□論 文□

과포화 다이아몬드형 인터체인지의 교통신호제어모형의 개발

Traffic Signal Control for Oversaturated Diamond Interchanges

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목 차

요 약

다이아몬드형 인터체인지는 고속도로와 노면가로가 교차할 경우 방향별 교통류를 처리하는데 보편적으로 사용하는 인터체인지이다. 인터체인지에 교통량의 부하가 과도해지면 인터체인지내부의 교차로에서 발생한 대기차량이 종종 고속도로본선으로 역류하여 본선의 교통소통에 문제를 야기하며 특히 고속도로의 안전에도 큰 위험요소가 된다.

본 논문은 과포화상태의 다이아몬드형 인터체인지의 교통신호제어를 다루며, 신호시간계획을 산출하는 動的 最適化模型(dynamic optimization model)을 제시한다. 최적화 모형은 지체도최소화를 목적함수로 하며, 整數計劃法의 형태가 된다. 본 모형의 핵심은 신호제어에 따라 발생하는 대기차량길이를 모형화하여 대기차량길이가 정해진 상한치를 초과하지 않도록하는 신호시간계획을 산출하는데 있다. 또한 신호시간의 빈번한 전이에 따른 문제점을 해소하는 제어전략을 제시한다.

제시된 동적모형은 다이아몬드형 인터체인지의 신호시간을 위하여 널리 사용되는 PASSER Ⅲ와 최적해를 상호비교한다. TRAF-NETSIM을 통한 시뮬레이션의 결과에 따르면 동적모형이 우수한 결과를 보이며, 대기차량의 길이를 효과적으로 제어하는 것으로 판명된다.

I. INTRODUCTION

1. PROBLEM STATEMENT

The signalized diamond interchange is a widely used form of freeway-to-street interchange. A typical diamond interchange is shown in Figure 1. The design includes one-way frontage roads and U-turn bays, and is different from the urban arterial with two intersections in that left-turn bays on the internal links extend to the upstream links. Efficient movement of traffic through a interchange is critical in maintaining an acceptable level of service in the freeway corridor. During high-volume and possibly saturated conditions, inappropriate traffic control may produce long queues and excessive delays. Long queues can become a safety problem when the ramp (or frontage road) queues overflow onto busy freeway mainline, or arterial queues block adjacent intersections, as demonstrated in Figure 2.

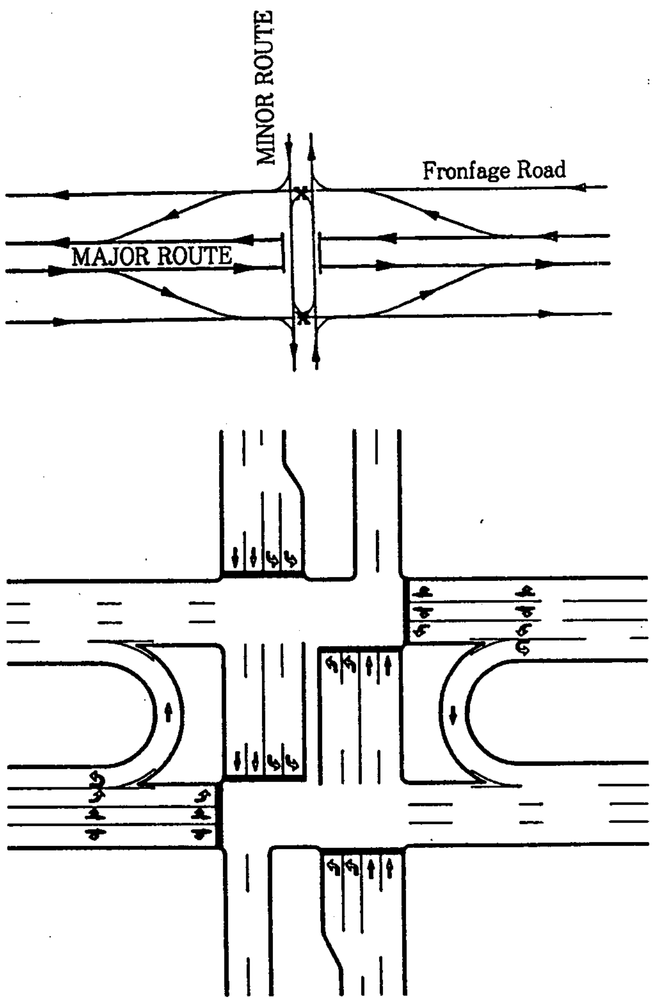


Figure 1. Diamond Interchange

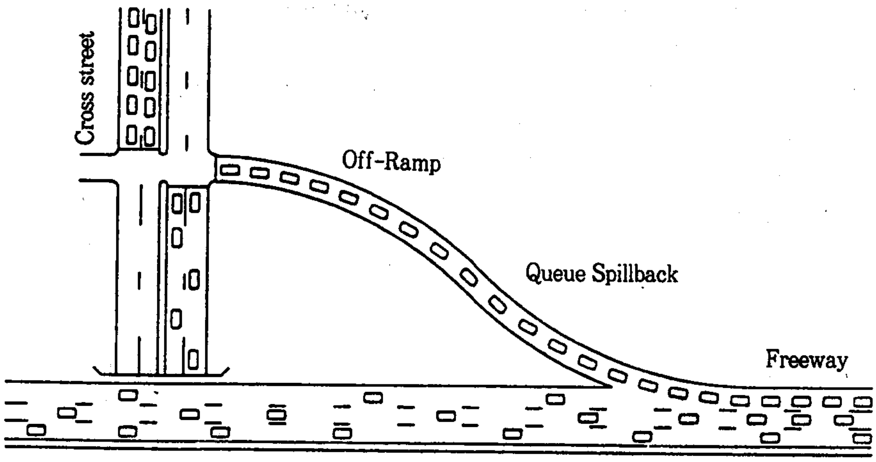


Figure 2. Queue Spillback to Freeway Mainline

Pignataro et al. (1) defined *Congestion* as a condition in which all waiting vehicles cannot pass through the intersection in one signal cycle. They also defined *saturation*(or *oversaturation*) as the condition when vehicles are prevented from moving freely, either because of the presence of vehicles in the intersection itself or because of back-ups in any of the exit links of the intersection. In interchanges, oversaturation occurs when traffic demand exceeds interchange capacity. Queues fill entire blocks or exit ramps, and interfere with the performance of adjacent facilities when this heavy demand continues for a long time period. Sometimes freeway exit ramps are blocked by the extended queues. Queue spillback to freeway mainlines may occur in the heavily loaded interchanges. In oversaturated conditions, congestion is unavoidable, thus the control policy should be aimed at postponing the onset and/or the severity of secondary congestion caused by the blockage of adjacent intersections or freeway off-ramps that are not the originators of the congestion.

Control strategies have been developed and applied successfully for the control of undersaturated signalized interchages, but most of them appear to be ineffective or invalid when traffic volumes become excessively high. Traffic engineering models like PASSER Ⅲ(2) and TRANSYT-7F(3) have been produced to assist traffic engineers in developing signal timing plans for signalized diamond interchanges. None of these are applicable to oversaturated environments. It would be almost impossible to modify these programs to produce an optimal control policy for oversaturated interchanges. There is an urgent need to develop optimal control strategies for oversaturated diamond interchagnes.

2. RESEARCH OBJECTIVES

The goal of this research was to develop an optimization model to provide optimal traffic signal control policies for oversaturated dismond interchagnes. Generally the control objective of traffic signal timing has been to obtain maximum bandwidth and/or minimum delay. However, the control objective for oversaturated environments should be to maximize throughput in the system during the control period, *i. e.*, the ***productivity***. When demands are extremely high, the control policy should be such that queue lengths on all internal links of the roadway system do not exceed queue storage capacity, and simultaneously all available green times are utilized as fully as possible in order to obtain maximum system productivity. This control objective has been pursued for the control of freeway on-ramps*(4, 5)*,

For the control of oversaturated interchanges, the optimization model should have the capability of controlling queue lengths on external approaches. When traffic demand exceeds interchange capacity, the queue formation on specific approaches depends on the magnitude and duration of the heavy demand. Consequently, the traffic engineering model should be dynamic to accommodate the variation in demand during the control period.

The specific objectives of this research were as follows.

1. Develop dynamic optimization models which have a queue management capability for signalized diamond interchagnes; and
2. Use the dynamic models to develop optimal control strategies for the oversaturated diamond interchages and evaluate the control strategies.

Ⅱ. DYNAMIC OPTIMAIZATION MODEL

1. CONTROL STRATEGIES

Suppose the queue storage capacities on the external approaches to a diamond interchange are limited and the queue spillback to the intersections or freeway mainlines adjacent to the interchange can cause severe operational problems. As the main objective of traffic signal control during oversaturated conditions is to obtain maximum system productivity, this control objective can not be accomplished without considering potential queue spillback on the external approaches. This spillback may cause serious operational problems on the interchange, including the adjacent intersections or thransportation facilities.

The two-fold control objectives proposed by Michalopoulos *(6)* appear reasonable to address this condition. First, the queues devleoping on external approaches must be restricted so that adjacent upstream intersections are not blocked. Second, total system delay during the entire control period should be minimized. It is believed that control strategies satisfying these objectives would also maximize total system productivity.

It is evident that queue formation depends largely on the magnitude and duration of the heavy demand. Conventional static models like PASSER Ⅲ (2) and TRANSYT-7F(3) that use one set of demand data during the entire control period are not appropriate for dynamic queue management. These static models consider only the average magnitude of heavy demand in signal timing optimization, not is duration. The optimization model for queue management should accommodate dynamic variation in traffic demand.

The following assumptions were used in the mathematical formulation of the dynamic model:

1. The traffic demand during each time slice is uniform;

2. The control period is divided into several time slices of 15 minutes; and

3. Demands are known for all time slices during the entire control period.

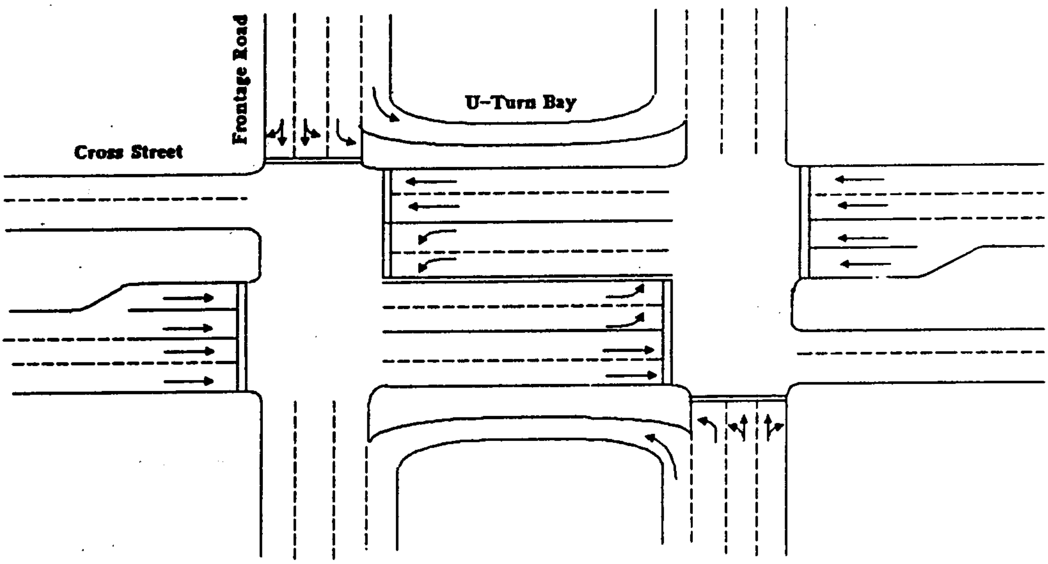
The dynamic model was formulated to obtain minimum delay subject to queue length constraints. The model includes formulas to define relationships of queue carryover between time slices. The model has two groups of constraints; one for individual time slices and the other for the control of queue lengths. The constraints adjust green indication times between time slices and approaches so that maximum queue length does not exceed the allowable storage capacity. In the dynamic model, signal timing plans, usually the green splits, change with every time slice. Green splits are adjusted toward minimizing delay and permitting queues to build to a predetermined upper bound.

One special feature of the dynamic model is that the signal timing plan chagnes for every time slice, usually every 15 minutes in response to changing traffic patterns. Since average traffic flows vary with time, it tis desirable to change signal timing accordingly. Such frequent chagnes in a fixed-time signal system, however, may sometimes cause serious operational problems. To change from one timing plan to the next, phase durations may meed to be either lengthened, shortened or possibly even omitted. During this process, excessive delay or unexpected intersection blockage can be caused by the loss of green time on some approaches and the loss of coordination for the entire network. Efficient methods for changing plans can minimize this transient delay. An effort to reduce these transition problems is addressed in the following paragraphs.

The 4-phase overlap signalization strategy (7) has an advantage over three-phase strategies for geometric design shown in Figure 3. A major advantage of the 4-phase strategy is that it generally does not produce queues on the internal links, particularly if no frontage road(or exit ramp) U-turns occur. That is, the 4-phase strategy results in nearly perfect progression between two closely spaced signalized intersections within the interchange, as shown in Figure 4.

An effort to reduce the signal timing transition problem is addressed in this section. In the dynamic model, green splits change every time slice, usually at 15-minute intervals. TRAFNETSIM (8) allows users to input a series of timing plans during a simulation period for fixed time controllers. TRAF-NETSIM also provides three signal transition options: immediate, two cycle, and three-cycle transitions.

Becaise frequent changes of the green times may cause operational problems at conventional diamond interchanges, a special coding scheme of the 4-phase overlap strategy in TRAF-NETSIM was prepared to minimize the problem, as shown in Figure 5. The phase sequence of 4-phase overlap strategy is ABC : ABC. Phases of the left intersection are normally coded as a sequence of ABC. Signal timing at the right intersection starts with Phase B of the duration identical to the overlap phase. Phases C and A are coded in intervals 2 and 3, respectively. Phase B is coded in interval 4 with the duration of Phase-B duration minus overlap, therfore, the sum of intervals 1 and 4 is the Phase-B duration. The offset between the first interval of the two intersections must be zero so as to insure perfect progression.



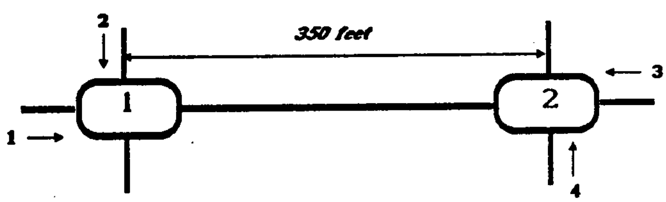


Figure 3. Diamond Interchange

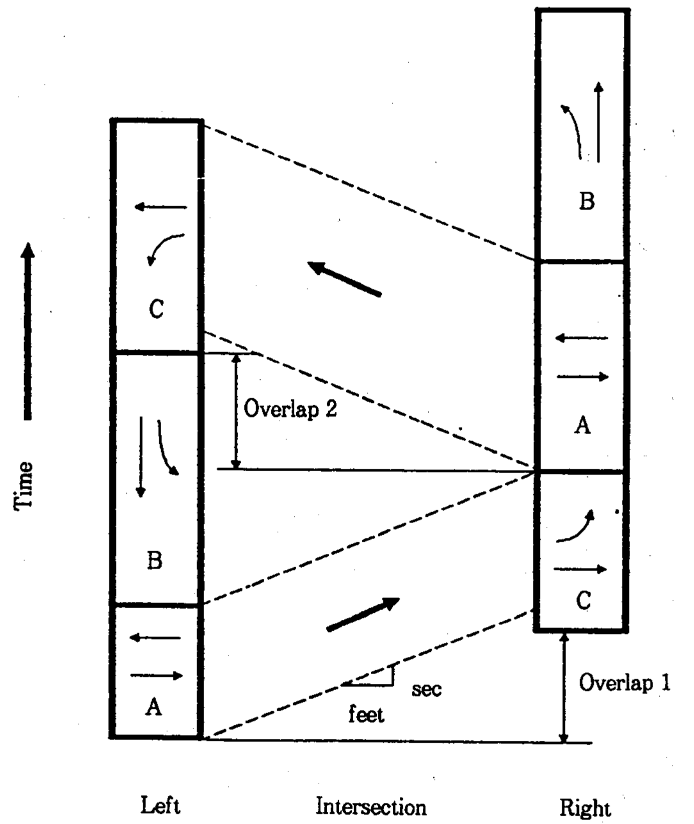


Figure 4. Four-Phase with Overlap at Diamond Interchange

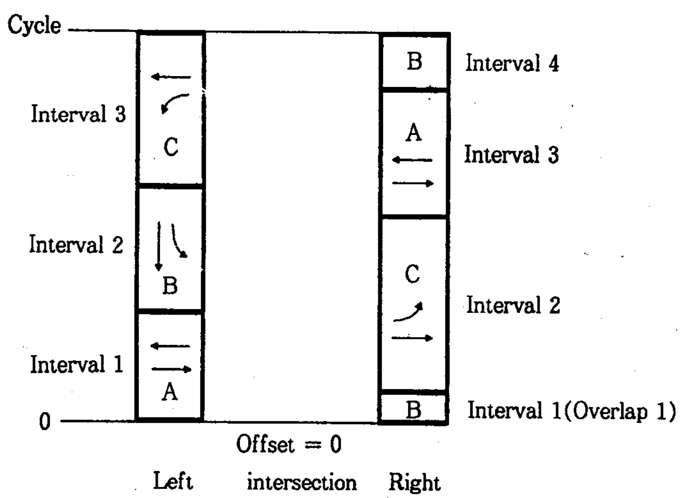


Figure 5. NETSIM Coding of 4-Phase-Overlap

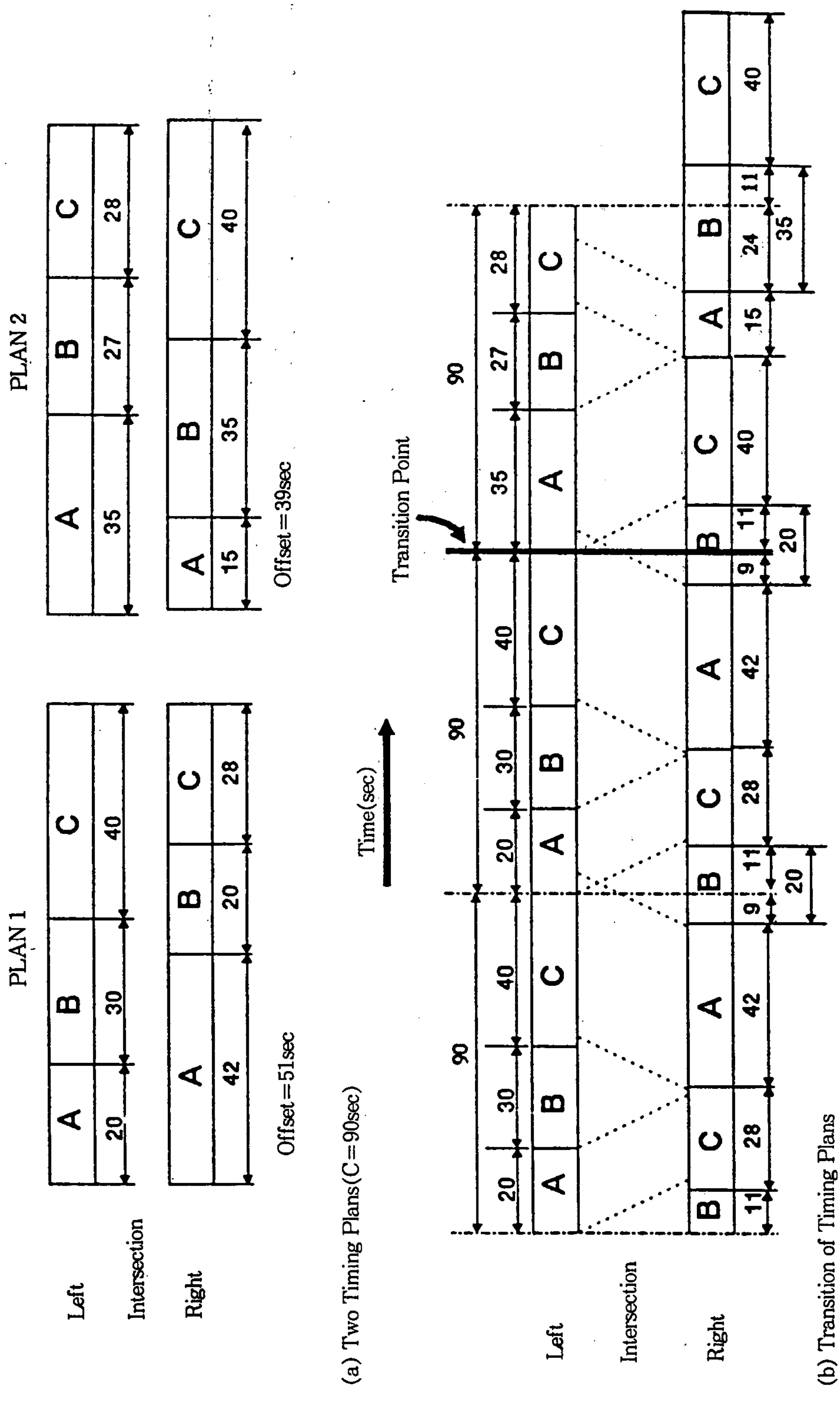


Figure 6. Transition of 4-Phase-Overlap

Immediate transition should be applied in TRAF-NETSIM for the coding scheme to be effective in reducing the transition delay. Figure 6 illustrates how the coding scheme works during the transition period. Time Slice 1 is followed by Time Slice 2. Figure 6(a) shows two timing plans obtained from the 4-phase overlap strategy. Signal plans change suddenly at the end of Time Slice 1. Plan 1 changes into Plan 2 without losses of green time and progression, as shown in Figure 6(b). This coding scheme minimizes transient delay.

2. Formulation

The numbering scheme of movements and the signal phase scheme are shown in Figures 7 and 8. The follwing notation is used in the formulation:

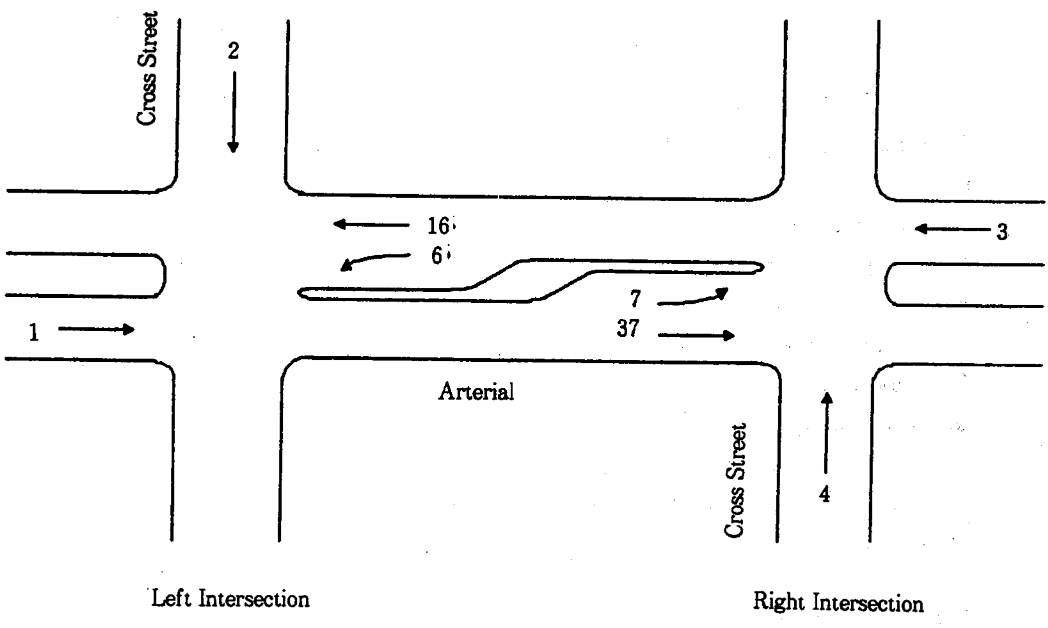


Figure 7. Arterial with Two Intersecions

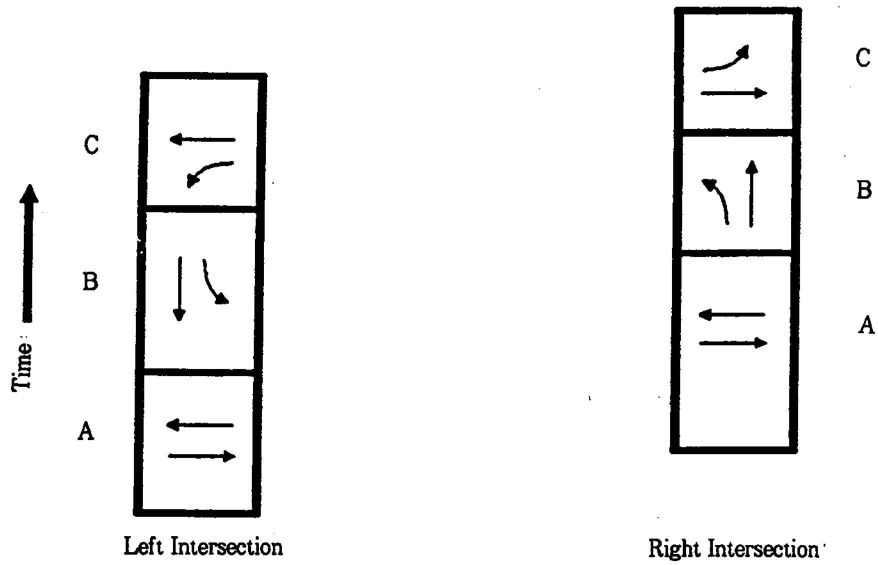


Figure 8. Three Basic Phases and Phase Sequence

i = signal phase and/or movement, i=1,2,3,4,6,7

j = time slice of duration $\triangle T$,

C = system cycle length, sec,

l = lost time per phase, sec,

G_{ij} = effective green time of phase i at time slice j, sec,

g_{ij} = G_{ij}/C, green ratio normalized to cycle length,

ϕ = one-direction overlap, sec,

V_{ij} = average input volume on approach i at time slice j, vps,

S_i = saturation flow of approach i, vpsg,

P_{ikj} = proportion of turning movement, as shown in Figure 9,

L_{ij} = queue length of external phase i at the end of time slice j, veh,

N_j = queue storage capacity of external phase j, veh,

D_i = delay for external approach i, veh-min,

TD = total external delay, veh-min,

$Z_{ji} = \begin{cases} 0 \text{ when approach is undersaturated, and} \\ 1 \text{ when approach is oversaturated} \end{cases}$

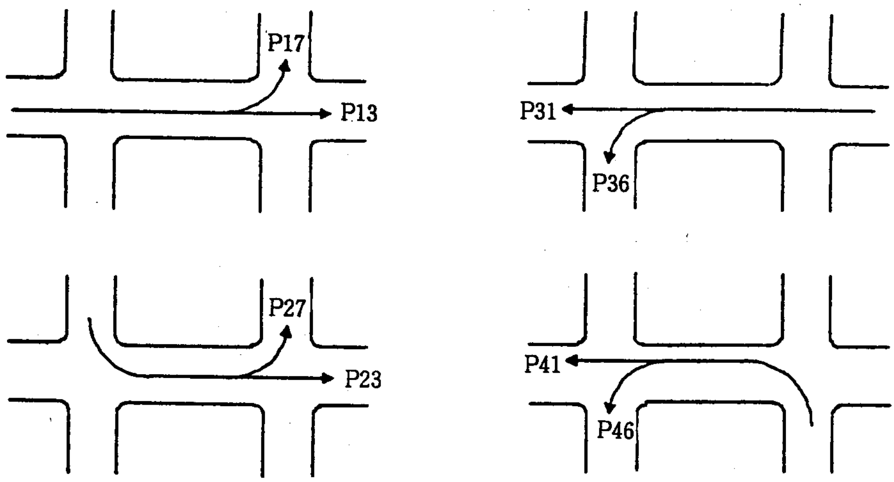


Figure 9. Notation for Turning Percentages.

Objective Function

The control objective of the dynamic model is to minimize total external delay. Total external delay is the sum of delays on all external approaches. Delay on individual approach i, D_i, is equivalent to the area formed by the X-axis and queue length line, as shown in Figure 10. Namely.

$$D_i=(1/2)\{(L_{i0}+L_{i1})+(L_{i1}+L_{i2})+\cdots+(L_{in-1}+L_{in})\}\triangle T$$
$$=(1/2)(L_{i0}+2L_{i1}+2L_{i2}+\cdots+2L_{in-1}+L_{in})\triangle T \quad (1)$$

where n is the number of time slices. If an undersaturated traffic condition before the initiation and after the termination of the control period is assumed, then:

$$L_0=L_n=0$$

and,

$$D_i=\triangle T\sum_{j=1}^{n}L_{ij} \qquad \text{for all i (2)}$$

Because the duration of the time slice is a fixed $\triangle T$, the product of the duration of the time slice and the sum of the queue lengths over all time slices is equivalent to total delay on the individual approach. Therefore, the objective function to minimize total external delay, TD, is:

$$\text{Minimize} \quad TD=\triangle T\sum_{\text{all } i}\sum_{j}L_{ij} \qquad (3)$$

Constraints

Set 1. In Coordinated signal systems, the sum of phase durations at the individual intersections must be equal to the system cycle length, C. For three-phase signals, this requirement leads to:

$$G_{1j}+G_{2j}+G_{6j}+3l=C \qquad \text{for all j (4a)}$$
$$G_{3j}+G_{4j}+G_{7j}+3l=C \qquad \text{for all j (4b)}$$

or, dividing by C to normalize the green splits:

$$g_{1j}+g_{2j}+g_{6j}=1-3l/c \qquad \text{for all j (5a)}$$
$$g_{3j}+g_{4j}+g_{7j}=1-3l/c \qquad \text{for all j (5b)}$$

Set 2. For the internal links, the input to the

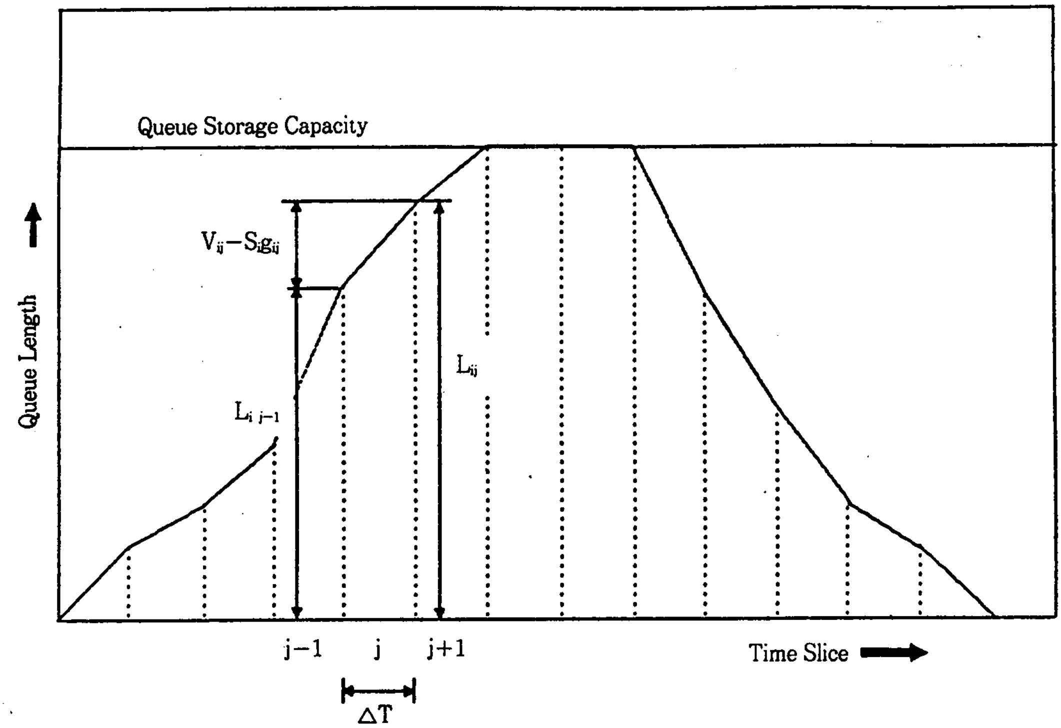


Figure 10. Queue Profile on Approach "i"

system must not be greater than the output in order to stabilize queue lengths over many cycles:

$S_1P_{17}g_{1j}+S_1P_{27}g_{2j}\leq\alpha S_7g_{7j}$ for all j (6a)

$S_1P_{13}g_{1j}+S_2P_{23}g_{2j}\leq\alpha S_{37}(g_{3j}+g_{7j})$ for all j (6b)

$S_3P_{36}g_{3j}+S_4P_{46}g_{4j}\leq\alpha S_6g_{6j}$ for all j (6c)

$S_3P_{31}g_{3j}+S_4P_{41}g_{4j}\leq\alpha S_{16}(g_{1j}+g_{6j})$ for all j (6d)

where α is an adjustment factor of saturation flow, usually not greater than 1.

Due to the complex lane configurations sometimes found on internal links and the stochastic nature of lane utilization, vehicles sometimes cannot fully utilize the available lanes. In this situation, it is desirable to adjust saturation flows using an α factor of less than 1. In most cases, however, the α factor should be 1. Smaller factor values give larger green times for internal phases, resulting in smaller green times for external phases.

Set 3. The 4-phase-overlap signalization strategy widely used in Texas was adopted for traffic control of the oversaturated diamond interchange in this research:

$G_{6j}+G_{7j}+2l=C-2\phi$ for all j (7)

or,

$$g_{6j}+g_{7j}\frac{(C-2\phi-2l)}{C} \qquad \text{for all } j \ (8)$$

Set 4. The queue lengths occurring at the end of each time slice must be non-negative. The non-negativity is achieved by letting:

$$L_{ij}=\text{Max}\{0,\ L_{ij-1}+(V_{ij}-S_ig_{ij})\triangle T\} \qquad \text{for all i, j (9)}$$

which is equivalent to:

$$L_{ij}\geq 0 \qquad \text{for all i, j}$$

$$L_{ij}\geq L_{ij-1}+(V_{ij}-S_ig_{ij})\triangle T \qquad \text{for all i, j}$$

$$L_{ij}\leq MZ_{ij} \qquad \text{for all i, j}$$

$$L_{ij}-\{L_{ij-1}+(V_{ij}-S_ig_{ij})\triangle T\}\leq M(1-Z_{ij}) \qquad \text{for all i, j}$$

where $M > 0$ is sufficiently large such that $L_{ij}\leq MZ_{ij}$ is redundant with respect to any active constraint. Z_{uj} is an integer variable having binary values.

As demonstrated in Figure 10, queue length on external approach i at the end of time slice j, l_{ij}, is the sum of any queues transferred from the previous time slice and the difference between input and output at the current time slice; namely:

$$L_{uj}=L_{ij-1}+(V_{ij}-S_ig_{ij})\triangle T \qquad (10)$$

The queue length estimation of Equation 10 is based on the Input-Output Analysis methodology. If $V_{ij} \geq S_ig_{ij}$, the queue length increases; otherwise, the queue length decreases. When an approach becomes undersaturated, the right-hand side of Equation 9 can have a negative value, as illustrated in Figure 11. Suppose the queue dissipates at time slice j and then grows again at time slice j+1. The actual profile of the queue length follows line ABCDE in Figure 11. Without the non-negativity constraints on L_{ij}, the dynamic model may predicts an erroneous queue profile along line ABGHI. This prediction causes some false estimation of the queue length starting from time slice j+1.

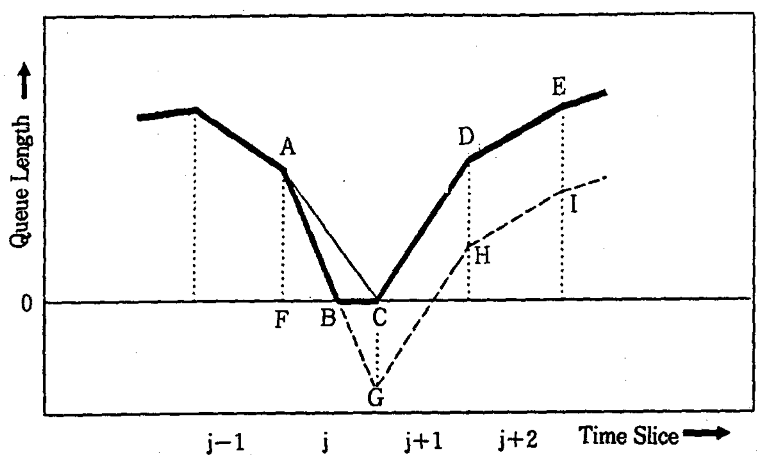


Figure 11. Role of Non-Negativity Constraints

This problem is solved by adding non-negativity constraints to the calculation of queue lengths in the dynamic model. The non-negativity constraints make the model estimate the queue profile along line ACDE. The queue length starting from time slice j+1 is estimated reasonably by adding the non-negativity constraints to the formulation. Actual delay at time slice j is the area of AFB, whereas the dynamic model slightly overestimated the delay as the area of AFC. The overestimation has little effect on the optimal solution because the overestimated delay is usually minor compared to the total delay. Integer variables, Z_{uj} were introduced for modelling the non-negativity of the queue lengths; thus, the dynamic model take a form of Mixed Integer Linear Programming (MILP)

Set 5. The queue lengths at the end of each time slice must not exceed available queue storage capacity of their respective external links:

$$L_{ij} \leq \beta N_i \quad \text{for all i, j} \quad (11)$$

where β is the adjustment factor for queue storage, usually not greater than 1. The adjustment factor, β, makes the queue storage capacity smaller than it actually is so as to provide a storage buffer to absorb some natural fluctuations in demand. The queue storage capacity of external link, N_i is an upper limit for queue length on approach i. This storage capacity is calculated using the following equation:

$$N_i = \frac{(\text{Link Length, feet}) \times (\text{Number of Lanes})}{(\text{Vehicle Storage Length, feet})} \quad (12)$$

Set. 6 The green time must be greater than the minimum green time:

$$g_{uj} \geq g_{i\ min} \quad \text{for all i j} \quad (13)$$

where g_{imin} is minimum green ratio for phase i. The minimum green ratio can be determined from pedestrian crossing requirements or driver expectancy considerations.

3. DISCUSSION

The dynamic model proposed was designed to produce optimal signal timing plans for oversaturated traffic conditions. This model may generate an undesirable solution for undersaturated conditions. Even if the rush hour is selected as the control period, traffic conditions may be undersaturated during some time slices. The dynamic model solution for these time slices may not be desirable because it sometimes assigns only minimum green time to an approach and excessive green times to the other approaches. The signal timing may result in unbalanced levels of service between the approaches. One can find the undersaturated time slices by analyzing the optimal solution of the dynamic model and handle this problem as follows:

The integer variable Z_{ij} indicates the traffic condition on the associated approaches. If Z_{ij} is equal to zero for all approaches i at time slice j, then the time slice j is an undersaturated time slice. It is desirable to adjust the solution of the LINDO output for the undersaturated time slices. The dynamic model solution for these time slices should be replaced by a green split based on flow ratio. This green split can be calculated using the conventional static model like PASSER Ⅲ.

When heavy traffic demand lasts for a long period of time and queue storage capacity is limited, the dynamic model has an advantage over conventional static models due to its queuecontrol capability. Users may want to constrain the queues of all competing approaches to predetermined upper limits. The dynamic model, however, may not always produce a feasible solution if the queues on all external approaches are bounded.

If heavy demand lasts for a long time period, the dynamic model attempts to assign green times to commpeting approaches and critical time slices to reduce queue lengths. When available green times are exhausted, all queue storage capacities are full of stopped vehicles, but demand for service still remains. The dynamic model cannot provide a feasible solution for this situation, due to its physical queue constraints. In this case, users must increase the queue constraint limits until the model produces a feasible solution. The dynamic model always produces a feasible solution if the queue length of at least one approach is unbounded.

Ⅲ. EVALUATION

The dynamic optimization model was proposed for traffic control of congested signalized diamond interchanges. In this section the performances of the model were evaluated by comparing them with current signal timing model.

1. STUDY APPROACH

The dynamic model were evaluated by comparing the performances of signal control strategies generated by the model to that of conventional model. Control strategies were generated twice; once using the conventional model and once using the newly developed dynamic model.

Three different test cases were generated for each type of roadway system. Case 1 was designed as a base case. It was prepared so that its demand level was slightly over the capacity calculated by the conventional model. Cases 2 and 3 were designed by increasing demand level and providing different time-varying demand profiles and/or origin-destination traffic patterns. Three-hour control periods divided into fifteen-minute time-slices were selected in designing the signal timing plans. PASSER Ⅲ (2) was selected as the conventional model for diamond interchanges. PASSER Ⅲ is a deterministic optimization model designed exclusively for the signal timing of conventional diamond interchanges.

The TRAF-NETSIM simulation of each control strategy was replicated five times using different random seed numbers. TRAF-NETSIM provides various measures of effectiveness (MOE'S). Among them the following measures were used for evaluation purposes:

1. Total travel(vehicle-miles) – the total distance traveled by all the vehicles released within the roadway system during the pre-determined control period,

2. Vehicles discharged(vehicles) – the number of vehicles exiting the roadway system during the control period,

3. Queue length(feet) – the distance occupied by stopped vehicles,

4. Average delay(seconds/vehicle) – the time lost per vehicle while traffic is impeded by traffic control devices, and

5. Stops per trip – the average number of stops experienced by vehicles released into the system.

Maximizing system productivity was a major control objective of the models proposed in this research. Total travel and vehicle discharge were the MOE's representing system productivity; thus, they were selected as major MOE's. The dynamic model's queue management capability was evaluated by examining queue lengths on external approaches. Average delay and stops, widely accepted MOE's were also used to evaluate the performance of specific links.

Statistical analyses were performed to test the superiority of the proposed model. A number of tests are available to test two samples. Nonparametric tests are appropriate when sample sizes are small and the normality assumption is not valid. In this research, five replications were conducted in simulating each signal control strategy. The same random seed numbers were used for the paired simulation trial of PASSER Ⅲ output and the dynamic-model output.

Hays(9) stated that the Mann-Whitney and Wilcoxon tests are generally regarded as the best of the order tests for two samples of all nonparametric tests. The Mann-Whitney test for two independent samples was not suitable since the samples in this research were paired. Consequently, the Wilcoxon rank-sum test for paired observations was selected for testing the simulation results. The null hypothesis of the statistical tests was that the dynamic model did not improve system performance as compared to the selected conventional models typically used by traffic engineers. The research hypothesis was that the dynamic model improved the system performance. Improved system performance resulted in increased total and vehicle discharge, but reduced delay, stops, and queue backup.

2. RESULTS

The performance of the dynamic model modified for diamond interchanges was evaluated by comparing it with results generated by PASSER Ⅲ. Three cases were prepared for the evaluation as follows:

Case 1. v/c=1.07, Simultaneous peak time, Heavy cross-street left-turn,

Case 2. v/c=1.13, Alternate peak time, Moderate cross-street left-turn, and

Case 3. v/c=1.13, Random Demand, Moderate cross-street left-turn.

Demand in Case 1 was slightly higher than interchange capacity. Peak hour volume (PHV) was raised for Cases 2 and 3. Simultaneous peak time in Case 1 means that the peak demands on the four external approaches occurred at almost the same time. Alternate peak time of Case 2 means that peak demands on the external approaches did not occur simultaneously. The PHV of Approaches 1 and 2 occurred between Time Slices 1 and 4; PHV of Approaches 3 and 4 occurred between Time Slices 6 and 9. Random Demand in Case 3 means that demand profiles for the approaches were uneven and irregular. When compared with the other test cases, left-turn traffic from the cross street to the frontage road was heavy in Case 1. Approximately 60 percent of total traffic at internal approach of the right intersection turned left. The left-turn traffic for Cases 2 and 3 was reduced to 40 percent.

Cycle length is an important signal timing variable. Messer (7) found that shorter cycle lengths produced larger interchange capacity in unconstrained diamond interchanges when the 4-phase overlap strategy was applied and total overlap was longer than total lost time. In this research an 11-second overlap and a 4-second phase lost time were used for each direction. Total overlap for both directions is 22 seconds and total lost time for the four external phases is 16 seconds. Because the total overlap is longer than the total lost time, short cycle lengths can increase interchange capacity for the 4-phase overlap strategy according to Messer's finding. A short cycle length, however, can create an oversaturation problem at the internal left turn phase, as illustrated Table 1.

Table 1. SIGNAL TIMING GENERATED BY PASSER Ⅲ FOR CASE 2

		Left-Side Intersection			Right-Side Intersection		
		Phase Duration(second)					
Cycle Length	v/c Ratio	A	B	C	A	B	C
55sec	1.09	15.6	20.6	18.8	25.6	15.2	14.2
90sec	1.14	22.1	30.2	37.7	38.0	21.7	30.3

Table 1 illustrates signal timing plans generated by PASSER Ⅲ for Case 2 using the 4 phase overlap strategy. PASSER Ⅲ produced the minimum delay cycle length of 55 seconds for Case 2. PASSER Ⅲ chooses the minimum delay cycle by examining a range of feasible cycle lengths. In the 55-second cycle length, Phase A of the right side intersection is much longer than Phase C of the left side intersection. This signal timing causes oversaturation at the internal left-turn phase (Phase C) of the left side intersection when Phase A of the right side intersection is fully utilized. By increasing the cycle length was used for Cases 1 and 2 for this reason. In Case 3, a 75-second cycle length could eliminate the oversaturation problem and was used for the PASSER Ⅲ run. For the dynamic model, a 90-second cycle length was used for all cases.

Table 2 summarizes the results of simulation using TRAF-NETSIM. It can be seen that total travel and vehicle discharge with the dynamic model are consistently larger than those obtained using PASSER Ⅲ. The dynamic model increased delay by 2 percent in Case 1, while it decreased delay by 13 percent in Case 2, and by 14 percent in Case 3. As expected, this result means the dynamic model is more favorable when peak demands do not occur simultaneously among the approaches.

The dynamic model assigns large green times to an approach in peak traffic when the peak demands for the competing approaches occur alternately. In the next time slice, the large green time is assigned in a timely manner to other approaches experiencing heavy demand. if the peak demands occur at the same time, the dynamic assignment of the green times produces limited effectivenesss when compared to the conventional static model. The dynamic modle attempts to minimize total delay and to constrain maximum queue lengths. A model simply minimizing total delay without controlling queues would produce less delay than a model minimizing total delay as well as constraining queue lengths. The dynamic model can control queue lengths, but cannot reduce total delay for the case of simultaneous peak traffic demands.

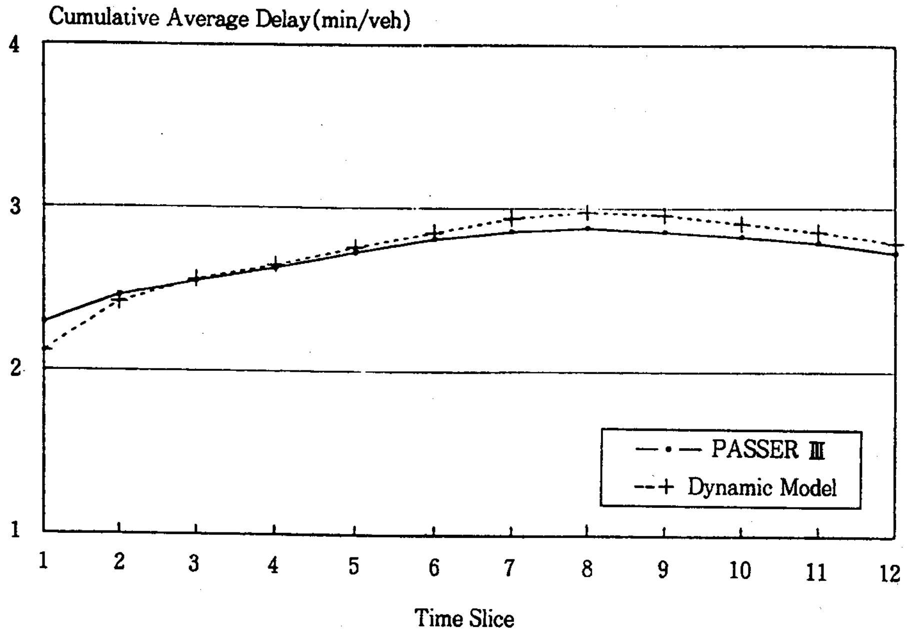
Table 2. Comparison Of Performances Between Passer Ⅲ And Dynamic Model Using Traf-Netsim

	Total Travel (veh-mile)	Vehicles Discharged	Delay (min/veh)	Stops per Trip	Queue Backup(sec)
Case 1. V/C=1.07, Simultaneous Peak Time, Heavy Cross-Street Left-Turn					
PASSER Ⅲ	8,542	16,483	2.73	1.5	0
DYNAMIC MODEL	8,560	16,521	2.79	1.5	0
% DIFFERENCE	+0.2	+0.2	+2	0	
Improve ?[1)]	yes	yes	no	no	
Case 2. V/C=1.13, Alternate Peak Time, Moderate Cross-Street Left-Turn					
PASSER Ⅲ	9,008	17,559	3.51	1.5	0
DYNAMIC MODEL	9,171	17,791	3.04	1.5	0
% DIFFERENCE	+1.8	+1.3	−13	0	
Improve?	yes	yes	yes	no	
Case 3. V/C=1.13, Random Demand, Moderate Cross-Street Left-Turn					
PASSER Ⅲ	8,814	17,391	2.08	1.3	0
DYNAMIC MODEL	8,876	17,521	1.78	1.3	0
% DIFFERENCE	+0.7	+0.7	−14	0	
Improve?	yes	yes	yes	no	

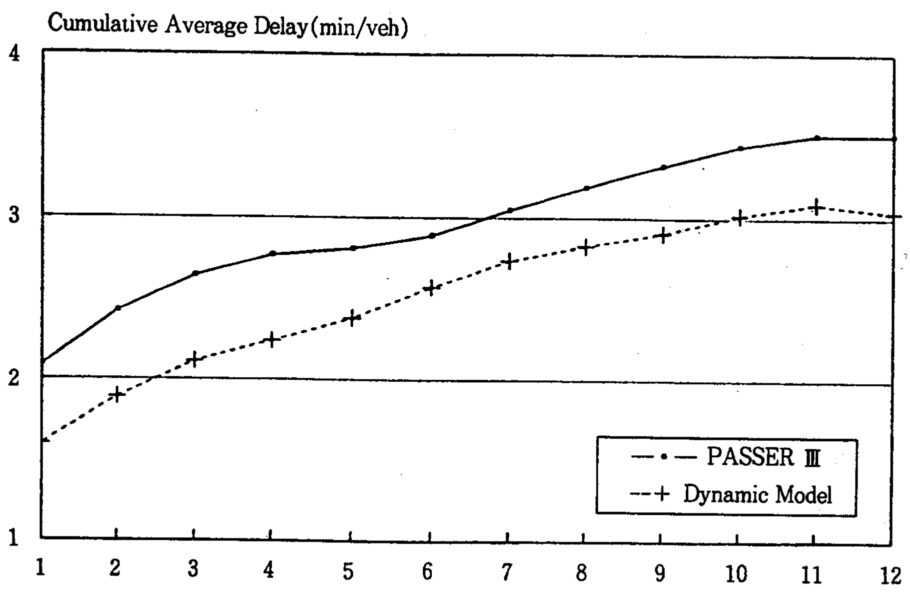
1) Results of Wilcoxon rank-sum test
Research Hypothesis : Dynamic Model improved system performance.
Significance Level=0.05
Sample Size=5

Figure 12 shows the cumulative average delay for the total interchange, as estimated by TRAF-NETSIM. In Case 1, delay obtained using the dynamic model increases faster than that obtained using PASSER Ⅲ. In Case 2, delay produced using the dynamic model is consistently smaller than that produced using PASSER Ⅲ during the entire control period. No differences were observed in the number of stops. No queue spillback was estimated for either model since its elimination is a major advantage of the 4-phase overlap strategy.

Figure 13 shows queues on the external approaches as estimated from the Input-Output model in Case 1. In this figure, the queue length data points for the dynamic model were derived form the solution of the dynamic model. The dynamic model estimates queue lengths using the Input-Output model. Input is the time-slice demand and output is the number of vehicles discharged at the stop line. The difference between input and output is the queue length estimated by the dynamic model. The queue lengths for PASSER Ⅲ were also estimated



(a) Case 1



(b) Case 2

Fagure 12. Cumulative Delay at TUDI(Cases 1 and 2)

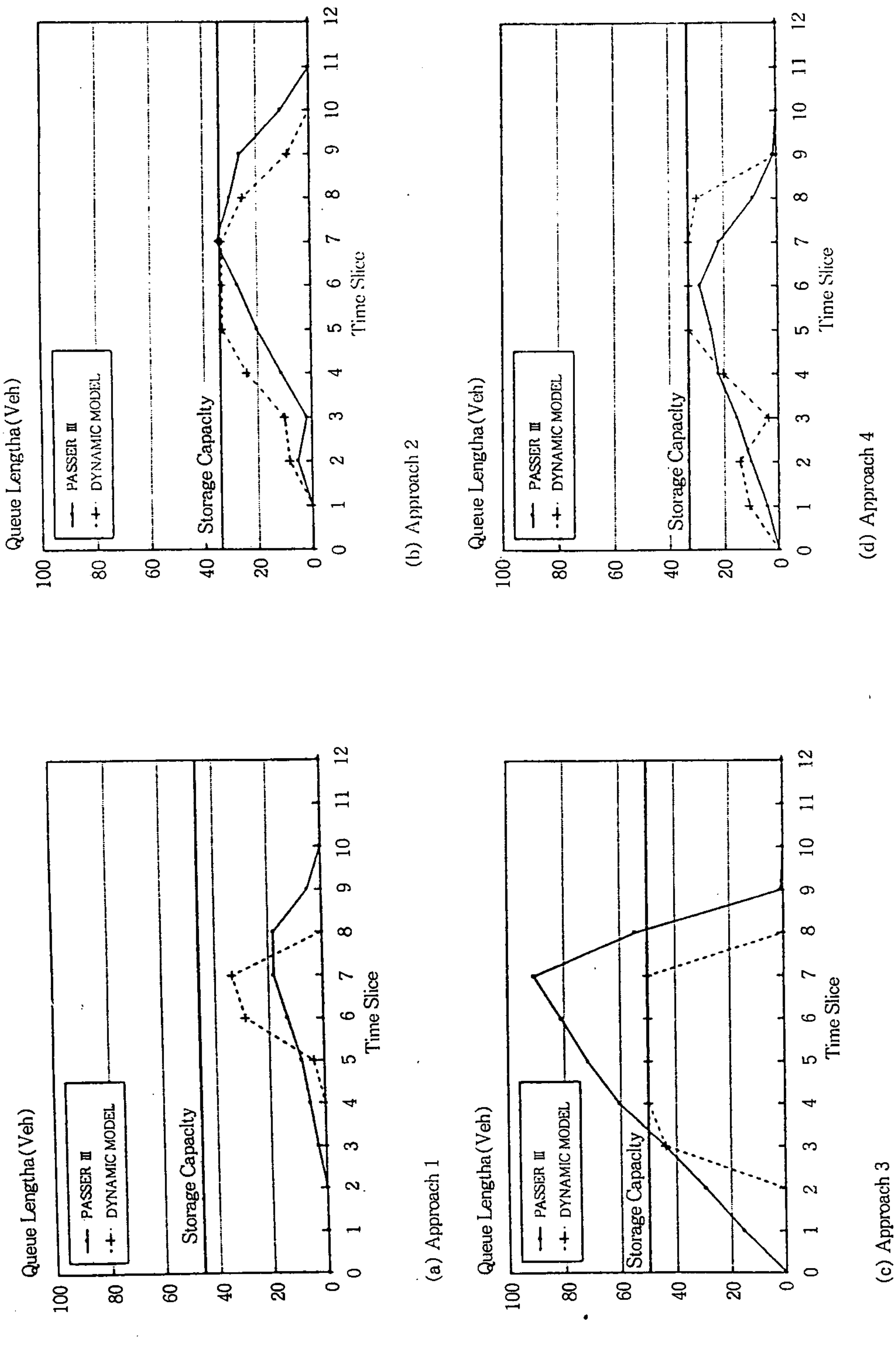


Figure 13. Queues by Input-Output Analysis at TUDI(Case 1)

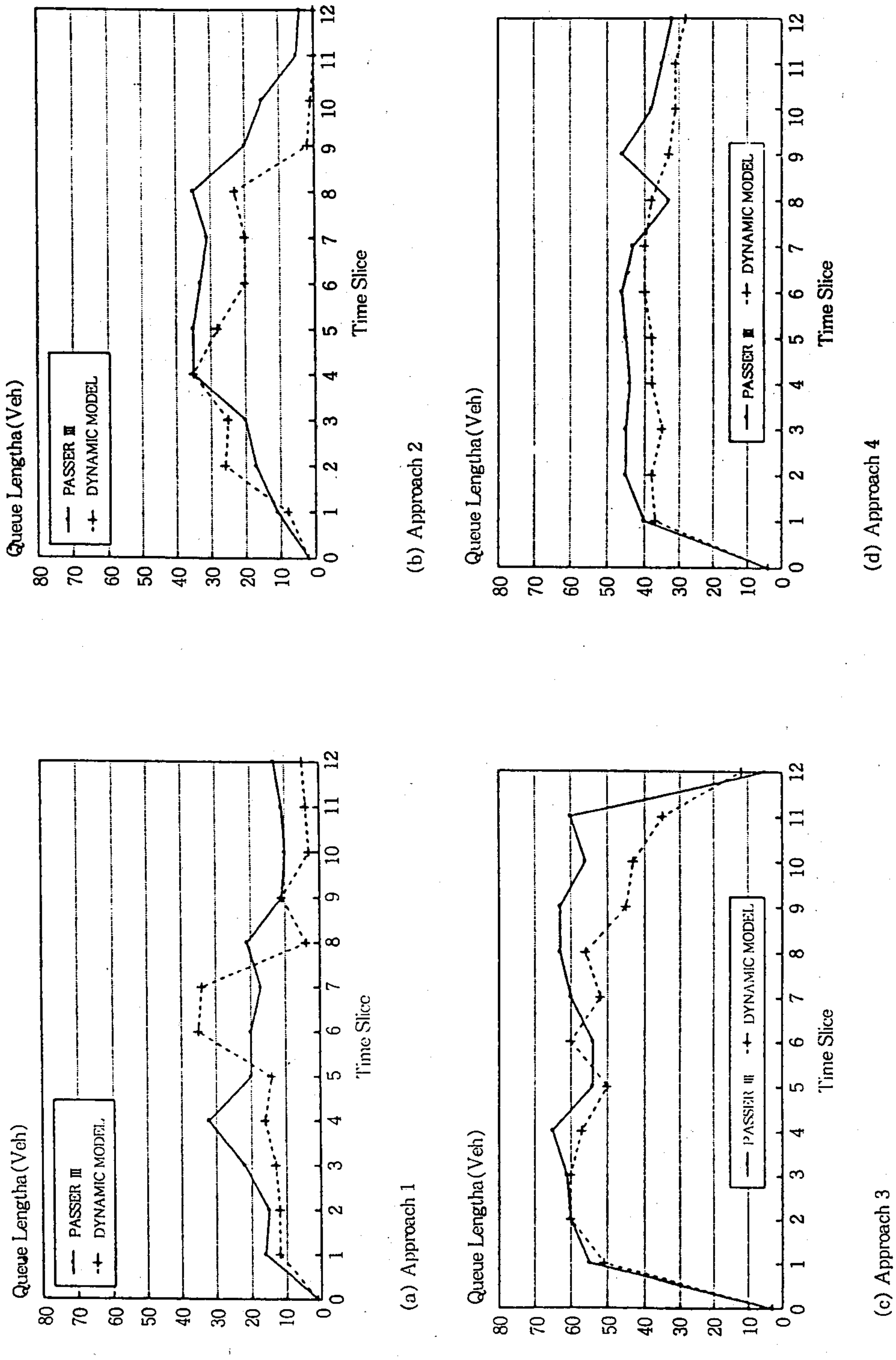


Figure 14. Queues by TRAF-NETSIM at TUDI(Case 1)

using the Input-Output model because PASSER Ⅲ cannot estimate timing based on varing queue lengths on the external approaches.

From Figure 13, the PASSER Ⅲ timing plan proudced very long queues on Approach 3, exceeding the queue storage capacity. The dynamic model reduced this queue to the storage capacity and thereby produced slightly longer queues on the other approaches than those produced using PASSER Ⅲ. Even if some portion of the queues on the critical approaches transfer to noncritical approaches, the overall queues were reduced by using the dynamic model due to its responsive green split capability.

Queues estimated by TRAF-NETSIM in Case 1 are shown in Figure 14. Queue profiles by TRAF-NETSIM have trends similar to that of the Input-Output model, but they are not the same. In Approach 3, PASSER Ⅲ queues do not exceed 60 vehicles while the PASSER Ⅲ queues reach 90 vehicles from the Input-Output model. The reason for this difference is that 1, 200 feet was coded as the external link length for TRAF-NETSIM. TRAF-NETSIM produced an error message of out -of memory for any link longer than this length. If too many vehicles to be exhausted, resulting in an out-of-memory error.

Figure 15 shows comparisons of queue estimations between the Input-Output model and TRAF-NETSIM. Regression analysis was conducted to estimate the best fit line. The slopes of the regression lines are less than 1. This result means that Iput-Output Analysis underestimates queue lengths compared to TRAF-NETSIM. The reason for this difference is that the Input-Output model estimates queues based on uniform traffic demand only, and does not consider the effect random demand has on queue length estimation. The β factor in Equation 11 should be adjusted based on this result. From the regression analysis, a value of .65 appears to be reasonable for the β factor.

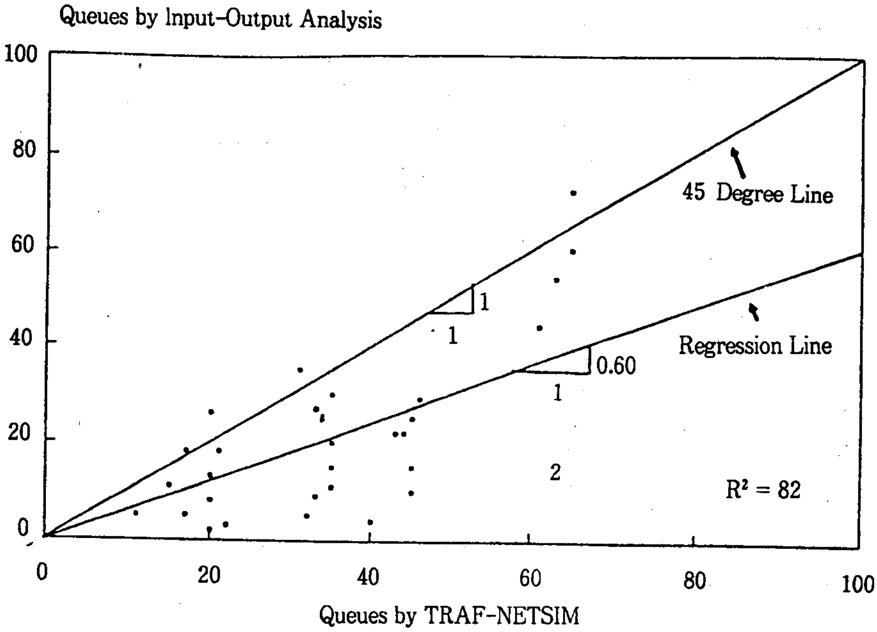
Figure 16 illustrates how signal timing responds to queue length constraints. Figure 16 (a) shows queue profiles on Approach 3 of Case 1. Two different queue constraints were applied: 93 and 50 vehicles. Figure 16(b) presents the green times of Approach 3 produced by the dynamic model. In the 50-vehicle queue constraint case, queue on Approach 3 reaches its upper bound at the end of Time Slice 4. To prevent queue spillback, the dynamic model assigns large green time to the 50-vehicle queue constraint case compared to the 93-vehicle case.

Ⅳ. CONCLUSIONS AND RECOMMENDATIONS

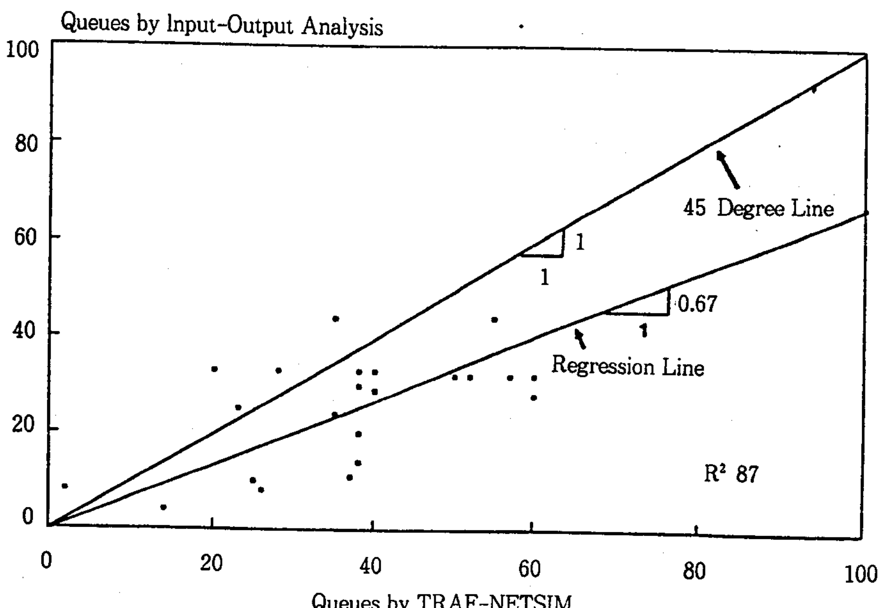
Dynamic optimization model was developed for signal control of congested diamond interchanges. The performance of the dynamic model was evaluated using the TRAF-NETSIM simulation program. Conclusions drawn from this research are described as follows.

1. The proposed dynamic model produces an optimal signal timing plan for traffic control of the signalized diamond interchange during oversaturated traffic conditions.

2. The dynamic model consistently outperforms conventional model, PASSER Ⅲ, with regard to system productivity. This conclusion was drawn from the TRAF-NETSIM simulation. Total trave and vehicle discharge in the

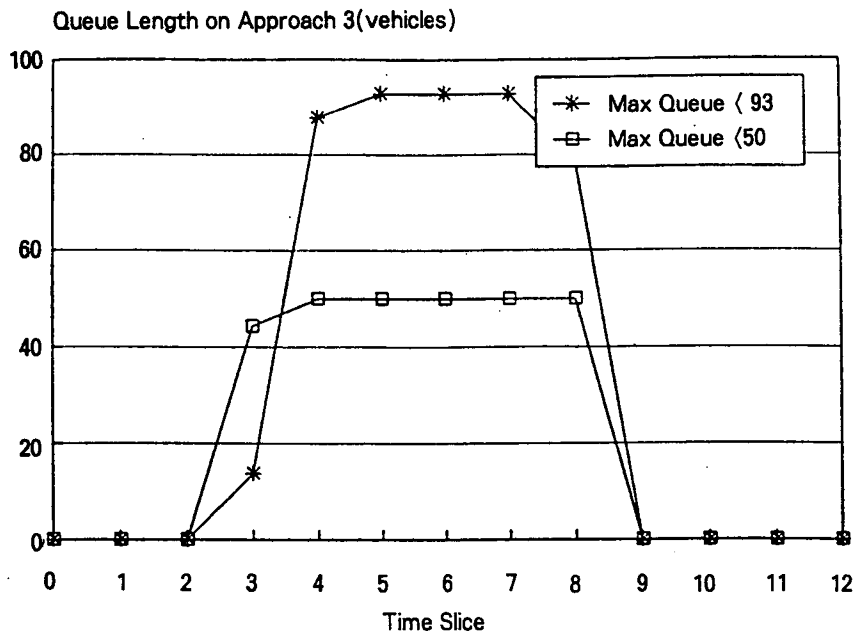


(a) PASSER Ⅲ

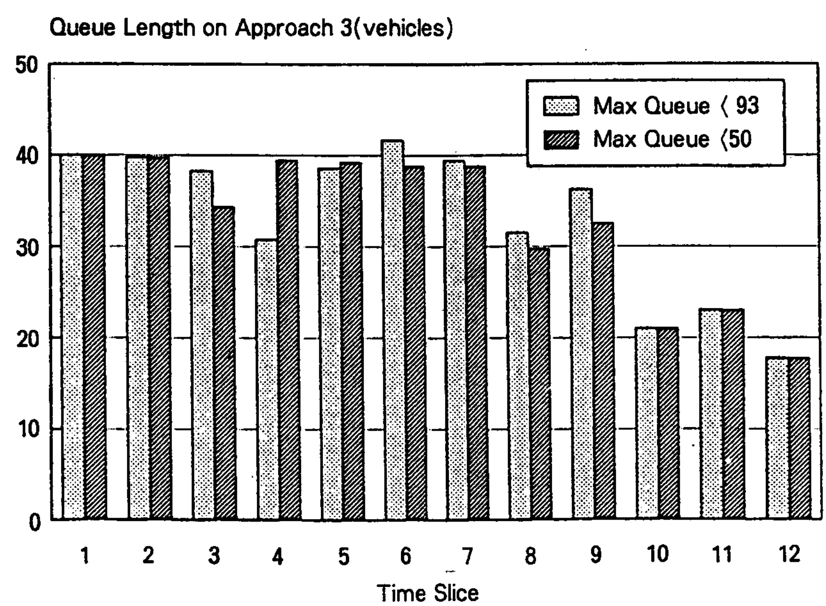


(b) Dynamic Model

Figure 15. Comparisons of Queue Estimation by Input-Output Analysis versus TRAF-NETSIM



(a) Queue Profile on Approach 3



(b) Green Time for Approach 3

Figure 16. Relationship between Queue Constraint and Green Time

TRAF-NETSIM output indicated increased productivity of the control systems.

3. The dynamic model solution tends to reduce total system delay while it increased delay slightley for a test case. The dynamic model generally increases the number of stops as compared to PASSER Ⅲ because it more fully utilizes the forward storage capacity of the signalized network.

4. Queue management on external approaches is a primary concern in the traffic contraol of congested conventional dismond interchanges. The capability of queue management is a unique feature of the dynamic model. This capability was demonstrated by the Input-Output analysis and the TRAF-NETSIM simulation. The dynamic model is superior to PASSER Ⅲ in queue management for congested interchanges.

5. The dynamic model controls lengths through efficient and timely changes of signal timing plans as demand changes. The frequent change of signal timing may cause unexpected operational problems, however. Traffic control strategies presented in this paper were designed to minimize the transitional delay. The control strategies appear to be effective in reducing this delay.

6. The dynamic model is regarded as a low-cost transportation improvement technique. This model is effective in relieving congestion within the traffic demand range that the queue storage capacity of the roadway system can accommodate. Beyond this acceptable range, however, the model is not very helpful, and major geometric improvements should be condidered to solve the remaining congestion problems.

7. TRAF-NETSIM was used as an evaluation tool to test the control strategies developed for oversaturated traffic conditions. Its ability to simulate queue spillback and intersection blockage was very important in evaluating the traffic control of the oversaturated conditions. It was found that TRAF-NETSIM was able to simulate these phenomena. The graphic presentation of the dynamic simulation process greatly aided the interpretation of operational results.

This research was an initial attempt to employ a dynamic optimization model for signal control of oversaturated diamond interchanges. Based on the simulation results, the dynamic model showed improved performance, and its applicability from a practical viewpoint was demonstrated successfully. Further studies are recommended to enhance the dynamic model, as follows:

1. Field validation of queue management control is recommended to confirm the benefits estimated by TRAF-NETSIM for the dynamic model.

2. The dynamic model has a weakness in signal timing during undersaturated time slices. Slack green times exist when all competing approaches are undersaturated. The dynamic model should be improved by introducing a routine to efficiently allocated these slack green times. This problem can be solved using a two-step optimization procedure like PASSER Ⅱ-87(10).

3. TRAF-NETSIM appeared to produce acceptable results in the simulation of congested traffic conditions. Studies are recommended to verify the reliability of its simulation results for congested traffic conditions through field tests.

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