sequence guarantees that the internal link can be fully utilized as queues storage for maximization of out put. In addition the green intervals are allocated for the queues never to overpass the length of the internal link. Two types of phase sequences are set up to ensure the requirements mentioned above as shown in Figure 5. If there is frontage road, the phase for the road is common with phase 1 in Type 1 and phase 2 in Type 2, respectively. Under the phase sequences, both incoming vehicles from the ramps to the internal links and straight-ahead vehicles that can't pass the downstream intersections during their right of phase are included into the queues in the internal link.

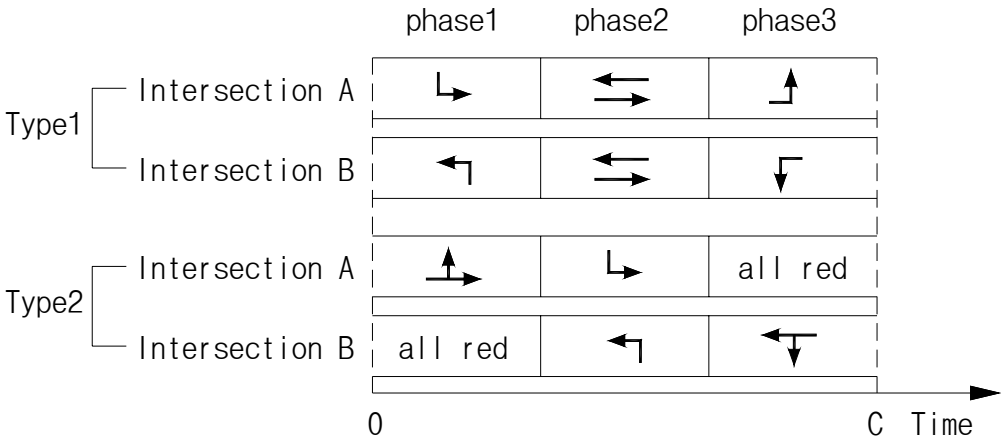

Figure 5. Phase Sequence Types for Traffic Control in Proposed TUDI

Here the right-turn traffic movement i (i = 9, 10, 11, 12) with exclusive lane is supposed not to need its right of phases, green intervals for traffic movement 7 and 8 are equal to those for movement 1 and 3, respectively. Under the conditions above-mentioned, we denote g_i as a green interval for traffic movement i (i =1,2,…,6 in Figure 3). Next satisfying the conditional selection of one (Type 1 or Type 2) from the two types of phase sequences, each green interval should satisfy following constraints:

$$g_5 - g_6 \le \alpha \tag{1}$$

$$g_5 - g_6 \ge \alpha \tag{2}$$

$$g_1 - g_3 \le \alpha \tag{3}$$

$$g_1 - g_3 \ge \alpha \tag{4}$$

$$g_2 - g_4 \le \alpha \tag{5}$$

$$g_2 - g_4 \ge \alpha \tag{6}$$

$$g_1 - g_5 \le 1 - \alpha \tag{7}$$

$$g_1 - g_5 \ge \alpha - 1 \tag{8}$$

$$g_3 - g_6 \le 1 - \alpha \tag{9}$$

$$g_3 - g_6 \ge \alpha - 1 \tag{10}$$

$$g_2 - g_4 \le 1 - \alpha \tag{11}$$

$$g_2 - g_4 \geq \alpha - 1 \tag{12}$$

where α is a binary integer, $\alpha = 1$ for Type 1, $\alpha = 0$ for Type 2.

The cycle lengths of the two intersections must be same for an signal operation. For satisfying the condition, a constraint for the green intervals needs as follows.

$$g_1 + g_2 + g_3 + g_4 + g_5 + g_6 + 6Y = 2C \tag{13}$$

where Y is intergreen interval, and C is cycle length.

Let the minimum cycle length and maximum one be $C_{\min}$ and $C_{\max}$, respectively. The constraints on the cycle length can now be specified as

$$C_{\min} \leq C \leq C_{\max} \tag{14}$$

In order to guarentee reseonable range of the cycle length, we let C be greater than minimum cycle length (C_0) for undersaturated traffic flow and the range of the cycle length include the cycle length minimizing delay (C_{web}) at the intersection developed by WEBSTER (1966) as

$$C_0 = L/(1-Y) \tag{15}$$

$$C_{web} = (5+1.5L)/(1-Y) \tag{16}$$

Mathematical expression for the vehicles incoming to the internal links from the external links during their right of phase depends on whether the corresponding phase is oversaturated or not. Now, let q_i be arrival rate (vps), and s_i be saturation flow rate (vps) for movement i in the exeternal links, where i = 1,2,3,4.

If the phase for movement i is oversaturated, then incoming volume during its phase is equal to the capacity flow for green time, $s_i g_i$; otherwise, vehicles arriving during the cycle time, $q_i C$.

Let incoming volume of movement i to internal link from the external link be $V_{in,i}$, then the volume can be expressed as

$$V_{in,i} = Min\{q_i C, s_i g_i\} \tag{17}$$

The right hand side of above equation cannot be solved directly using linear programming. The equation must be transformed into the following 4 equivalent linear forms:

$$V_{in,i} \le q_i C \tag{18}$$

$$V_{in,i} \le s_i g_i \tag{19}$$

$$q_i C - V_{in,i} \le M\delta_i \tag{20}$$

$$s_i g_i - V_{in,i} \le M(1-\delta_i) \tag{21}$$

where M is sufficiently large such that equation (17) is redundant with respect to any active constraint, and δ_i is a binary integer variable. For $\delta_i = 0$ (undersaturated condition), $V_{in,i}$ is equal to $q_i C$; otherwise, $V_{in,i}$ becomes $s_i g_i$.

We let the queues of movement i be $Q_{in,i}$. The queues in the internal link are incoming vehicles of movement 2 and movement 4 from the ramps, and the straight-ahead vehicles that cannot pass the downstream intersection among incoming vehicles of movement 1 and movement 3 in each direction. Since the offset = 0 and upstream and downstream intersection are operated in a signal, the queues of straight-ahead traffic movement i (i = 1,3) could be assumed as the volumes incoming during the time traveling the internal link. Therefore $Q_{in,i}$ is calculated as follows.

$$Q_{in,i} = V_{in,i} \text{ for i = 2, 4} \tag{22}$$

$Q_{in,i}$ for i =1, 3 depends upon whether the phase is oversaturated or not. If the phase is oversaturated, then the queues are equal to $s_i T_{in}$; otherwise $q_i T_{in}$, where T_{in} is the time traveling the internal link. Therefore $Q_{in,i}$ for movement 1 and movement 3 must be set as

$$s_i g_i - q_i C \ge -M\delta_i \tag{23}$$

$$s_i g_i - q_i C \le M - M\delta_i \tag{24}$$

$$Q_{in,i} - q_i T_{in} \ge -M\delta_i \tag{25}$$

$$Q_{in,i} - q_i T_{in} \le M\delta_i \tag{26}$$

$$Q_{in,i} - s_i T_{in} \ge M\delta_i - M \tag{27}$$

$$Q_{in,i} - s_i T_{in} \le M - M\delta_i \tag{28}$$

where for $\delta_i = 0$ (undersaturated condition), $Q_{in,i}$ is equal to $q_i T_{in}$; otherwise, $Q_{in,i}$ becomes $s_i T_{in}$.

The total queues (Q_{in}) are the sum of the queues for the movements incoming to the internal link in each direction as follows.

$$Q_{in} = \sum Q_{in,i} \text{ , where i = 1, 2; i = 3, 4} \tag{29}$$

In order not for the total queues to overflow the internal link, the following constraint should be satisfied in each direction:

$$Q_{in} \le \frac{n_{in} L_{in}}{S} \tag{30}$$

where n_{in} is the number of the internal link lanes, L_{in} is the internal link length (m), S is saturation headway (m).

For the case of high traffic demand, requirement above-mentioned can be requested to external links, especially ramps, as follows.

$$q_i C \le \frac{n_{ex} L_{ex}}{S} \tag{31}$$

where n_{ex} is the number of the external link lanes, L_{ex} is the external link length.

3.2 Objective Function

Since the internal links are used as the temporal queue storages for an signal operation as described in section 2.1, the delay of the proposed design mainly depends upon the external links. Here, since the delay is not linear form but strongly interrelated with queues, we introduced the minimization of the queues for 4 external links as the objective function. Under the under-saturated traffic flow, the queues with respect to arrival traffic volume, q_i, can be depicted as shown in Figure 6. Introducing the queues as the decision variables in the objective function, we classified the queues into primary queues and secondary queues as shown in Figure 6, where the primary queues are the queues occurring during red interval, $Q_{ex1,i}$, and the secondary queues during the queue clearance time among the green interval, $Q_{ex2,i}$, which can be set as

$$Q_{ex1,i} = q_i r_i \tag{32}$$

$$Q_{ex2,i} = \frac{q_i^2 r_i}{s_i - q_i} \tag{33}$$

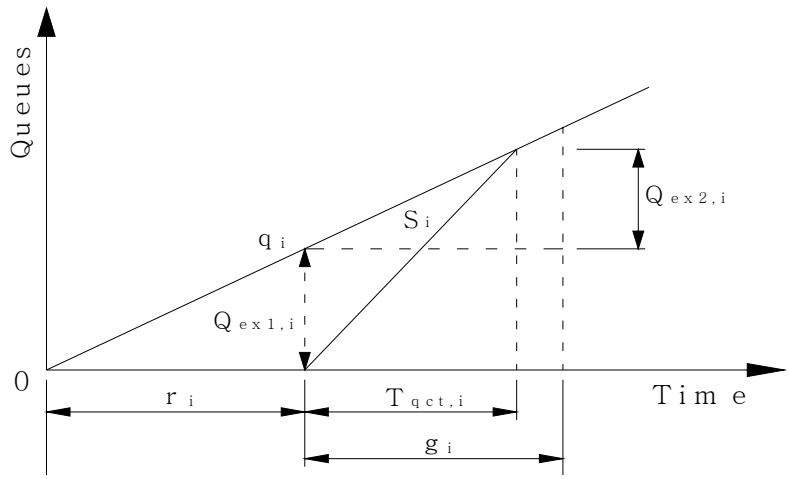

Figure 6. Queues for Under-saturated Traffic Flow at An Signalized Intersection

Since the delay time for $Q_{ex1,i}$ is generally greater than that for $Q_{ex2,i}$, we introduced adjustment multiplier (K_i , $K_i \geq 1$) for favor of $Q_{ex1,i}$ to $Q_{ex2,i}$ in objective function as follows:

$$Minimize \quad \{K_i q_i + \frac{q_i^2 r_i}{(s_i - q_i)}\} r_i \tag{34}$$

The signal timings are optimized to minimize the sum of the weighted two types of queues in external links because the internal links are used temporal queue storage in the BMILP. The binary integer variables are introduced to select the phase sequence, whereas the continuous variables are to calculate queues and signal timings (i.e., cycle length, and green intervals). Therefore the proposed model can be formulated as the following BMILP:

Objective function: (34)

subject to constraints in (1) - (14), (8) – (30), and optional constraint, (31).

Subjected to phase sequence and offset resulted from BMILP above; the green intervals are optimized to minimize delay and stops by TRANSYT-7F in the combined model.

4. APPLICATIONS AND EVALUATIONS OF MODEL

Level of services of diamond interchange are mainly related with through and left turn traffic volumes and traffic signals at the two at-grade intersections. The evaluations of the proposed models were undertaken comparing average delay times resulted from TSIS under a variety of traffic conditions. The evaluations were performed for 9 traffic volume scenarios:

- 3 scenarios for total traffic volume (i.e. 80% for Volume 3, 100% for Volume 2, 120% for Volume 1)
- 3 scenarios for the ratio of left-turn traffic to the total traffic volume, respectively (i.e. 10% of left turn, 20% of left turn, 30% of left turn traffic to each total traffic volume)

Test intersections were depicted in the Figure 2 for TUDI and Figure 3 for proposed TUDI in section 1 and 2, respectively. We assumed some variables: saturation flow rate for each movement is 1800 pcphpl, yellow interval 3 seconds, and the internal link length widely used, 70m. The traffic volume scenarios were illustrated in Table 1. LINDO package was employed to obtain solutions of BMILP for the proposed TUDI and a delay-based model of TRANSYT-7F for TUDI. BMILP for optimizing phase sequence and TRANSYT-7F for optimizing green intervals were applied to the combined model for the proposed TUDI. In any case, TSIS microscopic simulation package was employed to evaluate signal timing solutions.

Table 1. Traffic Volume Scenarios (vph)

		10% of Left turn	20% of Left turn	30% of Left turn
Volume 1	Left turn	180	360	540
	Through	1260	1080	900
	Right turn	360	360	360
Volume 2	Left turn	150	300	450
	Through	1050	900	750
	Right turn	300	300	300
Volume 3	Left turn	120	240	360
	Through	840	720	750
	Right turn	240	240	240

Table 2 illustrates simulation results by TSIS for 10 % of left turn to total volume. It shows that the delay times of the two proposed models for the proposed TUDI are much less than those of TRANSYT-7F for TUDI, where the more traffic volume, the greater the effect. Combined model is superior to BMILP in reducing delay time for 10 % of left turn to total volume.

Table 2. Average Delay Time for 10% of Left Turn Volume to Total Traffic Volume (sec/veh)

	TRANSYT (A)	BMILP (B)	Combined (C)	A-B	A-C	C-B
Volume 1	57.1	33.5	11.6	23.6	45.5	-21.9
Volume 2	42.4	33.4	10.1	9.1	32.3	-23.2
Volume 3	19.6	14.5	8.8	5.0	10.8	-5.8

Using the proposed two models, Table 3 illustrates that the effects of reducing delay time for 20% of left turn volume to total volume are greater than those for 10% of left turn volume to total volume, where the effects are not consistently increased as the traffic volumes increased. Anyway the effect for Volume 1 is the best. Combined model is superior to BMILP in reducing delay time similarly to Table 1, where the effect is much less for Volume 2 and

volume3 but greater for Volume 1.

Table 3. Average Delay Time for 20% of Left Turn Volume to Total Traffic Volume (sec/veh)

	TRANSYT (A)	BMILP (B)	Combined (C)	A-B	A-C	C-B
Volume 1	111.7	49.5	16.3	62.3	95.4	-33.1
Volume 2	69.4	16.8	12.9	52.6	56.5	-4.0
Volume 3	68.2	12.9	11.3	55.4	56.9	-1.5

Table 4 illustrates the two proposed models greatly decrease delay times all over the test volumes. The more traffic volume, the much greater the effect. Contrary to Table 1 and 2, BMILP is superior to combined model in reducing delay time, where the more traffic volume, the larger the effect. According to Table 1, Table 2 and Table 3, the effects of reducing delay time using proposed models are best for 30% of left turn volume to total volume.

Table 4. Average Delay Time for 30% of Left Turn Volume to Total Traffic Volume (sec/veh)

	TRANSYT (A)	BMILP (B)	Combined (C)	A-B	A-C	C-B
Volume 1	149.8	21.4	50.2	128.4	99.6	28.8
Volume 2	109.2	14.5	21.1	94.7	88.1	6.5
Volume 3	83.8	12.1	15.5	71.7	68.3	3.4

As illustrated in Table 5, the more the total volume, the greater the delay time for all models. Similarly the more the left turn volume, the larger the delay time for TRANSYT-7F and Combined model; the increasing amount of the delay for TRANSYT-7F is much larger than that for Combined model. It is noticed that influence on the delay of left turn volume is greater than that of total volume. On the other hand, the delay time is decreased as the left turn volume increases for BMILP. Therefore BMILP is superior both to TRANSYT-7F and to Combined model for high left turn volume.

Table 5. Average Delay Time for Models (sec/veh)

	Left turn 10%			Left turn 20%			Left turn 30%		
	TRANSYT	**BMILP**	**Combind**	**TRANSYT**	**BMILP**	**Combind**	**TRANSYT**	**BMILP**	**Combind**
Volume 1	57.1	33.5	11.6	111.7	49.5	16.3	149.8	21.4	50.2
Volume 2	42.4	33.4	10.1	69.4	16.8	12.9	109.2	14.5	21.1
Volume 3	19.6	14.5	8.8	68.2	12.9	11.3	83.8	12.1	15.5

5. CONCLUSION AND RECOMMANDATIONS

In this paper a new geometric design and signal optimization models were proposed to cope with the problem for the left turn traffic from minor road to ramp in TUDI. The proposed design and traffic control allows the left-turn traffic to make left-turn and enter to the ramp at the upstream intersection. As the result the left-turn traffic doesn't have to go to the downstream intersection, the proposed method handles less traffic volumes than those of TUDI at the two intersections. Next we proposed two types of signal timing optimization models for the proposed TUDI. One is BMILP model that optimizes all signal timings for the proposed TUDI. The other is combined model that optimizes green intervals by TRANSYT-7F subject to the phase sequence and offset resulted from the BMILP.

Based on the simulation results, the two proposed models for the proposed TUDI certainly be superior to TRANSYT for TUDI all over the test scenarios, where the more traffic the greater the effect. These advantages are most pronounced when there are heavy through traffic and left-turn traffic at the two at-grade intersections. For the two proposed models, combined model was superior to BMILP for 10% and 20% of left turn volume to total volume, vice versa for 30% of left turn volume to total volume. The delay time was decreased as the left turn volume increased in BMILP, vice versa in combined model. It is therefore noticed that combined model is recommended for low left turn volume but BMILP for high left turn volume.

The evaluations of the proposed models were undertaken based upon a variety of traffic conditions. Level of service of the proposed diamond interchange mainly depends on the traffic conditions and the length of the internal link. Therefore further evaluations of the proposed models will be undertaken varying the length of the internal link. Since the proposed geometric design for diamond interchange is different from that of CDI, field validation of the proposed design must be carried out prior to implementation, and further investigation is needed to design facilities that requested to separate the left turn traffic from minor road to ramp and opposing through traffic and to design information system for the drivers to safely make left turns in the proposed design. Further investigation is needed to develop the signal timing optimization model reflecting dynamic transition of the queues in the internal links and traffic fluctuations in the external links.

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