비보호 좌회전 포화유률 추정

Estimation of Unprotected Left-Turn Saturation Flows

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ABSTRACT

When the capacity and traffic operation at signalized intersections are analyzed in Korea, the unprotected left-turn saturation flow rate, which is an important parameter for the analysis, is estimated from the USHCM model. Thus, exact analysis of the left-turn is not possible because of the difference of traffic environments between two contries. In order to improve this problem, it is undertaken in this study to develop techniques for the estimation of unprotected left-turn saturation flows based on Korean drivers' data.

As study intersections, signalized or unsignalized intersections on the 6, 4and 2 lane streets are selected. The data for the saturation flow measurement and gap-acceptance behavior analysis are inputed in a notebook computer on the sites.

The critical acceptance gaps of the 6, 4, and 2 lane steets are analyzed to be 6.0 secs, 4.6 secs, and 4.3 secs respectively. The average minimum headway of the left-turn vehicle was observed to be 2.6 secs. As the model to estimate unportected left-turn saturation flows, the Drew model is recommended for 6 and 4 lane streets, and a graph is suggested for the 2-lane street.

As the values of the parameters of the Drew model, the 2.6 secs of this study is recommended for the average minimum headway of the left-turn. But, the critical acceptance gap varies according to the approach speed of opposing traffic and driver population, it requires field survey to measure the gap of an intersection; however, the values of the gaps studied in this study may be used for the general intersections in urban area in Korea.

I. INTRODUCTION

In this study, the unropected left-turn is defined as the left-turn movement through a conflicting opposing vehicle flow on a solid green at signalized intersections. Thus, the unprotected left-turn saturation flow can be defined as the flow rate of left-turn vehicles that would be obtained if there were a continuous queue of left-turn vehicles given 100 percent green time. Pedestrian traffic on the cross street that might interfere with vehicles attempting to turn is assumed to be negligible. Since the flow at signalized intersections is controlled by the amount of green time allotted, the left-turn capacity at the intersections is then given by the actual possible number of left-turns in one hour, considering the effects of the signal.

For the unsignalized intersection, if vehicle and pedestrian traffic on the crossing minor street is negligible, the movement of left-turn from the major street is the same as that of unprotected left-turn at signalized intersections, and the left-turn capacity at the intersection is the saturation flow itself.

Presently, the capacity and traffic operation analyses at signalized intersections in Korea are performed according to the Highway Capacity Manual (HCM) of U.S.. Among the parameters of the HCM technique, the parameters of saturation flow rate, start-up lost time, and clearance lost time have been studied in many studies with Korean drivers' data. In these studies, it has been found that Korean drivers' behavior is different from American drivers', and the values of the parameters for Korean drivers are suggested. But, the unprotected left-turn saturation flow has not been studied in Korea yet. It can be inferred from the cases of the above parameters that the saturation flow rate of Korean drivers would be different from the rate of the HCM model. It is needed to test the HCM model with Korean drivers' data and develop a new model if needed in order to analyze the unprotected left-turn exactly.

The purpose of this study is to develop the model to estimate the unprotected left-turn saturation flow. For the purpose, actual unprotected left-turn saturation flows for various opposing flows were observed at the intersections having sufficient left-turn demand, and the drivers' behavior was analyzed to estimate the critical acceptance gap and the mean minimum headway of left-turn vehicles for the development of the model.

In this study, mathematical and computer simulation models were tested for the model and a notebook computer was used to collect field data. The types of event and the event time were inputed into the computer at the site.

The result of this study may be used for the exact analysis of the saturation flow and capacity at not only signalized but unsignalized intersections in Korea.

II. LITERATURE REVIEW

Considering the development of a practical model, four types of models were reviewed in this study. The definition of the notation used in the models is as follows.

S_{LT} = unprotected left-turn saturation flow (vph)
s_{LT} = 〃 〃 (veh/sec)
Q_o = opposing flow (vph)
q_o = 〃 (veh/sec)
τ = critical gap (sec)
α = mean minimum headway of opposing flow (sec)
β = mean minimum headway of left-turn flow (sec)

2.1 HCM Model

The Highway Capacity Manual of U.S. estimates the saturation flow using the below equation (TRB, 1985). The model, as a simple equation, is very convenient for use, but the model does not consider the number of opposing lanes and the size of the critical gap, which are considered to be important factors determining the saturation flow. Especially, it is known for the model to underestimate the saturation flow under the condition of high opposing flows.

$$S_{LT} = 1{,}400 - Q_o \quad \cdots\cdots\cdots\cdots (\text{eq. } 1)$$

2.2 SIDRA Model of Australia

Australian Road Research Board (ARRB) developed the SIDRA (Signalized Intersection Design and Research Aid) - 2 computer program for the design and research of traffic operation at signalized intersections (Akcelik, 1989). The program is based on the analytical analysis presented in Australian Road Research (ARR) No.123 (Akcelik, 1981) of ARRB, but more advanced techniques are used in the program.

The unprotected left-turn saturation flow during unsaturated green time of opposing flows is estimated using the following equation in SIDRA-2 program.

$$s_{LT} = \frac{\lambda \cdot \theta \cdot \exp\{-(\tau - \alpha)\lambda\}}{1 - \exp(-\beta \cdot \lambda)} \quad \cdots\cdots\cdots\cdots (\text{eq. } 2)$$

The parameters, λ and θ in eq.2 are calculated using the following equation.

$$\lambda = \sum_i \frac{\phi_i \cdot q_i}{(1 - \alpha \cdot q_i)}$$

$$\theta = \prod_i (1 - \alpha \cdot q_i)$$

where

i = number of opposing lanes

ϕ_i= bunching factor (proportion of unbunched vehicles in 'i'th opposing lane)

q_i= flow rate in 'i'th opposing lane

For the SIDRA model, it is difficult to estimate the prameters, λ and , but the model has merits to be sensitive to the number of opposing lanes and to consider the lane utilization rate. The comparison of the HCM and SIDRA model is shown in Figure 1, where the 'N' represents the number of lanes.

2.3 Michalopoulos's Model

Michalopoulos et al. observed the saturation flows at the signalized and unsignalized intersections with left-turn lanes. The intersections had the traffic charateristics of isolated intersections, standard geomertic structure, and high left-turn demand. The range of approach speed at the intersections was 48 - 56 kph.

In the study, three types of mathematical models which were predominant as the model at that time were tested for the model, and it was found that non of the models represented the real situation universally. Thus, the statistical model presented in eq. 3 was developed in the study (Michalopoulos et al., 1978). But, the model requires calibration with enough data for application in the area with different traffic environments.

$$S_{LT} = -0.233 \cdot \tau \cdot Q_o + 0.000015 \cdot \tau^2 \cdot Q_o^2 + 126 \cdot L + 103 \cdot S + 955 \quad \cdots\cdots\cdots\cdots (\text{eq. } 3)$$

where

L = 0: when opposing lane is 1

1: when opposing lane is 2

S = 0: unsignalized intersection

1: signalized intersection

2.4 Fambro's Model

Fambro et al. suggested a simplified equation (eq. 4) as the model by estimating the two parameters in the Drew model with field data (Fambro et al., 1977). Fambro's model is convenient for use and more sophisticated than the HCM model; but the model has the same problems as the HCM model.

$$S_{LT} = \frac{Q_o \cdot \exp(-4.5 \cdot q_o)}{1 - \exp(-2.5 \cdot q_o)} \quad \cdots\cdots\cdots\cdots\cdots \text{(eq. 4)}$$

III. DATA COLLECTION AND ANALYSIS

Presently, the unprotected left-turn is only adopted in Seoul city in Korea; about 20 signalized intersections where left-turn was permitted on solid green signal in Seoul were investigated, but only one intersection had standard geometric structure and sufficient left-turn demand to satisfy the conditions for the study intersection. The intesection is located on the six-lane street, Taehakro and near a entrance of Haehwa subway station. Therefore, the unsignalized intersections at which the left-turn behavior from major streets is very similar to the behavior of unprotected left-turn at signalized intersections (because of very low vehicle and pedestrian traffic at cross streets) were investigated in the three cities: Masan, Changwon, and Chinju. An intersection in Masan was selected as the study intersection on four-lane streets. Any intersection on two-lane streets in the cities did not satisfy the conditions as a study intersection. However, an intersection on a two-lane street in Chinju was selected to analyze gap-acceptance behavior only since the intersection had standard geometric structure but low left-turn demand. The range of approach speed at the study intersections was 40 - 50 kph and most of the vehicles were passenger cars.

Using VTR systems is a common technique to collect drivers' gap-acceptance data at intersections; but the technique requires double man power for recording the drivers' behavior in fields and reproducing that for collection of the data in laboratories, and the vision is restricted when the behavior is reproduced on monitors. Thus, in this study the data was collected at the site using a notebook computer and "C" language computer program. The tehnique employing the notebook computer makes it possible to collect accurate data with clear and wide vision, and save man power; but using the technique is more troublesome in bad weather conditions. At each study intersection, the times of opposing vehicles' arrival and left-turn vehicles' arrival/departure were inputed into the computer by distinguishing the periods during which there was a queue of left-turn vehicles or not.

For the measurement of the saturation flow, only the left-turn flow rate when there is a queue of left-turn vehicles is needed to be analyzed. But the critical acceptance gap and mean minimum headway were analyzed for the development of the model to eatimate the saturation flow in this study.

3.1 Critical Acceptance Gap and Mean Minimum Left-Turn Headway

The proportions of gaps accepted at the study intersections are listed in Table 1 for each range of gaps. The gaps of the range of 7 - 8 secs are accepted by 100% on the two-lane street, but the gaps on the six-lane street are accepted by 100% when those are greater than 12 secs.

The previous studies of Ashworth et al. (1977), Solberg et al. (1966), and author (1986) suggested that the normal distribution could be used to represent the critical acceptance gap distribution. Thus, in this study the mean and standard deviation of the critical gaps are analyzed using the Probit Analysis (Finney, 1971), and the results are presented in

Table 2. The corrected mean in the Table 2 is the mean corrected by using the Ashworth technique (Ashworth, 1968), which is presented in eq. 5. The Ashworth technique corrects the error associated with considering gaps and lags together. When gaps and lags are considered together for the analysis of the critical acceptance gap, the more data can be utilized, but the error arises because a driver having a large critical acceptance gap will reject several gaps before accepting one.

$$\mu_d = \mu_p - q_o \cdot \sigma_p^2 \qquad \text{(eq. 5)}$$

$$\sigma_d = \sigma_p$$

where

μ_d = mean of the probablity density function (sec)

μ_p = mean of cumulative probability function (sec)

σ_d = standard deviation of probability density function (sec)

σ_p = standard deviation of cumulitive probability function (sec)

<Table 1> Proportions of Gaps Accepted for Gap Size and Number of Opposing Lanes

Gap Size (sec)	Six-Lane Streets			Four-lane Streets			Two-Lane Streets		
	Total No.	Accepted No.	Per-cent	Total No.	Accepted No.	Per-cent	Total No.	Accepted No.	Per-cent
0 - 1	480	0	0	218	0	0	35	0	0
1 - 2	388	0	0	150	0	0	94	0	0
2 - 3	188	0	0	129	7	5.1	78	8	10.3
3 - 4	95	5	5.2	100	12	12.0	43	7	16.3
4 - 5	53	9	17.2	48	20	41.7	19	6	31.6
5 - 6	28	7	25.0	34	17	50.0	16	7	43.8
6 - 7	25	9	36.0	24	19	79.2	8	7	87.5
7 - 8	15	6	40.0	13	13	100.0	7	7	100.0
8 - 9	11	6	54.5	15	14	93.3	2	2	100.0
9 - 10	13	10	76.9	8	8	100.0	0	0	100.0
10 - 12	14	13	92.9	18	18	100.0	9	9	100.0
12 - ∞	84	84	100.0	23	23	100.0	25	25	100.0
계	1,394			780			336		

<Table 2> Probit Analysis Results of the Critical Acceptance Gap

Unit: sec

Parameters	Six-Lane Streets	Four-Lane Street	Two-Lane Streets
Mean	7.1	5.0	4.6
Corrected Mean	6.0	4.6	4.3
Standard Deviation	1.92	1.38	1.08
R^2	0.85	0.90	0.81

Analyzing the critical gaps based on the corrected ones, there ia a large difference in the size between the six-lane and the four-lane street, but small difference between the four-lane and the two-lane street. In the study conducted by author in 1986, the critical gaps on four-lane and two-lane streets were analyzed to be 3.4 and 3.0 secs, respectively. Comparing the critical gaps of this study with those of the 1986's study, the critical gaps of this study are much larger than those of the 1986's study. The reason would be that most of the sample drivers in the 1986's study were professonal drivers as taxi drivers but most of them in this study were owner drivers, although different characteristics of the study intersections affected the results.

The mean minimum left-turn headway, which is one of important parameters for the construction of the model to estimate the saturation flow, is estimated to be 2.6 secs. This value is much larger than the 1.6 secs of the protected left-turn flow (MOC, 1992). It is because the driver turning left under unprotected conditions consumes more time than the driver under protected conditions in selecting and passing through gaps in a conflicting vehicle flow.

3.2 The Saturation Flow

In this study, the mean saturation flow for a range of 200 vph of opposing flow rates was calculated by counting the number of left-turn and through vehicles during the period in which there was a queue of left-turn vehicles. When the interval, from the time that the last vehicle in a left-turn queue turned left to the time that the next through vehicle passed the intersections, was larger than the critical acceptance gap, the saturation flow was calculated substituting the interval with the critical gap. The mean unprotected left-turn saturation flows for the ranges of opposing flow rates are listed in Table 3, where the value in the column of opposing flow rates is the median rate of each range.

<Table 3> Uprotected Saturation Flows for Number of Opposing Lanes

Opposing Flows (vph)	Six-Lane Streets(A)		Four-Lane Streets(B)		B - A (vph)
	No. of Data	Saturation Flows(vph)	No. of Data	Saturation Flows(vph)	
1,700	6	206	4	260	54
1,500	7	230	4	286	56
1,300	6	239	4	354	115
1,100	7	308	14	404	96
900	2	349	7	478	129
700	4	381	15	598	217
500	2	530	7	947	417
300	7	589	-	-	-

As shown in the Table 3, the smaller the opposing flow rate is, the larger the difference is in the saturation flow between the four-lane and the six-lane street.

IV. DEVELOPMENT OF MODELS

Two types of models, mathematical and graphical moelds, were considered for the model to estimate the saturation flow for a specific opposing flow rate. The graphical model was conceived as a form of chart which had different curves for different critical acceptance gaps.

For the mathematical model, the Drew model, which was origionally developed based on the analysis of gap-acceptance behavior for drivers merging at freeway ramps (Drew, 1968), was selected. Drew model was found to be the best model on four-lane streets in the Michalopoulos's study (1978) in which two-lane and four lane streets were studied, and the model was used in Swedish HCM (Bang, 1978) and Fambro's model (Fambro et al.,1977). The model is practical for use as a concise equation, and the theoretical background of the model is not weak as including important parameters in the model. The Drew model which was selected as the mathematical model in this study is:

$$S_{LT} = Q_o \cdot \frac{\exp(-q_o \cdot \tau)}{1 - \exp(-q_o \cdot \beta)}$$

where

Q_o = opposing traffic flow (vph)

q_o = 〃 〃 (veh./sec)

τ = critical acceptance gap (sec)
β = mean minimum headway of left-turn flow (sec)

For the graphical model, a computer simulation model was developed to provide the data for drawing the curves by using the SLAM computer simulation language (Pritsker, 1986). Based on the results of the 1986's study of author, it was assumed that a fixed critical acceptance gap was assigned to each left-turn driver and the critical acceptance gap of each driver varies from one driver to another, in accordance with a postulated normal distribution. And the headway distribution of four and six lane streets was assumed to follow the negative exponential distribution but that of two-lane streets to follow the shifted negative enponential distribution considering the minimum headway for safety. The minimum headway was assumed to be 1.6 secs based on the results of a study (MOC, 1992).

The comparisons of the observed and the estimated by the Drew, HCM, and simulation model for four and six lane streets are shown in Figure 2 and Figure 3. In those models, the corrected mean in the Table 2 was used for the critical acceptance gap; and 2.6 secs, which was estimated in this study, was used for the headway of left-turn vehicles. The estimated saturation flows by those three models for given opposing flow rates and the observed mean saturation flows for the ranges of opposing flow rates are shown in Figure 4, Figure 5, and Table 4 for four and six lane streets.

In the Figure 4, the value by the HCM model is close to the observed at the opposing flow rate of about 1,100 vph, but the model overestimates below the oppposing rates and underestimates above the opposing rates. The two curves of the simulation and Drew model are similar in shape, and both curves are close to the observed in wide range as compared with the HCM model.

The comparisons of the observed and estimated for the four-lane street are shown in the Figure 5. The eatimated by the three models are close to the observed up to the opposing flow rate of 650 vph, but the HCM model much underestimate the saturation flow above the opposing flow rate. The Drew model and the simulation model have almost same value, however the simulated are more close to the observed.

<Table 4> Comparison of Observed, Simulated, and Calculated Values of Unprotected Left-Turn Saturation Flows

Opposing Flows	Six-Lane Streets			Four-Lane Streets		
(vph)	Observed	Simulated	calculated	Observed	Simulated	Calculated
1,700	206	89	141	260	222	274
1,500	230	124	186	286	278	334
1,300	239	174	245	354	348	405
1,100	308	243	321	404	435	492
900	349	340	420	478	544	596
700	381	475	549	598	681	721
500	530	605	717	947	852	871
300	589	931	934	-	-	-
Statistics[1)]						
Standard Error of Estimate	-	148.9	155.6	-	57.0	82.5
R^2	-	0.974	0.980	-	0.933	0.920

1) Statistics of Simulated and Calculated values to Observed Values

The unprotected left-turn saturation flows by the Drew model for the critical acceptance gaps of the four and six lane street are shown in Figure 6. There is a difference as much as 170 vehicles between the two critical gaps in the Drew model, although no difference in the HCM model.

The statistics of comparing the calculated (by the Drew model) and the simulated with the observed are presented in Table 4, also. The coefficient of determinant, R^2 is not less than 0.92 for all cases, but the 'standard errors of estimate' of the simulated are a little smaller than those of the calculated by the Drew model.

Since the data of the saturation flow on two-lane streets could not be collected, the comparison of the simulated, the calculated by the Drew, and HCM model is shown in Figure 7. There is a great difference between the simulated and the calculated. It can be infered that the simulated represent the actual situation better than the calculated based on the facts that the simulated are superior to the calculated statistically for the four and six lane streets in this study and the results of other studies (Akcelik, 1989; Michalopoulos, 1978) like the Figure 1.

For the simulation model to be used for the estimation of the unprotected left-turn saturation flow, a chart which indicate the saturation flow for given values of the critical acceptance gap and opposing flow rate should be drawn, and it is troublesome to find the flow on the chart. If the mathematical model is a concise equation like the Drew Model, the model is more convenient than the chart for use. Considering this aspect and the facts that the simulated and the calculated are very close in the figures and statistical tests, the Drew model is sugessted for the estimation of the saturation flow for four and six lane streets. But, for two-lane streets, since the Drew model seems to overestimate greatly, the chart of Figure 8 which is drawn by the simulated saturation flows in accordance with the critical acceptance gaps and opposing flows rates is suggested. In the Figure 8, when the critical acceptance gap is 4.0, the curve is very close to the HCM model.

As the parameter values of the Drew model, the value of 2.6 secs is recommended for the mean minimum left-turn headway. In order to determine the critical acceptance gap for a specific intersection, field studies are required because the gap has different value according to the approach speed and driver population at the intersection, however the critical acceptance gaps analyzed in this study may be used for general streets in urban areas in Korea.

V. CONCLUSIONS

In this study, the unprotected left-turn saturation flow, the critical acceptance gap, and the mean minimum headway of the unprotected left-turn flow at the intersections in urban areas in Korea were analyzed. Based on the results of this study, the models for the estimation of the unprotected left-turn saturation flow were developed.

Study intersections were selected from the three types of streets, six, four, and two lane streets, and a notebook computer was employed to collect field data by inputing the event time into the computer at the site. The unprotected left-turn saturation and opposing flows were observed at the intersections on four and six lane streets, and the mean saturation flows for the ranges of 200 vph of opposing flows presented in the Table 3. The mean critical acceptance gap for six, four, and two lane strets were analyzed to be 6.0, 4.6, and 4.3 secs respectively. There is a great difference in the gap size between six and four lane streets, but a small difference between four and two lane streets. The mean minimum headway of the unprotected left-turn flow was observed to be 2.6 secs.

For the models to estimate the saturation flow, the Drew model of eq. 6 is recommended for the streets of more than four lanes, and the Figure 8 recommended for two-lane streets. As the parameter values of the Drew model, the value of 2.6 secs is recommended for the mean minimum headway of unprotected left-turn flows, and the critical acceptance gap analyzed in this study may be used for general streets in urban areas in Korea.

In future studies, it is recommended to build a model for the estimation of the saturation flow on two-lane streets based on field data, and to analyze the effect of traffic signals on the saturation flow.

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