REVISED PLANNING METHODOLOGY FOR SIGNALIZED INTERSECTIONS AND OPERATIONAL ANALYSIS OF EXCLUSIVE LEFT-TURN LANES

– A Simulation-based Method –

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16. Abstract

The study investigates the application of simulation along with field observations for estimation of exclusive left-turn saturation flow rate and capacity. The entire research has covered the following principal subjects: (1) A saturation flow model for protected phase that allows engineers to incorporate local-specific driving behavior in the analysis; (2) A permitted saturation flow model that accounts for the impact of driving behavior and traffic conditions at both the target and the opposing upstream intersections on the permitted saturation flow rate; (3) A queue length model for estimation of the resulting queue length under various phase schemes; and (4) A hybrid bay length impact model that provides engineers a convenient tool to assess the impact of various bay lengths on the left-turn capacity at both isolated and coordinated intersections. This report also describes the procedures for direct estimation of intersection capacity with simulation, including a set of rigorous statistical tests for simulation parameter calibration from field data.

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Chapter 1. INTRODUCTION

1.1 Background

The presence of left-turning vehicles at signalized intersections tends to cause excessive delay, increase the accident potential, and lower the intersection capacity. Hence, accommodating left-turning vehicles with effective signal control strategies has long been a source of concern for traffic engineers. In practice, depending on the use of shared or exclusive lanes for left-turning vehicles, traffic engineers must make a selection of left-turn phasing which best satisfies the left-turn demand and minimize the operational difficulties incurred by left turns. An appropriate tool or procedures to evaluate the proposed design strategies (i.e., permitted, protected, protected/permitted) thus become quite essential.

Over the past several decades, although highway agencies and research institutions have developed various guidelines for left-turn capacity analysis, the most widely used are the procedures included in Chapter 9 of ***the 1985 Highway Capacity Manual*** (HCM). In fact, the 1985 HCM has been used by more traffic and transportation engineers in the past seven years since it was published than the 1965 HCM in 20 years.

However, due to the lack of sufficient empirical validation in their developments, many procedures or models recommended by the 1985 HCM are subjected to revision. This is particularly true of Chapter 9 “Signalized Intersections”. In many situations, the output from an analysis of capacity either does not agree with field observations or may yield vast different results with slight variations in input data. For instance, the procedure for the division of left turn volume between the protected and permitted phase is not satisfactory. The resulting level of service under a given demand varies substantially with a user’s allocation of traffic volume to the protected and permitted phasing period. In view of various technical deficiencies identified with given applications for using the HCM signalized intersection methodology, attempts are being made to modify the current procedures or develop new procedures. This is one of the several research projects sponsored by FHWA for revising the current HCM procedures.

1.2 Scope of Work

The objectives of this research project are:

1. Develop specific recommendations on text, tables, and illustrative materials adequate to revise the methodology for analyzing exclusive left-turn lanes on Chapter 9 of the HCM.

2. Develop a more appropriate traffic model for the operational analysis of exclusive left-turn lanes.

More specifically, the methodology proposed in this project shall contain, as a minimum, saturation flow adjustment factors which can realistically replicate traffic flow for the following phasing schemes: protected, permitted, protected/permitted, and permitted/protected. The methodology shall address the effect of both left-turn bay length and the number of lanes on capacity. Specific recommendations shall be provided on text, tables, and illustrative materials adequate to revise the methodology for the operational analysis of exclusive left-turn lanes in Chapter 9. The methodology to be developed shall not require the user to calculate the demand for protected and permitted phases.

1.3 Research Focus of this Report

According to the contract, this task shall be focused on identifying literature sources and review relevant reports on methodologies for analyzing left turns from exclusive lanes. Attention shall be given to reports addressing technical weaknesses of the HCM methodology. The literature review shall include, but not be limited to, Transportation Research Information Service (TRIS), published bibliographies, state-of-the-art reports, and transportation periodicals, both domestic and international.

1.4 The Report Organization

This report, mainly for literature review, is organized as follows: Next chapter first illustrates the current HCM procedures for exclusive left-turn capacity analysis, and then indicates their deficiencies from a traffic practicing engineer's perspective. Also included in this chapter are a list of potential directions for improvement. Chapter 3 concentrates **on** the investigation of critical issues as well as literature related to the saturation flow estimation. Chapter 4 summrizes all left-turn capacity estimation methods under protected, permitted, protected/permitted, and permitted/protected phasings, including some vital issues to be addressed in the future research. Chapter 5 presents a preliminary framework of the research methodology for the Next Task - Development of Traffic Models. Some recommendations regarding the research directions also constitute the core of Chapter 5.

CHAPTER 2. REVIEW OF CURRENT HCM PROCEDURES AND DEFICIENCIES

This chapter presents a brief overview of the ***Highway Capacity Manual's*** operational analysis procedures for left turn lanes at signalized intersections. Issues and deficiencies associated with the current procedure are also identified and described. The chapter concludes with a discussion of some of the difficulties of using HCS, the Highway Capacity Software, from the perspective of a practicing traffic engineer.

2.1 Current HCM Procedures for the Operational Analysis of Left Turn Lanes

The 1985 ***Highway Capacity Manual,*** which was published as Transportation Research Board Special Report 209, presents procedures to perform operational analyses of signalized intersections with exclusive left turn lanes. Application of the signalized intersection procedure will result in the calculation of an average delay time and a corresponding level of service for each lane group, for each approach and for the overall intersection. Level of service at signalized intersections is defined in terms of average individual delay as shown in Table 2-l.

Table 2-1. Average delays corresponding to levels of service.

Level of Service	Average Delay (seconds/vehicle)
A	< 5.0
B	5.1 - 15.0
C	15.1 - 25.0
D	25.1 - 40.0
E	40.1 - 60.0
F	> 60.0

With respect to exclusive left turn lanes at signalized intersections, the procedure for each exclusive left turn lane group consists of the following steps:

1. The adjusted lane group flow rate is computed as:

$$v=(V / PHF)*U$$

where:

v	=	adjusted demand flow rate for the left turn lane group, in vehicles per hour.
V	=	hourly volume, in vehicles per hour.
PHF	=	peak hour factor.
U	=	lane utilization factor.

The lane utilization factor for 1 lane is 1.00. For 2 lanes, it is 1.05. The implicit assumption is that the more heavily used lane carries 52.5 percent of the total flow.

2. Compute the saturation flow rate for the left turn lane group as follows:

$$s = s_0 \; N \; f_w \; f_{HV} \; f_g \; f_p \; f_{bb} \; f_a \; f_{LT}$$

where:

s	=	saturation flow rate for the subject lane group, expressed as a total for all lanes in the lane group under prevailing conditions, in vphg.
s_0	=	ideal saturation flow rate per lane, usually 1,800 pcphgpl.
N	=	number of lanes in the lane group.
f_w	=	adjustment factor for lane width; 12 ft lanes are standard.
f_{HV}	=	adjustment factor for heavy vehicles in the traffic stream.
f_g	=	adjustment factor for approach grade.
f_p	=	adjustment factor for the existence of a parking lane adjacent to the lane group and the parking activity in that lane.
fbb	=	adjustment factor for the blocking effect of local buses stopping within the intersection area.
f_a	=	adjustment factor for area type.

f_{LT} = adjustment factor for left turns in the lane group.

The left-turn adjustment factor (f_{LT}) accounts for the fact that left turns cannot be made at the same saturation flow rates as through movements. The left-turn adjustment factors for left turns from exclusive left turn lanes are summarized in Table 2-3.

Table 2-3. Adjustment factors for left turns from exclusive left turn lanes.

Number of Left Turn Lanes	Type of Phasing ,	Left-turn Factor (f_{LT})
1	Protected	0.95
1	Permitted	Special Procedure
1	Protected plus permitted	Factor is derived though an iterative process; 0.95 is the starting value.
2	Protected	0.92

When a lane group includes permitted left turns, the left-turn adjustment factor must be computed using a complex series of equations. The equations approximate the effect of equilibrium flows which result from the interaction of left-turning vehicles, through vehicles, and opposing flows. A worksheet was developed to simplify the computations and was presented in the ***Highway Capacity Manual.*** The worksheet is shown in Figure 2-1.

3. Compute the flow ratio, capacity and v/c ratio for the left turn lane group as follows:

$$\text{Flow Ratio} = V_i / S_i$$

$$c_i = s_i \times (g/c)_i$$

$$X_i = v_i / c_i$$

where:

c_i = capacity

v_i = adjusted demand flow

s_i = adjusted saturation flow

$(g/c)_i$ = effective green-to-cycle length ratio

SUPPLEMENTAL WORKSHEET FOR LEFT-TURN ADJUSTMENT FACTOR, f_{LT}				
INPUT VARIABLES	EB	WB	NB	SB
Cycle Length, C (sec) *(Estimated)*	90	90		
Effective Green, g (sec) *(Estimated)*	18.5	18.5		
Number of Lanes, N	1	1		
Total Approach Flow Rate v_a (vph)	495	742		
Mainline Flow Rate, v_M (vph)	424	624		
Left-Turn Flow Rate, v_{LT} (vph)	71	118		
Proportion of LT, P_{LT}	1 0	1.0		
Opposing Lanes, N_o	2	2		
Opposing Flow Rate, v_o (vph)	624	424		
Prop. of LT in Opp. Vol., P_{LTO}	0.0	0.0		
COMPUTATIONS	EB	WB	NB	SB
$S_{op} = \dfrac{1800\,N_o}{1 + P_{LTO}\left[\dfrac{400 + v_M}{1400 - v_M}\right]}$	3600	3600		
$Y_o = v_o / S_{op}$	0.173	0.118		
$g_u = (g - CY_o) / (1 - Y_o)$	3.54	8.93		
$f_s = (875 - 0.625\,v_o) / 1000$	-	-		
$P_L = P_{LT}\left[1 + \dfrac{(N-1)\,g}{f_s g_u + 4.5}\right]$	1.0	1.0		
$g_q = g - g_u$	-	-		
$P_T = 1 - P_L$	0.0	0.0		
$g_f = 2\,\dfrac{P_T}{P_L}\left[1 - P_T^{\,0.5\,g_q}\right]$	0.0	0.0		
$E_L = 1800 / (1400 - v_o)$	2.32	1.84		
$f_m = \dfrac{g_f}{g} + \dfrac{g_u}{g}\left[\dfrac{1}{1 + P_L(E_L - 1)}\right] + \dfrac{2}{g}(1 + P_L)$	0.31	0.48		
$f_{LT} = (f_m + N - 1) / N$	0.31	0.48		

Source: *Highway Capacity Manual*, 1985.

Figure 2.1 Worksheet that can be used to compute the left-turn adjustment factor for exclusive left turn lanes with permitted phasing.

4. Compute the delay for each lane group using the following equation:

$$d = 0.38\ C\ \frac{[1 - g/C]^2}{[1 - (g/C)\ (X)]} + 173\ X^2\ [(X - 1) + ((X - 1)^2 + (16X/c))^{1/2}]$$

where:

d	=	average stopped delay per vehicle for the lane group, in sec/veh;
C	=	cycle length, in sec;
g/C	=	green ratio for the lane group; the ratio of effective green time to cycle length;
x	=	v/c ratio for the lane group; and
c	=	capacity of the lane group.

Unlike other lane groups, it should be noted that no progression adjustment is made to the delay estimate for the left turn lane group with protected phasing. Hence, the delay estimate computed from step 4 is compared to the look-up table (see Table 2-l) to derive a corresponding level of service for the left turn lane group. A footnote to the progression adjustment factor table in the Highway Capacity Manual indicates the following:

> When LT's are included in a lane group encompassing an entire approach, use the factor for the overall lane group type. Where heavy LT's are intentionally coordinated, apply factors for the appropriate through movement.

There are a variety of commercially available software programs that replicate the manual procedures for signalized intersections. One of the more widely used packages is HCS, the Highway Capacity Software package, which was developed under the sponsorship of the Federal Highway Administration (FHWA). Data must be specified as input for the following parameters:

- Turning movement volumes (vehicle&r) for each approach.
- Signal phasing and timing (e.g., amount of green time and amber+all red time for each phase).

- Number of lanes and lane use (e.g. permissible turn movements that can be made from that lane) for each approach. [The default lane width of 12 ft can be overridden.]

Other input parameters have default values that can be overridden. These include those shown in Table 2-2.

Table 2-2. Default values for selected input variables to HCS.

PARAMETER	DEFAULT	OTHER ACCEPTABLE VALUES
Area Type	CBD	"Other"
Right Turns on Red for each approach	0	* Up to 100% R.T.
Type of signal operation	Fixed Time	Semi-actuated or Fully Actuated
Lost time per phase change	3.0 sec	*
% Grade for each approach	0%	*
% Heavy Vehicles for each approach	2%	*
Presence of adjacent parking	Yes	No
Number of parking maneuvers for each approach	20/hr	*
Number of local buses stopping at this intersection	0/hr	*
Peak Hour Factor for each approach	0.9	*
Number of Conflicting Pedestrians for each approach	50/hr	*
Arrival Type	3 ("Random")	1 (most on red) to 5 (most on green)

* - Variable is continuous. No constraints on the value that can be specified as input.

2.1 Difficulties and Issues Related to the Application of Current HCM Operational Analysis Procedures for Exclusive Left-Turn Capacity

Despite the wealth of available literature on the topic, there still remains unanswered questions about the validity of the current HCM model for exclusive left turn lanes. For example, from a theoretical basis, does the HCM operational analysis procedure for exclusive left turn lanes yield reasonable estimates of delay time? For protected only phasing? For permitted phasing? For protected-permitted phasing?

There have been several contentions that the HCM operational procedures for exclusive left turn lanes are deficient. ***In Transportation Research Circular*** No. 371 (June 1991), the lack of an adequate methodology for analyzing left turns from exclusive lanes was identified as one of the apparent weaknesses in chapter 9. The following were identified as issues to be addressed:

- Saturation flow adjustment factors for protected, permitted, protected/permitted, and permitted/protected phasing.
- The relationship between phase sequence and adjustment for progression.
- The splitting of demand between protected and permitted phases.
- Saturation flow rates to be used in the delay equations.
- Validation of FHWA study results for shared lanes (i.e., the methods should converge when a shared lane operates as an exclusive left turn lane).
- Validation of dual left turn lane factors.
- Effect of turn bay length on capacity utilization.

There has been a substantial amount of criticism aimed at the procedure of splitting the left turn traffic demand for exclusive left turn lanes that are served by protected and permitted left turn signal phasing. The HCM procedure requires that the left turn volume be split between the protected only and the permitted phase intervals. In most cases, traffic turn turning movement count data are not collected to differentiate the portion of left turns made during the protected phase versus the portion made during the permitted phase (or vice versa). Hence, the data is not generally available. However, the HCM procedure requires the analyst to make some type of determination. Moreover, <u>the selection of a particular option can yield widely disparate results and possibly change the corresponding level of service.</u>

The current HCM procedures may not accurately reflect the operation of fully actuated controllers at isolated intersections or actuated controllers operating within coordinated signal systems. If the signal is actuated, an adjustment factor is applied to the delay equation for through and right lanes but not exclusive left-turn lanes. Moreover, although the operating efficiency of actuated control depends on detector placement and type, these factors are not considered by the HCM procedure. When a coordinated timing plan is superimposed on an actuated controller, some features of actuated control (e.g., phases skipped or gap out in the absence of vehicle demand) are retained but a pre-specified minimum amount of green time is guaranteed within a specific cycle length for the coordinated phase(s). Since the HCM procedures were calibrated primarily from fixed-time controlled intersections, the validity of the procedures for actuated controllers is questionable. One study concluded that the HCM method does not adequately address the impact of timing settings and detector configuration. (Lin)

There are also been criticisms of the delay estimation equation for near or over-saturated conditions (e.g., vehicle arrivals exceed the available capacity). In addition, the effect of cycle length may not be properly accounted for in the model. Experience has shown that observed saturation flow rates may actually decrease for long cycle lengths (i.e., > 180 seconds). This is especially noticeable for through traffic.

In addition, the number of sneakers (e.g., vehicles entering the intersection during the yellow or all red intervals) appears to be influenced by left turn signal phasing (protected only vs. protected-permitted), opposing traffic flow, left turn flow and signal timing (e.g., max green time for protected phase), and geometric conditions. The current HCM model does not appear to adequately reflect these relationships. At many intersections, several left turning vehicles will attempt to enter when the amber left-arrow is displayed. This is especially true for left turns from exclusive left-turn lanes during over-saturated conditions and at intersections with long cycle lengths. At intersections with protected-permitted left turn phasing, left turning vehicles routinely enter and clear the intersection during both the change interval for the left turn phase (e.g., phases 1 plus 5) and the change interval for the subsequent through phase (e.g., phases 2 plus 6).

For permitted left turn analysis, the current HCM procedure assumes that the opposing queue clears and then is followed by unsaturated opposing flow. Some studies have concluded that it may be unrealistic. With good progression, a tightly packed platoon arriving after the queue may, in fact, block opposing left turns as completely as the standing queue. (Roess)

The current HCM procedure does not include the length of the left turn lane or bay in the model. This has been perceived as a major deficiency. Under certain flow conditions, the queue of left turning vehicles can exceed the available storage. At other times, the queue in an adjacent through lane can actually block entry into the left turn lane of vehicles desiring to turn left. Intuitively, there should be a difference in the capacity and saturation flow of a 100-ft long left turn lane compared to a 350-ft long turn lane.

Other criticisms of the HCM procedure relate to the saturation flow for exclusive left turn lanes includes the following:

- **A** fixed peak period of 15 minutes is used for analysis of level of service (LOS), assuming average conditions during the 15 minute period. The cycle-to-cycle variation in flows, delays and queues is not considered.
- The LOS analysis is not performed when the **V/C ratio** is over 1.2.
- The effect of platooning on ovefflow delays is overestimated through the application of progression factors which have poor correspondence to signal offsets.
- No estimate is made of the maximum overflow queue.
- No estimate is made of how long it will take to clear peak period congestion.

2.3 Summary of Deficiencies from the Perspective of the Practitioner

Most traffic engineers use the HCS or some other similar software package to determine the LOS at a signalized intersection. To gain greater insight into the problems that practitioners have with the current model, it is important to understand how and why they are using the HCM. Basically, the signalized intersection module of HCS is used primarily for the following:

- Operational analysis of existing signalized intersections, including evaluations of possible timing, phasing, geometric or other operational changes. This type of analysis typically is categorized by existing volumes and the condition that the intersection is currently signalized.
- Analysis of future conditions at intersections that either (1) do not exist, (2) currently are unsignalized, or (3) will undergo significant geometric changes (e.g., fourth leg added, major road widened with median, etc.). This type of analysis is typically categorized by predicted future volumes and the condition that it is a new intersection or there will be significant changes to an existing intersection.

From a practitioner's perspective, there are several deficiencies associated with the current HCM model for analyzing signalized intersections with exclusive left turn lanes. The following discussion summarizes some of the difficulties encountered by users of the HCS program, although many of the comments apply equally to other commercially available signalized intersection capacity software packages and the manual HCM procedures.

Signal Phasing

For future year analysis of new intersections or intersections converted from 3 to 4 legs, the user is left to his own devices to select the phasing. Many transportation professionals, notably entry level personnel and those with transportation planning backgrounds, have had great difficulty determining the most appropriate phasing for a new or substantially reconstructed intersection. It is possible for certain inconsistent phasing-lane geometry combinations to be accepted. For example, HCS will accept permitted left turn phasing for an approach with dual left turn lanes. Moreover, if left turning volumes are not that high, then it is possible to run the model with unrealistically low green times for protected left turn phases.

Users indicate that there is a great deal of latitude with respect to phasing input. Given the task of identifying the functional requirements for a future intersection, the user can vary the phasing and timing to get the average delay low enough for LOS "D" or LOS "C" for the intersection and/or the left turn lane group. Users will "play" with the left turn phasing and timing till they achieve an acceptable level of service. However, some users complain that there are no guidelines or guidance in the HCM to help them decide when a protected left turn phase is justified. Often, the decision is based on safety issues, the available intersection sight distance from the left turning vehicle to opposing traffic, a volume cross product (e.g., when left turns opposing traffic exceed 50,000 or 100,000), or policies established by the jurisdiction or the state.

Signal Timing

Most signalized intersection capacity software programs, including HCS, do not compute signal timings. For signalized intersections with actuated controllers, the procedure to estimate phase lengths in the appendix to HCM Chapter 9 is cumbersome, unwieldy, and time consuming. Based on a limited sample of practicing traffic engineers, it appears that very few use this manual procedure. There have been no definitive studies to indicate that the average green times estimated using the HCM procedures reflect the true operation of an actuated controller. There are also no guidelines to help traffic engineers estimate average phase interval durations for use with the HCM procedure based on actuated timing parameters (e.g., minimum green times, passage times, detector length and placement, maximum green times), average approach speeds, and average approach lane volumes.

For existing actuated controllers, it also appears that few actually measure times in the field. However, even if average effective green times are measured, those average green times may not be appropriate for future traffic volumes or different geometric conditions. Moreover, green times cannot be measured if the intersection is new.

Many practitioners admitted to using a trial-and-error approach to determine signal timing, especially when they are performing an analysis of future conditions.

Additional Input Required for Left Turn Lane-s with Protected and Permitted Phasing

For protected-permitted or permitted-protected left turn signal phasing, HCS forces the user to select one of the following three options:

1. Assign no vehicles to the permitted phase. (e.g., All left turns are made during the protected phase interval.)

2. Assign the maximum number of left turns to the permitted phase interval (i.e., the capacity of the left turn as calculated in Step 10, page 9-30 of the ***Highway Capacity Manual).***

3. Assign left turns to the permitted phase such that the v/c ratios for the permitted phase and the protected phase are equal. (i.e., assigned to achieve a balanced v/c ratio for protected/permitted portions.)

<u>Depending on the geometry. flows and green times specified, the selection of one option versus another can change the average delay calculation and LOS</u>! The user has no guidelines to determine which option is the most appropriate. Depending on the situation and (whether the traffic engineer represents the petitioning developer or the reviewing County agency), the user often uses the following logic:

- Most Conservative (option 1).
- Least conservative (option 2).
- Reasonable compromise when data are not available (option 3).

This aspect is one of the least desirable features of the current Highway Capacity Manual procedure. Often, the user does not have data to support the selection of one option over the other two options.

Real World Situations Not Adequately Modeled by HCS

Several traffic engineers questioned the appropriateness and applicability of the HCM operational procedures to specific situations related to exclusive left turn lanes at signalized intersections. The concern was that the current methodology does not adequately consider the events related to the situation and therefore taints the credibility of the results. The following describes several of those situations.

- A queue in the left turn lane that exceeds the left turn storage capacity. The queue adversely impacts traffic flow in the adjacent through lane.

- A queue in the through lane that blocks the entry of other vehicles into the exclusive left turn lane.

- Driveways on the receiving roadway that are in close proximity to the intersection. Based on observations and experience, some traffic engineers recognize that the operational efficiency of an intersection can be degraded when high volume driveways or intersecting streets are located too close to a signalized intersection. Improved access management principles and design practices have emerged over the years in response to the operational problems that result from poor access design for strip retail centers, frontage roads that are too close to intersections, less than adequate channelization, etc.

- Intersections that serve a high volume of U-turning traffic. If the median width and/or width of roadway in the opposite direction (i.e., the road that receives the u-turns) is not adequate, then the operational efficiency of the intersection can be adversely impacted. However, the current HCM procedure does not consider the effect of U-turns, turning radii, or intersection angle (e.g., skew). When these combinations exist, the HCM procedure may yield overly optimistic delay and LOS results.

- Left turn lanes on an up-hill approach that carry heavy trucks. <u>The current procedure assumes that for the same percentage trucks and percent grade, the f_{HV} for an exclusive left turn lane is equal to the f_{HV} for a through lane</u>. Intuitively, the percentage trucks has a more adverse effect on the saturation flow from an exclusive left turn lane than a through lane. Trucks travel at a slower speed when turning left at a signalized intersection than when travelling straight through.

- Dual left turn lanes with unbalanced flows. The HCM model assumes that the left turn traffic split between dual left turn lanes is 52.5 percent vs. 47.5 percent. However, this is not true for many intersections. For example, there may be a driveway on the receiving roadway that provides access to a shopping/commercial center. During certain periods of the day, the traffic distribution between the two lanes may be markedly different. Consider another example. As a capacity improvement technique, an auxiliary receiving lane may be created on the minor leg of a heavily congested intersection to receive traffic from dual left turn lanes. However, because the auxiliary lane might end 500 to 800 ft from the intersection, the left turn traffic distribution might be highly unbalanced.

- Intersections with wide medians and permitted left turn phasing. Several traffic engineers contend that the number of opposing travel lanes and the effective median width influence the left turn capacity of permitted left turn phases. The current HCM model does not require input for the median width. Consequently, it appears that the current model does not consider the effect of median or median width. One type of treatment that has been implemented is to offset the left turn lanes (i.e., closer to the opposing travel lanes) to improve intersection sight distance and increase operational efficiency at intersections with wide medians. The effect of this treatment on saturation flow has not been quantified.

Chapter 3 LEFT-TURN SATURATION FLOW ESTIMATION METHODS

As used in most existing methodologies, an accurate estimation of saturation flow constitutes the core of capacity analysis. Depending on the underlying assumptions, each methodology for saturation flow estimation employs different adjustment factors or models to capture the traffic flow interactions. Hence, in this chapter we first summarize the key features of existing methods for saturation flow estimation, and then discuss associated adjustment factors under various scenarios. The review intends to be as extensive as possible, including both international and U.S. literature. Some state-of-the-art approaches used in both academia and transportation agencies will also be reported.

3.1 Saturation Flow Estimation in the Revised HCM (Roess, 1989)

A revision to the HCM left-turn analysis methodology has recently been proposed by Roess, et. al. (1989). the revised procedure basically follows the original HCM concept, but recommends a different saturation flow rate as listed below:

Ideal saturation flow	:	1900 passenger cars per hour of green/per lane.
Ideal left-turn saturation flow	:	1805 passenger cars per hours of green per lane.

An empirical model for the determination of the left-turn adjustment factor in permitted phasing has also been prepared. The procedure takes the following variables into consideration:

g_g	=	amount of green time blocked to left turners by the clearance of an opposing queue of vehicles (seconds);
V_{Ltc}	=	average number of left-turning vehicles per cycle, vpc;
V_{oLc}	=	average number of opposing vehicles per lane per cycle;
P_0	=	The proportion of opposing vehicles, which arrive at the subject intersection approach during the green phase.

For exclusive left-turn lanes, the left-turn adjustment factor (f_{LT}) is given by a direct regression model as follows:

$$f_{LT} = 0.89 \sim 0.06 * g_g^{0.5} - 0.07 \times (V_{oLc} * V_{Ltc})^{0.5} \qquad \textbf{(3.1)}$$

where:

$$g_q = 9.532 * V_{oLc}^{0.56} x(1-P_o) \tag{3.2}$$

It can be noted that g_q increases with both increasing opposing flows and proportion of flows originating in standing queues. By definition, the average number of opposing vehicles per lane per cycle V_{oLc} is given by the equation:

$$V_{oLc} = \frac{V_o/N_o}{3600/C} \tag{3.3}$$

where:

V_o	:	opposing flow rate;
N_o	:	opposing number of lanes (excluding exclusive RT and LT lanes)
C	:	cycle length.

3.2 **Illinois Department of Transportation** (Rouphail, et. al. 1991)

Rouphail, et. al. (1991) in a study to validate the 1985 HCM procedures for capacity analysis, suggests the use of 2000 pcph.pl for through traffic and 1850 pcphpl for left-turn saturation flow rates. Based on some empirical results, they have further developed the following empirical models for predicting LT saturation flow rates.

<u>Saturation Flow Rates From Exclusive Left-Turn Lanes with Protected phasing</u>:

$$S_{LT} = S_i \; f_{LT} = 2000 \cdot \left(\frac{g}{C}\right)^{0.03146} \cdot F_y \tag{3.4}$$

where:

S_i	:	ideal saturation flow rate (2000 pcphpl)
g	:	the average displayed green duration per 15-minutes
C	:	the average cycle length per 15 minutes
F_y	:	the product of all other adjustment factors specified in the 1985 HCM

Saturation flow rates for exclusive left-turn lanes in the protected part of a protected/permitted phasing:

$$S_{LT}^{*} = 2000 \cdot \left(\frac{G}{C}\right)^{0.012} \cdot F_y \tag{3.5}$$

Permitted LT Adjustment Factors:

As used in the 1985 HCM, Rouphail et. al.(1991) calibrated the two principal components of LT adjustment factors based on empirical observations from Illinois. The mathematical equation for each component is presented below:

$$f_{LT} = f_{LT1} + f_{LT_2}$$

$$f_{LT1} = 1/S_1 \cdot \frac{g_u}{g_a} (1245 - 0.84 . AG_o) \qquad (3.6)$$

$$f_{LT_2} = 3.43/g_a$$

where:

f_{LT}	:	the adjustment factor for left-turning movements;
f_{LT1}	:	the component related to the left-turning vehicles which proceed in gaps in the opposing traffic during the permitted green time;
f_{LT2}	:	the component related to the number of left-turns occurring after the termination of the green period;
g_u	:	unsaturated portion of the permitted green time (second);
g_a	:	permitted green time (actual green) (second);
AG_o	:	opposing vehicles arrival rate during the green phase.

LT adjustment factor for the permitted subphase of the protected/permitted phasing from Illinois data is expressed as:

$$f_{LT} = g_u/g_a \cdot (1450 - 0.83 \cdot AG_o)/2000 + 5.19/g_a \qquad (3.7)$$

Note that the average number of sneakers is assumed to be 2.88 vehicles per cycle, slightly higher than that used in the 1985 HCM.

3.3 Japanese Roadway Capacity Manual (1978)

According to the results of extensive field measurements, the Japanese Roadway Capacity Manual adopts the following two different values for ideal saturation flow rates:

Through Traffic = 2000 pcphpl
Turning Movements = 1800 pcphpl

The above idea is that saturation flow rates can then be modified with appropriate adjustment factors to reflect the actual saturation flows under various conditions. These adjustment factors and their relations to the ideal saturation flow rate can be expressed as follows:

$$S_A = S_i \cdot \alpha_w \cdot \alpha_G \cdot \frac{\alpha_T}{\alpha_{RT}} \cdot \alpha_{LT} \tag{3.8}$$

where:

α_w : lane width (W) adjustment factor, and equal 0.95 if W = 2.5 - 3.0 m, or 1 if $W \geq 3.0$m. However, α_W is set to 1 if it is used for right-turning lanes and $W \geq 2.75$m.

α_G : grade adjustment factor which ranges from 0.95 to 0.75 when the corresponding grade varies from -6% to 6%.

α_T : adjustment factor for the large vehicles, which can be computed from the following expression:

$$\alpha_T = \frac{100}{(100-T) + E_T \cdot T} \tag{3.9}$$

T : the percentage of large vehicles in a mixed traffic flow; and

E_T : the pcu equivalent value of large vehicles (e.g., = 1.7) for through traffic.

α_{RT} : Adjustment factors for right turning vehicles (Note: equivalent to left-turning vehicles in U. S.) which can be obtained with the following equation:

$$\alpha_{RT} = \frac{100}{(100-k) + E_{RT} \cdot R} \tag{3.10}$$

Where R and E_{RT} denote the fraction of right-turning vehicles and their equivalent values in a through movement.

Note that under a shared lane scenario, E_{RT} is actually a function of various factors, including the adopted cycle length, opposing flow rate, opposing saturation flow rate, effective green time, etc.. The following equation provides an approximate value for E_{RT}:

$$E_{RT} = \frac{1.1}{f \cdot (S \cdot G - g \cdot c)/(G(S-q)) + 2k/G} \tag{3.11}$$

where:

S : saturation flow rate of the opposing traffic;

q : opposing through traffic;

k : the number of turning vehicles during the amber phase; and

f : the probability of turning successfully under various opposing flow rates.

Given the above adjustment factors, the right turning saturation flow in an exclusive lane and protected phasing can thus be expressed as follows:

$$S_{RO} = 1800 \cdot \alpha_W \cdot \alpha_G \cdot \alpha_T \tag{3.12}$$

A detailed discussion regarding the computation of capacity for exclusive lanes under various phasing strategies will be presented in the next chapter.

Note that α_{LT} is the adjustment factor for left-turning vehicles in a shared lane movement which is equivalent to right turns in U. S.. It can be approximated with Eq.(3.13):

$$\alpha_{LT} = \frac{100}{(100-L) + E_{LT} \cdot L} \tag{3.13}$$

where L and E_{LT} denote the fraction of left-turning vehicles and their through equivalent value, repectively. The E_{LT}, varying with traffic and pedestrian conditions, can further be computed from the following expression:

$$E_{LT} = \frac{1.1 \cdot G}{(1-f_p) \cdot G_p + (G-G_p)} \tag{3.14}$$

Where:

G_p : green time blocked by pedestrians

f_p : right-turn adjustment factor due to pedestrian movements.

In an exclusive left-turn lane operation the adjustment factor, considering only the pedestrian effect, can be simplified as follows:

$$\alpha_L = \frac{G - f_p(G-5)}{G} \tag{3.15}$$

3.4 Canadian Method for Saturation Flows Estimation (Teply, 1984)

There are three types of saturation flow defined in the Canadian Capacity Guide (1984), "Basic"," "Initial" and "Adjusted." The core concept of "Basic" saturation flow denotes the number of passenger car units which can discharge from the stopline of an "ideal" intersection lane and move straight through without any additional traffic friction under "ideal" Canadian weather conditions during an optimal length of the green interval. Canadian research (Teply, 1983) suggests that saturation flow varies with not only the population size of a community, but also the weather conditions and intersection environments. Some typical ideal saturation flow rates used in the Canadian Capacity Guide for through movements are presented below:

Table 3-1: Typical Ideal Saturation Flow Rates (pcphgl) in Canada

Area Type	Toronto	Calgary		Hamilton	Edmonton	
		summer	winter		summer	winter
Residential	1840	1750	1550	1650	1650	1400
Suburban	1750	NA	NA	NA	1550	1350
Industrial	1600	NA	NA	NA	1550	1350
CBD	1450	NA	NA	NA	1450	1350

The concept of "Initial" saturation flows reflects a set of typical conditions which modify intersection performance. The Canadian Guide recommends that a set of Initial saturation flow values be developed for every community or region based on local investigations. The "Adjusted" saturation flow can then be obtained by incorporating the specific local conditions in the "Initial" saturation flow.

The procedures for determining the "Adjusted" saturation flow for through lanes include the following factors:

- Lane width
- Gradient
- Queueing and discharge space
- Public transit
- Parking
- Duration of the green interval

For turning lanes, the "Adjusted" saturation flows need to further consider the geometric and traffic factors, such as:

- turning radius
- opposing traffic flows
- pedestrians
- effect of movement combinations which share one lane

Figure 3-1 summarizes the applicability of individual saturation flow adjustments to various lane function combinations.

However, it is well recognized that signal display practice is not uniform in Canada, and a careful local examination is thus highly recommended. In general, the Canadian Methodology considers no LT adjustment factor for the effect of the LT vehicles in the protected phasing design.

With respect to permitted phasing in exclusive left-turn lanes, a prediction model for left-turn saturation flow has been developed and used in Edmonton. The model has the following functional form:

$$S_L = S_o \cdot \exp[-0.00112 \cdot (f \cdot V_o \cdot \frac{C}{g_o})] - 100 \qquad (3.16)$$

where:

S_L	:	left-turn saturation flow, pcuphg;
f	:	gap probability and acceptance factor;
V_o	:	total opposing volume (opposing exclusive lanes not included), vph;
S_o	:	ideal saturation flow;
g_o	:	effective green interval for the opposing traffic (second).

Notably, the Canadian Guide emphasizes that all available procedures for the left-turn satuartion flow provide only an estimate value, and suggest the need to develop more accurate methods for dealing with more complex conditions.

3.5 **United Kingdom** (TRRL, 1986)

Due to the changes in vehicle performance, road markings, and lay out practice in recent decades, the Transportation and Research laboratory (TRRL) has conducted a study to

Lane Function	Possible Adjustments: Lane Width	Radius	Grade	Queueing Space	Discharge Space	Bus Stops	On-street Parking	Length of Green
Straight Through and Left	•		•	•	•	•	•	•
Straight Through and Right	•	•	•	•	•	•	•	•
Straight Through Only	•		•	•	•	•	•	•
Left Only	•		•	•		•		•
Right Only	•	•	•	•		•		•
Straight Right and Left	•	•	•	•	•	•	•	•

Source: (Teply, 1984)

Figure 3-1 An overview of possible adjustments to the Initial Saturation Flow value in relation to different lane functions update the saturation flow predicting model, originally published by Webster and Cobbe (1966). The proposed saturation model takes the effects of the following factors into account:

W/D	:	conditions of road surface
f	:	proportion of turning traffic
r	:	radius of turn
G	:	gradient
NS,NNS	:	lane position, nearside (NS) and non-nearside (NNS)
W	:	lane width
N_s	:	number of lanes at the stop line.

The basic saturation flow, containing no turning traffic and with gradient less than 1 percent, is given by:

S = (basic saturation flow) = 2080 pcu/h
S(r,f) denotes the saturation flows under the effect of r and f is given by:

$$S(r,f) = \frac{2080-140\delta_n}{1+1.5f/r}$$

$$\delta_n = \begin{cases} 1 \text{ for a nearside lane} \\ 0, \text{ otherwise} \end{cases}$$

By including the effects of gradient (-7.3% ~ 8.7%) and lane position, the saturation flow can be expressed as:

$$S(r,f,n,G) = \frac{(2080-140\delta_n-42\delta_g \cdot G)}{1+1.5f/r} \qquad (3.17)$$

Where δ_G equals 1 for uphill sites and 0, otherwise. Note that the coefficients, (140 and 42) were obtained from empirical observations.

For a roadway of non-standard lane with (3.25m), Equation (3.17) needs to be revised as following:

$$S(r,f,n,G,W_e) = \frac{[(2080-140\delta_n-42\delta_G G+100(W-3.25)]}{1+1.5f/r} \qquad (3.18)$$

In the case where opposed turners are present in the non-nearside lane, the above saturation flow needs to be further modified according to the following factors:

X_o : the intensity of traffic (the demand flow divided by the capacity per cycle) on the opposing arm;

f : the proportion of opposing turning traffic;

N_s : the number of storage spaces available within the intersection which opposed turners can use without blocking straight-ahead traffic;

λ : the number of signal cycles per hour 3600/C (where C is the cycle time in seconds)

The saturation flow for opposing turning traffic using an exclusive lane can thus be given by the equation:

$$S = S_g + S_c \tag{3.19}$$

where:

S_g : corresponding to the departures of vehicles during the effective green period, and can be obtained with Equation (3.20).

$$S_g = \frac{S-230}{1 + \left(\frac{0.5}{\gamma} + \frac{12X_0^2}{1-X_0^2}\right)} \text{, and } X_o = \frac{q_o C}{S_o g} \tag{3.20}$$

S_c : corresponding to departures immediately after the end of effective green (the clearance component), and is approximated by the following expression:.

$$S_c = P(1+N_s)\cdot x_0^{0.2}\cdot 3600\cdot c^2/g \tag{3.21}$$

Note that q_o and S_o are the arrival rate and saturation flows in the opposing stream; and P is the conversion factor from veh/hr to pcu/hr. In a mixed traffic flow, the conversion factor for use in Equations (3.20) and (3.21) can be computed from the following weighted average result:

$$P = 1 + \sum_i (a_i - 1) P_i \tag{3.22}$$

where P_i is the proportion of vehicles of type i, and a_i is the corresponding pcu value.

It should be mentioned that all of the above equations were developed on the basis of simulation experiments, but validated with field data collection. For permitted phasing, TRRL has further developed the relationship between opposing turning saturation flow and opposing volume from a comprehensive simulation experiment. The experimental results as well as relevant findings from other studies (Webster and Cobbe; and Kimber and Simens) are presented in the following Figure 3-2.

3.6 Swedish Method

The caculations of signal timing and capacity in Sweden between early 1960s' and 1970s' were mainly based on a manual developed by Nordgqvist(1958). The core concepts of this manual were mostly adopted from the 1950 HCM. In view of some deficiencies, the Swedish National Road Administration(SNRA) initiated a comprehensive study in 1971, including the development of computation methods for roads and for different types of unsignalized as well as signalized intersections. The final research results were published in 1978 as the revised Swedish Highway Capacity Manual. After the application for 10 years, the Swedish National Road Administration futher improved some procedures related to the capacity as well as signal timing design, and published the second revised Swedish capacity Manual and computer program (CAPCALZ) in 1989. The key concepts related to saturation flow estimation is briefly described below.

Base Values for saturation flow

To facilitate the application of appropriate methods, all lanes in the Swedish Capacity Manual are classified as different types as shown in Figure 3-3, depending on the presence of turning traffic and the degree of conflict experienced by the traffic in the lane.

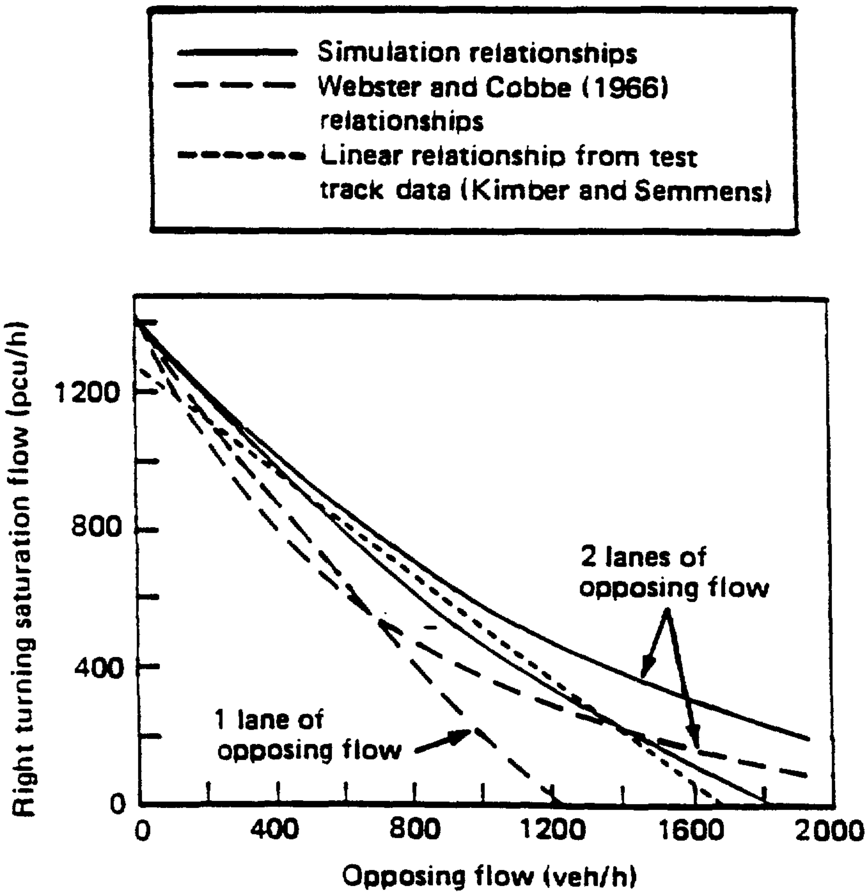
- Saturation flow for Lane Type A: 1700 vphg (i.e., only through traffic)
- Saturation flow for Lane Type B: 1500 - 1700 vphg, as a function of percentage turning
- Saturation flow for Lane Type C: 1500 vphg (i.e., only turning traffic)
- Saturation flow for Lane Types F and G (conflicts between left-turning and opposing flows)

$$S=\frac{3600}{g}(N_k+N_g+N_r) \quad vphg \tag{3.23}$$

Where:

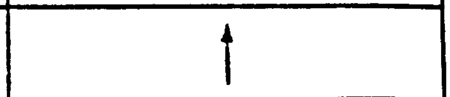
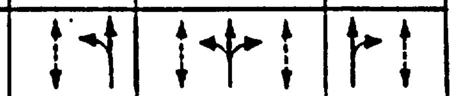
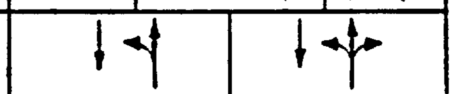
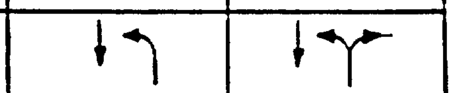
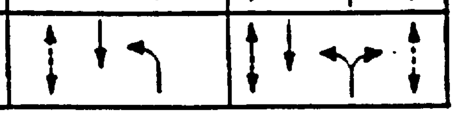
N_k: the total number of through and right-turning vehicles which can be discharged from the lane when the queue discharged from the opposite direction blocks left-turning vehicles during the first part of the green phase, g_k.

N_g: the total number of vehicles discharged through the remaining portion of green time, g_g=g-g_k, during which left-turning vehicles can pass the intersection when acceptable gaps occur in the opposing flow.



Source: (TRRL, 1986)

Figure 3-2 Relationships between right turning saturation flow and opposing flow for a number of studies

Type	Description	Illustration
A	Only through traffic	
B	Some turning traffic without conflict	
C	Only turning traffic without conflict	
D	Some turning traffic in conflict with pedestrians	
E	Only turning traffic in conflict with pedestrians	
F	Some turning traffic in conflict with opposing traffic	
G	Only turning traffic in conflict with opposing traffic	
D/F	Some turning traffic in conflict with opposing traffic and pedestrians	
E/G	Only turning traffic in conflict with opposing traffic and pedestrians	

Source: Swedish Capacity Manual

Figure 3-3 Classification of Lane-group Types

N_r: the number of vehicles passing through the intersection during the inter-green time (amber).

Based on the original notion by Gordon and Miller(1966), the Swedish Capacity Manual(SCM) adopts the following equation for estimating g_k:

$$g_k = \frac{q_m(c-g)}{S_m} \cdot \frac{1}{(1-\frac{q_m}{S_m})} = \frac{q_m(c-g)}{S_m-q_m} \qquad (3.24)$$

Where:

g	:	the length of green phase in the opposite direction
C	:	cycle time
q_m	:	flow(vph) in the opposite direction
S_m	:	saturation flow (vph) in the opposite direction

Note that with a given g_k the number of discharged vehicles, N_k, is a function of the fraction of left-turning vehicles that can queue at the intersection without blocking other vehicles in the same lane. Using a general probability, SCM has generated a numerical chart for estimating N_k (see Figure 3-4).

The total discharges during the second part of green phase, N_g, is a function of left-turning traffic P_1, and the probability for left-turners to have acceptable gaps in the opposing flow. Hence, it can be approximated with the following expression:

$$N_g = g_g \cdot S_g, \qquad S_g = \frac{q_m \cdot EXP(-a_g \cdot q_m)}{1-\exp(-a_f \cdot q_m)} \qquad (3.25)$$

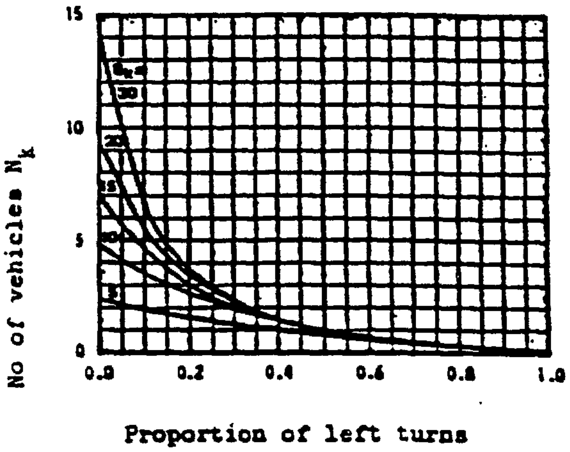
Where:

S_g	:	the saturation flow
a_f	:	the unit move-up time (=1/s)
a_g	:	the acceptable critical gap (from field measurement)

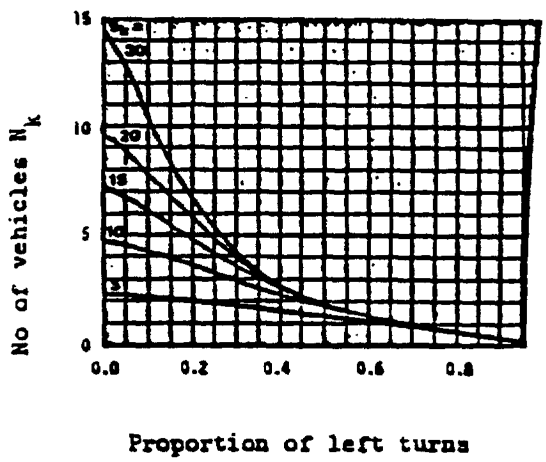
The number of vehicles which can be discharged during integreen (N_r) is recommended to be obtained from field measurements.

<u>Saturation Flow Adjustment for the length of approach lanes</u>

The SCM has explicity considered the effect of lane length on the saturation flow rate. For instance, in built-up areas the effective length of the curb lane is often limited due to parkings, bus stops or narrow street width. If the available length in a curb lane is too short, it may result in a substantial reduction in the capacity. Hence, the SCM recommends the following procedures for computing the reduced saturation flow:



No left-turning vehicle can queue without blocking the lane



One left-turning vehicle can queue without blocking the lane

Source: Swedish Capacity Manual (1978)

Figure 3-4 Number of vehicles, N_k, that can be discharged during g_k

Case A: The curb lane serves right-turning as well as through vehicles

It is assumed that the saturation flow will be reduced if the available lane length is shorter than the space occupied by the maximum number of vehicles (8m per vehicle) that can be discharged during the green time. More specifically,

$$S^1 = \frac{3600 \cdot L}{8.g} = \frac{450\ L}{g},\ if\ \frac{L}{8} < \frac{S \cdot g}{3600} \qquad (3.26)$$

where L denotes the available lane length, and S^1 represents the adjusted saturation flow.

Case B: The curb lane serves only right-turning vehicles

In this case the queue formed during red in the nearby lane might block the curb lane from being utilized to its full length. The likelihood for this blocking to occur is a function of lane length, green time, and the ratio of right-turning vehicles in the curb and nearby lanes.

The adjusted saturation flow S^1 can thus be obtained as:

$$S^1 = N \cdot \frac{3600}{g} \qquad (3.27)$$

where N is the number of right turns that can be made during a green phase. Note that under such a condition the curb and nearby lane are considered as one lane with a saturation flow equal to the sum of the individual saturation flows for each lane.

The described method with minor modifications can also be applied to estimate the effect of left-turn bay length.

Saturated Flows Estimation in the New SCM/CAPCAL 2 (Hasson and Bergh, 1989)

The procedure for estimating saturation flow rates is essentially the same as in the previous version, except the addition of some more correction factors such as bus stops and parking. However, a major difference lies in the definition and use of the saturation flow concept. Especially for a lane with secondary conflicts, the saturation flow is treated as a general, step-wise linear function, instead of a commonly-used constant value.

Original model: $S(g) = s \cdot g$

Revised model:
$$\begin{aligned} S(g) &= 0, \quad o< g <t_1 \\ &= S_1 \cdot (g-t_1), \quad t_1 \leq g \leq t_2 \\ &= S_1 \cdot (t_2-t_1)+S_2(g-t_2), \quad t_2 \leq g \leq t_3 \end{aligned} \qquad (3.28)$$

where:

S(g) is the number of vehicles that can be discharged during a green interval of effective length "g".

The key notion behind the revised model is that for a lane with secondary conflicts, the saturation flow rate will be zero during some part of green, and it may fall again when the initial queue has been emptied if the lane is short. A new procedure for estimating the time interval, t_1, t_2, etc. as well as the corresponding saturation flow rates S_1, S_2, etc., for each lane has been proposed in the new SCM. An example procedure for computing the saturation flow in the case of a short lane is described with the following example (see Figure 3-5):

Assumptions:

1. lanes 1 and 2 have a common green phase and no "blocked time".
2. The initial queues for lanes 1 and 2 (N_1, and N_2, respectively) are less than or equal to the maximum queue length (N) of the shorter lane.

In this case, let N_1=N and N_2<N, then during t_1 period:

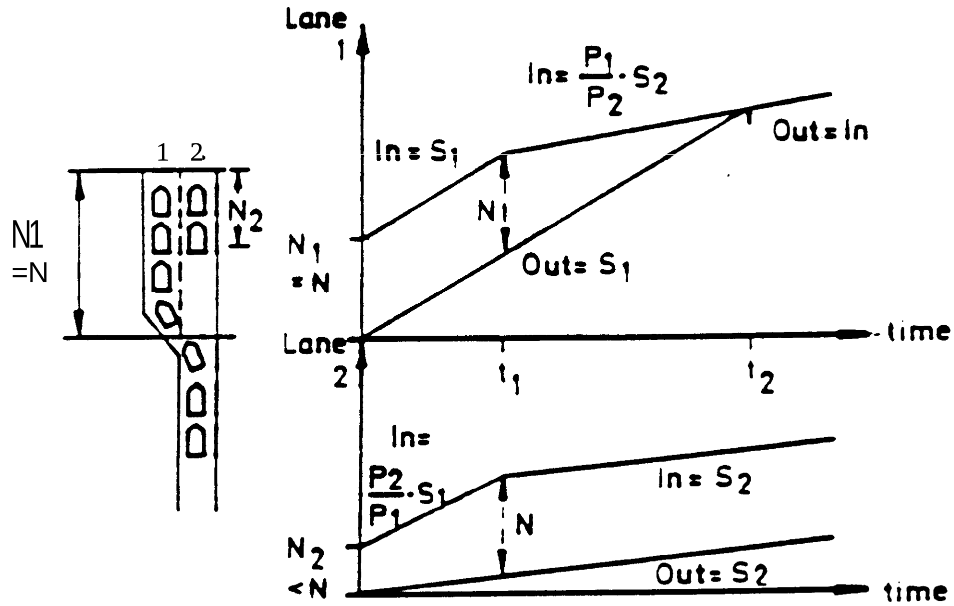
	Inflow	Outflow
lane 1	S_1	S_1
lane 2	$S_1 \cdot P_2/P_1$	S_2

Where S_1 and S_2 are unadjusted saturation flow for lanes 1 and 2, and P_1 and P_2 are the fractions of vehicles from upstream destinated to lanes 1 and 2, respectively. Assuming that $S_2 < S \cdot \frac{P_2}{P_1}$, then the queue in lane 2 will then grow, until it reaches the branching point after some time t_1. where:

$$t_1 = \frac{N_2 + (S_1 \cdot \frac{P_2}{P_1} - S_2)}{N} \tag{3.29}$$

Hence, the queue in lane 1 will be exhausted at some time t_2, etc., provided the green time is long enough and that the queue upstream of the branching point is also sufficiently long.

Note that the sum of the inflows to lane 1 and lane 2 cannot at any time exceed the capacity of a single lane with uninterrupted flow, namely, S_0 = 0.5 veh/sec. The initial inflows to lanes 1 and 2 would have been $P_1 \cdot S_o$ and $P_o \cdot S_o$, respectively.



Source: (Hasson and Bergh, 1989)

Figure 3-5 Example of a short lane saturation flow

Basically, the recently revised SCM considers all sequential events when a lane segment is either filled or emptied, in addition to the start and end of the effective green periods. At each such event, the queue, the inflow and the outflow to each lane is updated, and the times of future events revised. Such a procedure will thus result in a series of piece-wise linear saturation flow rates, $S_1, S_2, \ldots,$ with corresponding time intervals, $t_1, t_2, \ldots,$ for the short and for the adjacent lanes.

3.7 Australian Road Research Board Method (ARR 123, 1989)

The estimation concept introduced in Australian Research Record (ARR 123) has two distinct features: (1) saturation flows are expressed by "movement" rather than "phase", and (2) all related adjustment factors are expressed in vehicle units instead of through car units. The movements are described primarily according to the right-of-way provisions as determined by the signal phasing systems. The following rules based on the lane utilization and allocation are recommended for use in classification of movements:

- traffic in an exclusive lane shall be treated as a separate movement;
- traffic in an under-utilized lane shall be viewed as a separate movement; and
- combined traffic in lanes (including the shared lanes) with equal utilization shall be viewed as a separate movement.

The procedure for estimating movement saturation flows consists of three principal steps:

Step 1: Choosing a base saturation flow value for each lane allocated to the movement from Table 3.2 which gives general average saturation flows in through car units per hour (tcu/h) classified by the environment and lane types.

Step 2: Adjusting the base saturation flow value to allow for various factors affecting saturation flow in order to obtain an estimate of saturation flow in veh/hour for the particular movement.

Step 3: Adding lane saturation flows to determine the movement saturation flow.

Table 3.2 Average Saturation Plows in Through Car Units Per Hour for Estimation by Environment Class and Lane Type

Environmental Class	Lane Type		
	Through	Turning	Restricted Turning
A(ideal conditions)	1850	1810	1700
B(average conditions)	1700	1670	1570
C(poor conditions)	1580	1550	1270

Source: ARR 123 (1989)

The actual saturation flow can thus be obtained with the above base value and the selection of appropriate adjustment factors. The method can be summarized by the following formula:

$$\mathbf{S} = (f_w \cdot f_g / f_c) \cdot S_b \tag{3.30}$$

Where:

f_w: lane width adjustment factor, and

$$f_w = \begin{cases} 1.0 & \text{for } w = 3.0 - 3.7 \text{ (m)} \\ 0.55 + 0.14\ w & \text{for } 2.4 \text{ m} \leq w \text{ c } 3.0 \\ 0.83 + 0.05\ w & \text{for } 4.6 \geq w > 3.7 \end{cases}$$

f_g: gradient factor, and G_r is the percent gradient, and

$$f_g = 1 \pm 0.5\ g_r/100)$$

f_c: traffic composition factor (tcu per vehicle for a particular vehicle type and turning traffic mix).

The traffic composition factor is calculated from a weighted average value as follows:

$$f_c = \frac{\sum e_i \cdot q_i}{q} \tag{3.31}$$

where:

q_i: flow in vehicles for vehicle-turn type i

q: total movement flow

e_i: through car equivalent of vehicle-turn type i (tcu/veh)

It can be noted that f_c is actually a flow weighted average through car equivalent (in tcu/veh as for e_i). The through car equivalents for use in Eq.(3.31) can be taken from the following table.

Table 3.3: Through Car Equivalents for Different Types of Vehicles and Turns

	Through	Unopposed Turn		Opposed Turn
		Normal	Restricted	
Car	1	1	1.25	e_o
Hv	2	2	2.5	e_0+l

Source: ARR 123

<u>Saturation flow for opposed turns in exclusive lanes</u>:

In an exclusive lane, the saturation flow for opposing turns can be calculated with the same procedures as for a through lane, but using an opposing turn equivalent, Hence, an effective opposing turn saturation flow is given by:

$$s_O = 1800/e, \tag{3.32}$$

Where 1800 is the base saturation flow (in tcu/n) S_0 is in veh/h, and e_o is the opposed turn equivalent which can be obtained with the following expression:

$$e_o = \frac{0.5g}{s_u g_u + n_f} \tag{3.33}$$

where

g	:	green time(s) for the movement with opposing turns;
s_u	:	opposing turn saturation flow (veh/s);
g_u	:	unsaturated part of the opposing movements;
$s_u g_u$	:	number of turning vehicles (per cycle) which can depart during the green period g,; and
n_f	:	number of turning vehicles (per cycle) which can depart after the green period.

The basic notion behind Equations (3.3) and (3.4) can best be-illustrated with Figure 3-6, where the first model approximates the opposing turn saturation flow for share lanes, and Model 2 is most suitable for opposed turns from exclusive lanes.

The opposing turn saturation flow, S_u, during the unopposing part of opposing movement green period is derived from the the assumption of Poisson arriving patterns (Goldon and Miller, 1966; Fambro, et al, 1977, Peterson et al. 1978), and is given by:

$$S_u = \frac{q\ exp(-\alpha \cdot q)}{1 - exp(-\beta \cdot q)} \qquad (3.34)$$

where:

- q: opposing movement flow rate (veh/s)
- α: accepted critical gap (seconds)
- β: minimum departure headways for opposing turners.

Saturation Flow in Short Lanes

As is well recognized, a considerable number of situations exist which saturation flow on multi-lane approach roads is reduced due to what can in general terms be defined as short lane effects. The approach described in ARR 123 to deal with such an effect is quite similar to that used in United Kingdom, and can best be illustrated with Figure 3-7.
The total saturation flow S^1 in Figure 3-7 is equal to

$$S = S_1 \frac{g_1}{g^\cdot} + S_2 = S_1^1 + S_2 \qquad (3.35)$$

$$S_1^1 = 3600 \frac{D}{i \cdot g}$$

where:

- g_l : $(D/i)\cdot/S_i$ represents the time period during which the full saturation flow will last;
- D : the length of the short lane;
- i : the average queue space per vehicle;
- S_2 : the saturation flow of the available adjacent lane.

With the above notion, the following procedures are recommended for estimating the short lane saturation flow:

Step 1: Assuming that short lane effects do not occur and compute the full movement saturation flow;

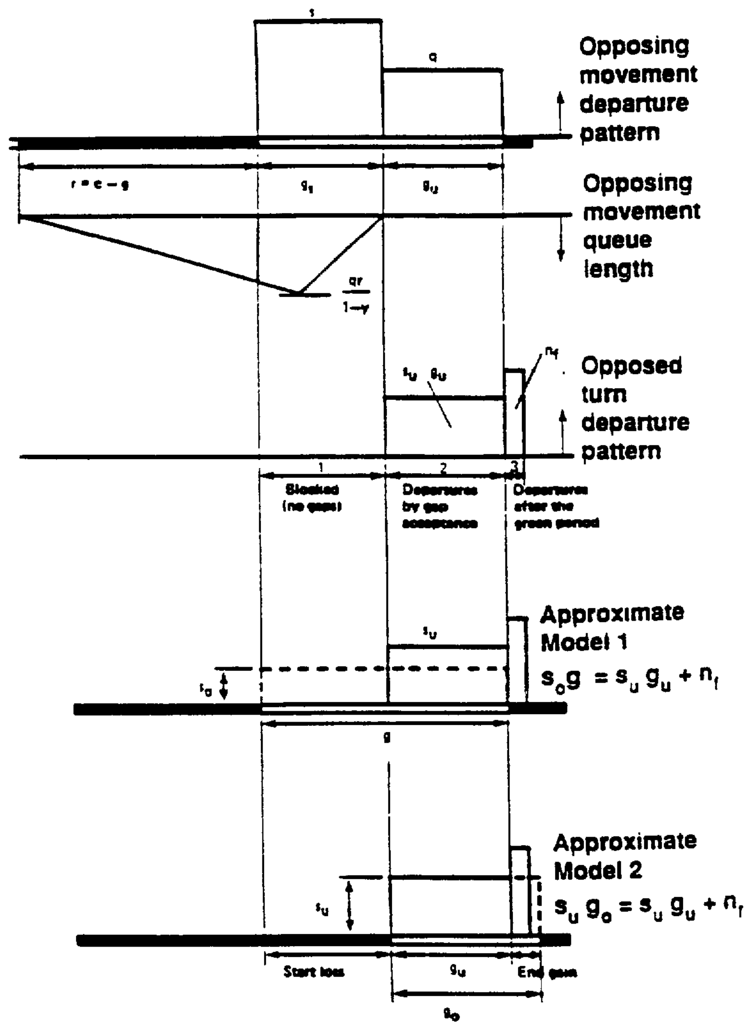
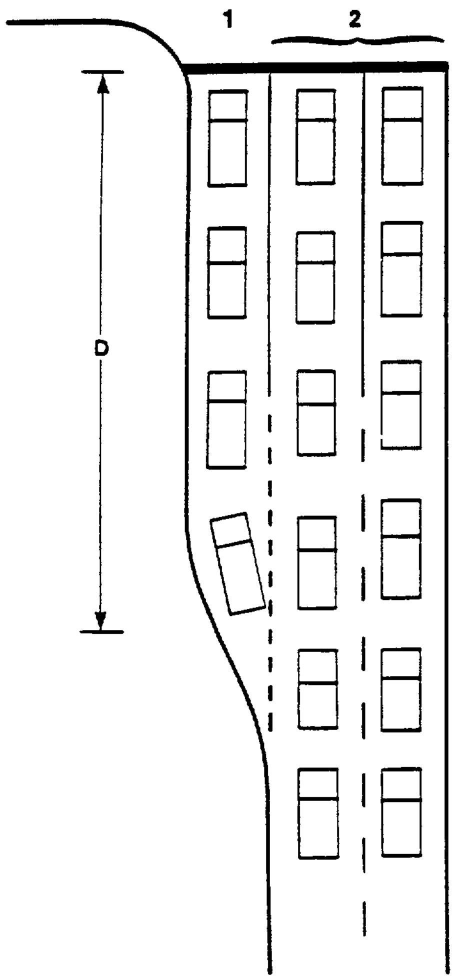
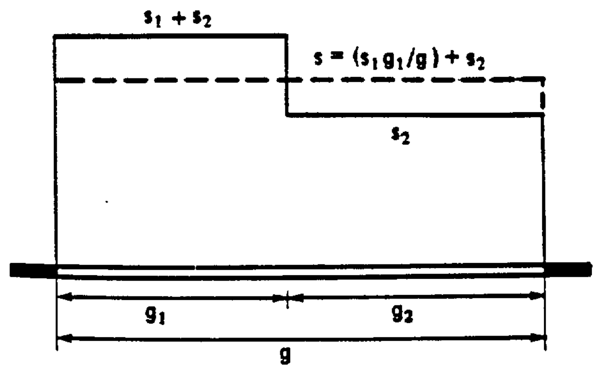


Figure 3-6 Opposed turn Model



—Queueing in an approach road with a short lane



—Short lane saturation flow

Figure 3-7 A Graphical Illustration of the Short Lane Saturation Flow Computation

Step 2: Computing the critical queuing distance, D_c from

$$D_c = \frac{iqr}{n(1-Y)} \tag{3.36}$$

where:

r : the effective red time (= c-g)
n : number of lanes available, including the short lane, and
Y : flow ratio (= q/s)

This formula is based on the assumption of uniform arrivals for calculating the maximum back of the queue in an average signal cycle.

Step 3: Compare the available short lane queuing distance (D) with the critical distance D_c, and concluding that no short lane effect, if $D \geq D_c$. Otherwise, computing S^1 using Eq. (3.35), if $D < D_c$.

Step 4: Check if the calculated short lane saturation flow, S_1^1, is less than the full lane saturation flow, S.

Note that to perform Step 4 one needs to convert S^1 to vehicle units (tcu's) as follows:

- Calculate $Q_1 = s_1^1$ (g/c) in veh/h
- Compute Q_1^* in tcu/n from

$$Q_1* \quad = q_L* + \frac{q_T*}{Q_T}(Q_1 - q_L), \quad if\ q_L < Q_1$$
$$= \frac{q_L^*}{q_L} Q_1, \quad otherwise \tag{3.37}$$

Where:

q_L, q_L^*: turning flow demand to use the lane under investigation in veh/h and tcu/h, respectively; and
q_T, q_T^*: through flow in veh/h and tcu/h, respectively.

- Calculate S_1^1 in tcu/h as $(Q_1^*/Q_1)\ S_1^1$, and check if this is less than S_1 (tcu/n). If so, calculate the movement saturation flow in tcu/h as $(S_1^1+S_2)$, where S_2 is the sum of basic saturation flows of the other lanes allocated to the movement. If $S_1^1 \geq S_1$ is found, use the full movement saturation flow.

3.8 Related Literature for Saturation Flow Estimation

In additional to the aforementioned procedures used in different countries, there are a number of saturation flow related studies in the literature. Some of those commonly referenced work are summarized below:

Tanner's Model for Left-turn Saturation Flows

For a single opposing flow, the saturation flow rate for a left turn can be estimated with the following equation:

$$S_L = \frac{2Q_0(1-Q_0\iota)}{\exp[2q(t_c-0.5\iota)]\,[1-\exp(-2q_0H)]} \tag{3.38}$$

$$\iota = 3 \text{ sec}, \quad t_c = 5\text{sec}, \quad H = 2.5 \text{ sec}$$

For two opposing flows:

$$S_L = \frac{2Q_0(1-q_0\iota)}{\exp[2q(t_c-0.5\iota)]\,[1-\exp(-2q_0H)]} \tag{3.39}$$

$$\iota = 1 \text{ sec}, \quad t_c = 6 \text{ sec}, \quad H = 2.5 \text{ sec}$$

Where:

S_L	:	left-turn saturation flow, veh/hr;
Q_0	:	opposing approach volume, veh/hr;
q_0	:	opposing flow rate, veh/sec/lane
ι	:	minimum headway of opposing flow;
H	:	average turning headway, sec, and
t_c	:	critical gap, sec.

Note that Tanner's model will yield the same results as that derived by Drew after removing the constraint on the minimum opposing headway. The basic assumptions underlying Tanner's model are that (1) opposing headways follow a negative-exponential distribution, (2) the gap-acceptance criterion is a step-function, and (3) two opposing flow q_0 are equivalent to a single stream with flow rate $2q_0$. This tends to underestimate the left-turn saturation flow in the case of two opposing flows. Moreover, the opposing traffic flow pattern after queue dissipation at signalized intersections are not the same as that by uninterrupted flows.

Drew's Saturation Flow Model

Drew's saturation flow for left turns is actually a special case of Tanner's equation, and therefore bears the same limitations. A closed form solution for left turn saturation flow is given by the following equation:

$$S_L = Q_0 \sum_{i=0}^{\infty} (i+1) P_r [t_c + iH \leq t \leq t_c + (i+1)H] \tag{3.40}$$

$$S_L = \frac{Q_0 \exp[-q_0 t_c]}{[1 - \exp(-q_0 H)]} \tag{3.41}$$

where:

S_L	:	left-turn saturation flow, veh/hr
Q_0	:	opposing traffic volume, veh/hr
q_0	:	opposing flow rate, veh/sec
t_c	:	critical gap, sec(e.g., 4.5 sec); and
H	:	turning headway, (2.5 sec if there is bay; otherwise, use 2.6 sec)

As can been seen from the equation, the entire model is grounded on three key assumptions:

1. The opposing traffic is an uninterrupted flow whose headway follows a negative exponential distribution;
2. There is a continuous left-turn queue; and
3. There are (i+1) left-turning vehicles that can go through the gap if the gap t is between t_c+iH and t_c+(i+1)H.

Note that both Drew's and Tanner's models concentrate the multiple opposing flows into a single stream, and disregard the stagger of gaps on several lanes. Based on such a saturation concept, Fambro and Webster have developed left-turn capacity methods for a permitted phase.

Regression Models for Left-Turn Saturation Flow Estimation

One of the most widely used models for left-turn saturation flow prediction has been developed by Michalopolus et. al. (1978). The proposed model is based on field observation results, and structured as the following multiple regression form:

$$S_L = -0.233 Q_0 t_c + 0.000015 Q_0^2 t_c^2 + 126X + 130Y + 995$$

where:

S_L	:	left-turn saturation flow per hour of green
Q	:	opposing approach volume, veh/hr
t_c^0	:	critical gaps
X=0,		for one opposing lane, and 1 for two opposing lanes
Y=0,		for unsignalized intersections and 1 for signalized intersections

Using the same approach, Mekemson has also calibrated a multiple regression model for estimating left-turn saturation flow. His model based on the simulation results contains no adjustment factors other than the opposing flows, but is the opposed turn procedure used in the SOAP program.

Some Studies Related to Exclusive Left-Turn Saturation Flow

Since the publication of 1985 version of the HCM, transportation professionals have been actively collecting suggestions of new HCM users, and devoting a significant level of effort on improving the existing methods, especially the procedures used in Chapter 9 - Signalized Intersections. Some studies which have identified serious deficiencies of the current HCM are reported below:

- The Effect of Platoons on Left-Turn Capacity: The current left-turn capacity model in the 1985 HCM was derived on the basis of using an average opposing flow rate throughout the cycle. This assumption may not be valid in the presence of platoons which may result in different arrival patterns during the red and green phases, respectively. Due to such concern, Mousa and Rouphail (1989) have investigated the effects of platoons on permitted left-turn capacity with field observations, and concluded that: (1) Permitted left-turn capacity decreases by improving progression for the opposing approach; and under the arrival types 2-5 the user can use the default values in the 1985 HCM for saturation flow estimation.

  Along the same line, Prevedouros and Jovanis (1984) have also investigated the effect of progression factors and actuated signal control on saturation flows. Their study indicated that the progression factors estimated from their field data are significantly different from the HCM values, and the saturation flows observed in the field are significantly higher than those in the HCM. They also reported that saturation flows for protected left turns in left-turn bays may be higher than those for through lanes.

- The Adjustment of the Opposing Flow Rate and Related Factor (Akcelik, 1989; Prassas and Roess, 1992): As can be noted from chapter 9 of the HCM, the ideal saturation flow of 1800 veh/hr is used in the computation of left-turn adjustment factor regardless of traffic as well as geometric conditions in the opposing lanes. This deficiency has been recognized by several researchers in the literature. Akcelik (1989) has further compared the results of HCM and SIDRA with some numerical examples, and indicated clearly that the opposed turn models in the HCM will grossly underestimate the degree of saturation and delay. Some simulation experiments conducted in this study have confirmed that the HCM model actually overestimates the left-turn capacity and underpredicts the total delay. This incorporation of such adjustments in the HCM opposed turn models thus becomes a vital research issue.

A recent study by Prassas and Rosess (1992) for permitted turns from share lane groups has further confirmed the necessity of modifying some left-turn capacity related factors in the HCM. These factors, at a minimum, include left-turn equivalent (Messer and Fambro, 1977), heavy vehicle equivalency, and the left-turn adjustment factor.

- The Effect of U-Turns on Saturation Plows: This issue has been recognized by practicing traffic engineers for quite a long time, but received increasing attention only recently. Preliminary results in the study by Adams and Hummer (1992) indicated that the saturation flows may be reduced up to 10 percent for U-turn percentages between 65 and 85, and these suggest that some type of adjustment factor may be necessary for left'lane groups with a large fraction of U-turns.

3.9 Saturation Flow Rates for Dual Left-Turn Lanes

In contending with the increasing traffic demand under existing limited urban network infrastructure, transportation professionals have recognized the increasing need to use dual left-turn lanes to relieve congestion or bottlenecks. One of the pioneering studies on this subject was undertaken by Capelle and Pinnell (1961). According to their collected time-headways in Houston, the saturation flow rates are calculated as follows:

Inside Lane	Outside Lane	Average	Through Lane	Average/Through
1500 vph	1616 vph	1568 vph	1714 vph	0.91

Ray (1965) reported on studies of dual left-turn lanes at signalized intersections in Sacramento county, California, and indicated a relatively low saturation flow of 1240 vph and 1230 vph, respectively, for inside and outside lanes. He also concluded that it would have at least a 75 percent increase in capacity by adding a second left-turn lane.

Along the same line, Assumes (1970) undertook field studies at seven cities in the Chicago area and found that the saturation flow rates for the inside and outside lanes were 1540 vph and 1550 vph, respectively. He noticed that the saturation flow of dual left-turn lanes might be affected by several factors, such as angle of turn, turning radii, medians on the approach, length of storage available, and volume in the adjacent through lane.

Kunzman (1978) computed the left-turn saturation flow rates from 175 locations in Orange county, and reported that the average value of saturation flow rates for a single and dual left-turn lanes are 1700 vphg and 1550 vphg, respectively.

The most recent study on this subject was conducted by Strokes st. al. (1986), in which the saturation flow rates at 14 intersections in three Texas cities were observed. The results of field studies revealed that the average saturation flows for dual left-turn lanes was 1636 vphg in the Austin and College Stations, and 1800 vphg in Houston sites. They recommended that a saturation flow rate of 1600 vphg be used for dual left-turn lanes in most planning applications. In the same study, Stokes et. al. investigated the interrelations between the saturation flow rates and traffic characteristics, and concluded that the following factors are significantly correlated with the left-turn departure headways:

- Turn bay taper length
- Turn bay storage length
- Approach grade
- Percent heavy vehicles
- Headway compression factor for each left-turn lane
- Left-turn green time

In that study, the "headway compression factor" was defined as the compression, or shortening, of the left-turn departure headways as the demand per cycle increases relative to capacity.

For convenience of comparison, the saturation flow rates for dual left-turn lanes observed in different studies are summarized in Figure 3-8.

Figure 3-8: Dual Left-Turn Saturation Flows

Source	Inside Lane	Outside Lane	Average
1. Capelle and Pinnell (1961)	1500 vphg	1636 vphg	1568 vphg
2. **Ray (1915)**	1240 vphg	1230 vphg	1235 vphg
-**3. Assmus (1970)**	1540 vphg	1550 vphg	1545 vphg
4. Kunzman (1978)			
Queue ≤4 veh/Iane		-	1439 vphg
Queue ≥5 veh/lane	-	-	1581 vphg
Other			1523 vphg
5. **Stoke (1986)**			
Austin and College Station			1636 vphg
Houston	.		1800 vphg
6. **HCM (1985)**			
10-ft lanes	1200 vphg	960 vphg	1080 vphg
11-ft lanes	1320 vphg	1056 vphg	1188 vphg
12-ft lanes	1440 vphg	1152 vphg	1296 vphg

Chapter 4 REVIEW OF EXCLUSIVE LEFT-TURN CAPACITY METHODS

The primary focus of this Chapter is to summarize existing literature related to the analysis of exclusive left-turn capacity, including the saturation flow and simulation based approaches. Some critical issues associated with the capacity estimation will also be discussed. This chapter is organized as follows: Methods related to the exclusive left-turn capacity under permitted, protected, and protected/permitted phasings are presented in sequence. This is followed by a discussion of the deficiencies of existing approaches and the research needs for improvements.

4.1 Exclusive Left-turn Capacity Under Permitted Phasing

The capacity estimation for permitted phasing is quite complex as it depends on a variety of factors, including traffic and geometric conditions in the subject lanes and opposing lanes, and driver characteristics. Existing analytical approaches for estimating the capacity under such a phasing plan often rely on some simplified assumptions which may not always be consistent with actual traffic characteristics. Hence, it remains a challenging issue for traffic professionals. Some commonly used methods for permitted left-turn capacity are summarized below:

- <u>**1985 HCM-Method**</u> - According to the procedures in Chapter 9 of HCM, the left-turn capacity (C_{LT}) can be expressed as:

$$C_{LT} = \frac{1800 \cdot F \cdot f_{LT} \cdot g}{C} \tag{4.1}$$

Where f_{LT} is the left-turn adjustment factor and F = product of all saturation flow rate adjustment factors other than the left-turn factor. For an exclusive left-turn lane, the factor f_{LT} is given by:

$$f_{LT} = \frac{g_u}{g}\left(\frac{1400-V_0}{1800}\right) + \frac{4}{g} \tag{4.2}$$

Thus, Eq. (4.1) can be restructured as follows for capacity estimation:

$$C_{LT} = (1800 \cdot F)\left[\frac{1400-V_0}{1800}\right]\frac{g_u}{C} + \frac{3600}{C}(2F) \tag{4.3}$$

where the the value (1800F) is the adjusted saturation flow in the LT lane if traffic in that lane were treated as through traffic; The factor $(1400-V_o)/1800$ is a saturation conversion factor from through to opposed left-turn traffic and is defined as $1/E_L$. The last term in Eq. 4.3 represents the capacity during the clearance interval (an average of two vehicles/cycle).

Note that Equation 4.2 employs an average saturation flow rate throughout the opposed green period. Also note that the ideal saturation flow (1800 veh/h) is used for the opposing saturation flow in HCM (Eq. 4.3).

- **1995 HCM - Revision (TRB, 1992)** - A revised procedure for left-turn capacity has been developed by the TRB Committee on Highway Capacity and Quality of Service and published in 1994. The major difference between the 1985 HCM and the revised procedures lies in the computation of the left-turn adjustment factor. The principal steps used in the revised procedures include:

Step 1: Compute g_f from an empirical regression model (Ross, et. al, 1988), $g_f = 0.0$ for exclusive permitted left-turn lanes.

Step 2: Compute g_q based on Equations (3.1) and (3.2)

Step 3: Compute g_u, where $g_u = g - g_q$ when $g_q \geq g_f$, and $g_u = g - g_f$ when $g_q < g_f$

Step 4: Select the appropriate value of equivalent factor from Table 4.1.

Step 5: Compute the left-turn adjustment factor using the following equations

$$f_m = [g_u/g]\,[\frac{1}{E_{L1}}] \tag{4.4}$$

$$g_q = 9.532\,V_{olc}^{0.560} \cdot q r_o^{0.819} - t_L \tag{4.5}$$

$$f_{LT} = [f_m + 0.91(N-1)]/N \tag{4.6}$$

where:

g_f = The portion of effective green until the arrival of the first left-turning vehicle (= 0, in an exclusive lane)

g_u = The portion of the effective green during which left turns filter through the opposing flow.

No. of Signal Phases	Type of Left Turn Lane	No. of Opposing Lanes	Opposing Flow, V_o					
			0	200	400	600	800	≥1000
2-PHASE	Shared	1	1.0	2.0	3.3	6.5	16.0*	16.0*
		2	1.0	1.9	2.6	3.6	6.0	16.0*
		≥ 3	1.0	1.8	2.5	3.4	4.5	6.0
	Exclusive	1	1.0	1.7	2.6	4.7	10.4*	10.4*
		2	1.0	1.6	2.2	2.9	4.1	6.2
		≥ 3	1.0	1.6	2.1	2.8	3.6	4.8
Multiphase	Shared	1	1.0	2.2	4.5	11.0*	11.0*	11.0*
		2	1.0	2.0	3.1	4.7	11.0*	11.0*
		≥ 3	1.0	2.0	2.9	4.2	6.0	11.0*
	Exclusive	1	1.0	1.8	3.3	8.2*	8.2*	8.2*
		2	1.0	1.7	2.4	3.6	5.9	8.2*
		≥ 3	1.0	1.7	2.4	3.3	4.6	6.8

* Generally indicates turning capacity only available at end of phase - "sneakers" only

Source: Messer and Fambro, "Critical Lane Analysis for Intersection Design," TRR 644, 1977

Table 4.1 Through-car Equivalents, E_{L1}, for Permitted Left Turns

g_q = The portion of effective green blocked by the clearance of an opposing queue.

N = Total number of lanes in a group.

$q\gamma_0$= Opposing queue ratio, i.e., the proportion of opposing flow rate originating in opposing queues, computed as 1 - R_{po} (g_o/C); and R_{po} = platoon ratio for the opposing flow.

Note that the recommended method treats the exclusive left-turn lane as a special case of share lane conditions, and employs the same traffic relations. The advantages of the revised method are that (1) reliable empirical regression models are used to estimate g_f, g_u, and g_q; and (2) a more accurate left-turn equivalent that considers the effect of the total number of opposing lanes is applied for the computation of the left-turn factor. In addition, the revised procedures also employs a different heavy vehicle equivalency based on the empirical study by Zegeer (1986). However, the impact of left-turn lane length or bay length on the capacity under different phasing schemes was not addressed.

- <u>**Recent Australian Model for Exclusive Left-turn Capacity**</u> (Akcelik, 1989) - The opposed-turn capacity estimation model in the early Australian capacity manual employed the adjustment concept as used in the HCM. The basic concept of the early version is to keep the green times unaffected but adjust saturation flows down based on the adjustment factor to allow for capacity loss. Such a method has been replaced recently by the lost time method, as illustrated in Figure 3-6, and implemented in SIDRA, an Australian computer program for capacity analysis. The key features of the recent Australian methods for exclusive opposed lane capacity are described below:

1. <u>Opposing saturation flow, S_{op}, computation</u>: HCM uses the ideal saturation flows (1800 veh/h) for the opposing flows regardless of the geometric and environmental factors. In contrast, the SIDRA model employs the estimated flows based on individual vehicle classes (e.g., heavy vehicles), and all other adjustment factors such as bus stops, lane width, gradient.

2. <u>Predicting the unsaturated part of opposing green period</u> (g_u): In the Australian method, g_u can be calculated from the following equation:

$$g_u = \frac{g-yc}{1-y} \tag{4.7}$$

where y and g are the flow ratio and effective green time for the opposing movement, and C is the cycle length.

3. <u>Estimating the opposed turn saturation, S_u, during g_u</u>: The filter turn saturation flows, S_u, in both the HCM and ARR 123 are based on an exponential gap distribution assumption (e.g. Gordon and Miller, 1966; Fambro et al., 1977),

which do not take the number of opposing lanes into account. In SIDRA, a new gap acceptance formula is implemented. The model, based on the research results of Gipps (1982) and Troutbeck (1984), takes the individual lane flows into account:

$$S_u = \frac{\lambda\theta\exp[-(\alpha-\Delta)\lambda]}{1-\exp[-\beta\lambda]} \tag{4.8}$$

where: α is the accepted critical gap (seconds), β is the minimum departure headway, Δ is the minimum headway in an opposing traffic lane, and the parameters λ and θ are calculated from:

$$\lambda = \sum \frac{\phi_i q_i}{1-\Delta q_i} \tag{4.9}$$

$$\theta = \pi(1-\Delta q_i) \tag{4.10}$$

where the summation and multiplication are for lanes i = 1 to N; ϕ_i is a bunching factor (proportion of unbunched vehicles in the i^{th} opposing transfer lane), q_i is the flow rate in the i^{th} opposing lane.

Note that based on Eq. (4.8), S_u will decrease as the lane utilization ratio increases, and it approaches the one-lane S_u value when one of the lane flows approaches the total flow value. Thus, the new implemented filtering model has the advantage of sensitivity to the number of opposing traffic lanes as well as the lane utilization in opposing traffic lanes.

4. <u>Computing the capacity per cycle</u>: The new Australian method employs the "lost time" approach as illustrated in Figure 3-6. The capacity per cycle can thus be computed from:

$$S = (S_u g_u/3600) + N_f \tag{4.11}$$

where N_f is the number of departures after the end of green period.

5. <u>Computing the effective green time, g_o, average saturation flow and capacity</u>: Given the capacity per cycle, one can then obtain the capacity per hour from the following sequence of computations:

$$g_o = g_u + (N_f/S_u) \quad (4.12)$$

$$s = \frac{S}{g_0}, \quad and \quad Q = \frac{sq_0}{C} \quad (4.13)$$

where s is the average saturation flow, and Q is the opposed turn capacity per hour.

Note that the latest Australian method (SIDRA) has eliminated the use of opposed turn adjustment factors, and employs a direct approach to model individual lane capacities. The sum of all individual lane capacities is used to represent a lane-group capacity.

- **Other Permitted Left-turn Capacity Related Methods** - Except for the new Swedish method, most early literature on permitted left-turn capacity employed the same concept as that used in HCM. Basically, the permitted opposed turn capacity in exclusive lanes is a function of the unsaturated portion of green time and the saturation flow. The procedures and factors for saturation flow computation, however, may vary with the assumptions used by different methodologies. For instance, the left-turn capacity using the method by Michelopulos (1978) is expressed as:

$$Q_L = [(gs-q_oC)\, S_L/C(s-q_o)] + 3600(K/C) \quad (4.14)$$

where:

S_L = left-turn saturation flow;
g = effective green time;
S = saturation opposing flow rate;
C = cycle length; and
K = number of left-turns during intergreen periods.

The first term in Eq. (4.14) is used in both Webster's and Drew's models and some other analytical approaches, except that each model may use a different method to compute the filtering rate of opposed saturation flows.

Rouphail et. al.(1991) developed a predictive model for assessing the permitted left-turn capacity in an exclusive lane, based on empirical observations collected in Illinois. The capacity model is expressed as:

$$C_{LT} = S_1 \cdot F \cdot f_{LT} \cdot \frac{g_a}{C} \quad (4.15)$$

where g_a is the actual permitted green time; F is the product of all saturation flow rate adjustment factors excluding f_{LT}. An empirical function for estimating f_{LT} is given by:

$$f_{LT} = \frac{g_u}{g_a} \cdot \left(\frac{1244 - 0.84 AG_0}{2000} \right) + \frac{3.43}{g_a} \qquad (4.16)$$

Although the proposed function is based only on a very small data set, and the observed opposing flow was confined in the range of 218 to 1062 vph, it provides a promising direction to incorporate essential variables in determination of the left-turn factor.

- <u>Simulation-Based Method</u> (Texas Model) - As is well recognized, mathematical models are useful tools for analyzing complex left-turn operations. However, to assure the mathematical tractability, one may need to make many simplifying assumptions which may not always be consistent with field observations. For this reason, Lin et. al. (1984) proposed an innovative method that integrates the simulation results with statistical models and directly estimates the left-turn capacity under various scenarios. Since the core of the methodology is a microscopic simulation program, one can realistically take all capacity related factors into account, including the effects of cycle length, cycle split, opposing lanes, left-turn bays, headway distributions, and trucks on left-turn capacity. The original concept used by Lin et. al. for permitted left-turn capacity estimation is illustrated below:

1. Defining the new variable "transparency": a term first adopted by Herman and Weiss (1961) in studying the highway crossing problem. It can be defined as the ratio of the total acceptable gap time to the total observed time, or the total time gap. To some extent, transparency characterizes the overall impedance of the opposing traffic and the signalization to left turns.

2. Conducting a set of simulation experiments with given traffic, geometric characteristics, and signal design strategies.

3. Recording all key input and output variables, including total observed time (or simulation time), total time after the opposing queue is cleared, total acceptable gap time, percent of time after the opposing queue is cleared, transparency, total number of left-turns through gaps during simulation time, total number of left-turns made in amber periods, the left-turn capacity, and the average left-turn processing time.

4. Establishing the empircal relation between transparency and opposing volume. According to Lin's study, the following linear relation can be identified:
   One opposing lane:

$$T = 1916 - 2.763Q_0\,(vph)$$

$$or \qquad = 0.5322 - 0.0007675Q_0\,(veh/second) \qquad (4.17)$$

5. Computing the average left-turn processing time ($\bar{t}$) as follows:

$$\bar{t} = (3600T)/Q_L \qquad (4.18)$$

Note that the average left-turn processing time was found to be approximately constant at 4.36 seconds for opposing volumes from 100 to 500 vph, and to converge to 3.0 seconds as the opposing traffic approaches saturation. Hence, when the opposing volume falls between 100 vph and 500 vph per lane, the left-turn capacity can be approximated by:

$$Q_L = 3600T/4.36 = 825T \qquad (4.19)$$

Replacing transparency (T) with Equation (4.17), the left-turn capacity can be approximated by a piecewise linear function of the opposing volume as follows:

$$Q_L = 439 - 0.634Q_0, \quad if\ 0<Q_0<500\ vph \qquad (4.20)$$

$$Q_L = 295 - 0.348Q_0, \quad if\ 500<Q_0<675\ vph \qquad (4.21)$$

The slope of 0.634 implies that one opposing vehicle is equivalent to 0.634 left-turn vehicles.

The effect of cycle length: A set of simulation experiments was conducted by Lin et. al, (1984) to examine the potential impacts of cycle length on the left-turn capacity. The results of experiments with the TEXAS model reveal that except for the likely increase in sneakers the left-turn capacity seems to be insensitive to changes in cycle length as long as the G/C ratio remains the same.

The effect of cycle split: Given the capacity from a fixed G/C ratio, Lin et. al claimed that such a relation can be extended to estimate the left-turn capacity under various cycle splits. For instance, an opposing volume Q_0^1 under any G/C ratio can be converted to an opposing volume Q_0 under a G/C ratio of 0.5. With some approximation and mathematical transformation, such a relation is given below:

$$T^1 = 1.064(G/C) - 0.0007675Q_0^1 \qquad (4.22)$$

Where Q_0^1 is the opposing volume under any cycle split, and T^1 is the resulting transpancy.

The left turn capacity for any G/C ratio can be caculated as follows:

$$Q_L^1 = \frac{3600T}{\bar{t}} = \frac{[3830(G/C) - 2.763Q_0^1]}{\bar{t}} \qquad (4.23)$$

Since $\bar{t}$ can be approximated by a constant of 4.36 seconds, if the opposing volume, Q_0^1, is less than 1000 vehicles per hour of green, Equation (4.23) can thus be simplified as follows:

$$Q_L^1 = 879(G/C) - 0.634Q_0^1 \qquad (4.24)$$

The effect of multiple opposing lanes: Unlike most existing analytical models which regard the multiple opposing flows as a single stream, the proposed transparency concept allows reasonable consideration of staggering opposing lanes, and multiple checkings for acceptable gaps on each lane. The basic concept is to view the actual left-turn capacity of multiple opposing lanes as an average of the worse and the best scenarios. Let $Q_L = f(Q_0)$ be the left-turn capacity for a single opposing flow, and Q_{LN} be the left-turn capacity for N opposing lanes. Then, the actual capacity under multiple opposing lanes can be stated as:

$$Q_{LN} = [f(Q_0) + f(Q_0/N)]/2 \qquad (4.25)$$

Where $f(Q_0)$ is the capacity that is obtained by treating all opposing flows as a single stream, and $f(Q_0/N)$ is the capacity for evenly distributed flows. Lin reported that the estimated left-turn capacities with Equation (4.25) on the average are only about six percent less than the simulation results.

Based on the notion of transparency, the proposed simulation-based method for left-turns can consider the effect of left-turn bay length, opposing headway distributions, and truck percentage on the capacity estimation. This is by far the most flexible, convenient, and perhaps most accurate method found in the literature to deal with the complex left-turn capacity analysis, if the employed simulation model functions properly and yields results consistent with field observations.

4.2 Left-Turn Capacity for Protected Phasing

The procedure for estimation of left-turn capacity for protected phasing is relatively straight-forward, compared to permitted phasing. In most existing literature, a protected left-turn capacity is always the result of the estimated left-turn saturation flow and the given G/C ratio for the left-turning phase. Such a relation can best be illustrated with the following equation:

$$C_{LT} = S_{LT} \cdot (g_{LT}/C) = S_1 \cdot f_{LT} \cdot (g_{LT}/C) \tag{4.26}$$

Where S_{LT} is the left-turn saturation flow; and g_{LT}/C is the green ratio for the left-turn lane. A reliable estimate of the capacity can be accomplished if the saturation flow is accurately predicted.

In reviewing existing literature for saturation flow estimation, as summarized in the previous chapter, it is notable that most methods use a simple adjustment factor to relate the ideal through saturation flow with the ideal left-turn saturation flow in a protected phase. For instance, HCM suggests the use of f_{LT}=0.95 for the protected phase from an exclusive lane. The potential impact of queue length during each cycle, especially under actuated control, on the left-turn capacity, however, has not been adequently addressed. A recent study by Rouphail et. al. (1991) attempted to address this issue by incorporting it into the determination of a left-turn adjustment factor. A multiplicative regression model based on their field data was proposed to represent the left-turn adjustment factor under protected phasing from exclusive left-turn lanes. The proposed model is expressed as follows:

$$f_{LT} = (g/C)^{0.03146}, \quad and \tag{4.27}$$

$$S_{LT} = 2{,}000 \cdot f_{HV} \cdot f_{LT} \tag{4.28}$$

According to the report by the authors, the above model for left-turn capacity eatimation seems to perform reasonably well on some selected sites in Illinois. However, it should be noted that Equation (4.27) was developed with a relatively limited data set. Its main contribution is to indicate a new direction for investigating an accurate left-turn adjustment factor.

4.3 Protected/Permitted and Permitted/Protected Phasings

The protected/permitted (PT/PM) or PM/PT are the most complex phasing plans for signalized operations. A thorough analysis of the left-turn capacity under such phasing plans involves the estimation of saturation flow rates for the primary and secondary green times, the computation of lost time, the allocation of arriving flows to each subphase, and the projection of headway distribution patterns. Hence, to develop a convenient yet reliable procedures for PM/PT and PT/PM remains as the foremost research issue. Some procedures available in the literature are summarized below:

Austrialian Method (ARR 123): The core concept presented in the ARR 123 report for dealing with the combination of PT/PM phasing is to model the opposed and unopposed periods separately as two subphases with two different saturation flows. The capacity for each subphase can then be estimated with appropriate methods and add up to be the total capacity of the given phasing.

Under such a concept, there are basically two alternatives for treating the problem of two saturation flows per movement. The first approach is to combine the opposed and unopposed periods and treat them as a single period with a capacity equivalent to the sum of the capacities available during the two periods. It can be expressed as:

$$S = (S_A g_A + S_B g_B)/g \tag{4.29}$$

Where:

S_A	=	the normal saturation flow during the unopposed subphase;
S_B	=	the reduced saturation flow during the opposed subphase;
g_A, g_B	=	effective green times for the unopposed and opposed phases;
g	=	$G_A+G_B+l_B$ = the total allocated green time during which the average saturation flow, S, is considered to exist.

The saturation flows (S_A, S_B) can be calculated independently with the methods presented in Chapter 3. The total capacity for a given PM/PT or PT/PM phase is thus the sum of two individual subcapacities. It should be noted that the number of sneakers between the opposed and unopposed periods is zero, because there exists no intergreen period between them. Care should also be given to the computation of the effective green time for each subphase so that the saturation flow can be properly estimated.

Illinois Model (Rouphail et. al. 1991): In an attempt to evaluate the 1985 HCM procedures for left-turn capacity, Rouphail et. al. conducted a rigorous field study, and developed a predictive model for each subphase of a PT/PM phasing design. Their empirical investigations lead to the following conclusions:

- The left-turn saturation flow rate for the protected subphase of a PT/PM phase is 1960 pcphgl, slightly higher than those found for the protected only case.

- The left-turn saturation flow during the protected subphase, varying with the queue length and G/C ratio, can be estimated with the equation:

$$S_{PT} = 2000\cdot(G/C)^{0.012\cdot F} \tag{4.30}$$

- The left-turn adjustment factor for the permitted subphase can be expressed as the following function:

$$f_{LT/PM} = \frac{g_u}{g_a}\cdot(1450-0.83\cdot AG_0)/2{,}000+5.19/g_a \qquad (4.31)$$

Where AG_0 is the average opposing flow rate during the green period.

- The capacities estimated under the permitted subphase is slightly higher than those predicted under permitted phasing only.

To facilitate the application, they further developed a combined model for estimating the capacity in both a permitted-only phase and a permitted subphase. The function to represent the left-turn factor in both cases is given by:

$$f^{*}_{LT/PM} = \frac{g_u}{g\cdot S_i}\cdot[1243.5+202.4\cdot\delta-(0.84-0.014X)\cdot AG_0]+[\frac{3.43+1.76X}{g_a}] \qquad (4.32)$$

Where S_i is the ideal saturation flow, and δ is a binary variable that is equal to 1 for PT/PM phasing and 0, otherwise.

<u>Revised HCM Procedures</u> (TRB, 1992): The TRB committee on Highway Capacity and Quality of Service has suggested the use of modified permitted and existing protected capacity estimation procedures for analyzing PT/PM phasing. The core idea is to separate the portions of a given PT/PM phase into two separate "lane groups", and apply the protected-only as well as permitted-only procedures for each subphase or lane group. The recommended procedures further assume that:

- The <u>first</u> portion of the phase, whether protected or permitted, is assumed to be fully utilized, i.e., and assumed to have a V/C ratio of 1.00, unless total demand is insufficient to use the capacity of that portion of the phase.

- Any remaining demand not handled by the first portion of the phase is assigned to the second portion of the phase, whether protected or permitted.

Note that with the above assumptions one does not need to arbitrarily divide the demand volume between the protected and permitted phases. However, the new procedures will require the users to carefully analyze the relations between phasing sequence and the resulting vehicle movements, especially for the selection of key parameters such as G (actual green time), g (effective green time), g_q, and g_f for the permitted subphase. These key parameters need to be properly estimated so that the capacity of the permitted subphase can be computed with those equations (see Eqs (4.31) and (4.32)) developed recently for a permitted-only phase.

Due to the complex interactions between phasing plans for each lane group, the proposed new concept for PM/PT or PT/PM does not provide any efficient steps for computing those key parameters. The users are required to graphically analyze the phasing, movement plans, and lost times so as to correctly determine those parameter values.

4.4 Research Issues Related to Exclusive Left-Turn Capacity Estimating

As is well recognized, the development of effective yet convenient procedures for left-turn capacity estimation remains to be a challenging issue for traffic researchers. A large number of articles related to the left-turn issues continues to appear in transportation conferences or literature over the past decade. This section intends to summarize some of the research issues discussed in the recent literature which are directly associated with the capacity estimation of exclusive left-turn lanes. These identified issues may also constitute the basis for the development of appropriate traffic models in Task B of this research project.

Some critical left-turn related issues which have not been adequately addressed by the HCM are described below:

- An extensive field measurement may be necessary to determine the accurate saturation flow rate. For instance, ideal left-turn and through saturation flow rates under different environmental conditions deserve a rigorous investigation, because some empirical studies indicate that the basic saturation flow rate is somewhat unstable over time and locations.

- In assessing the impact of opposing flows on the permitted left-turn capacity, it may be more appropriate to use adjusted flow rate than the ideal saturation flow rate currently used in the HCM.

- The permitted left-turn factor may be more related to the opposing flow rate during the green period than the average opposing flow rate during the cycle.

- The left-turn equivalent factor should incorporate the effect of the number of opposing lanes.

- The effect of opposing flow arriving patterns on the left-turn capacity should be quantified and allowed for a convenient measurement of key parameters, because the arriving patterns may vary over time, depending on their interactions with upstream traffic conditions.

- A rigorous field measurement may be necessary to accurately estimate the heavy vehicle conversion factor in through and turning movements.

- The effect of bay length on the exclusive left-turn capacity needs to be fully investigated, including its interaction with saturation flow rate in adjacent lanes.

- A more rigorous model than the existing one in the HCM is necessary to compute the opposed turn filtering rate under various opposing volumes.

- The potential effect of pedestrian movements on the left-turn flows has not been adequately addressed.

- The assumption used to develop the relation between an individual and lane-group saturation flow rates may need a careful reassessment.

- In estimating the left-turn saturation flow rate, the effect of intersection turning radius may need to be taken into account.

- The average queue length that is a function of G/C ratio may play an important role in the estimation of protected left-turn saturation flow rate.

- A reliable model or procedure, which is convenient and accurate for estimating the effective green time and all related parameters for each subphase of a protected/permitted design is one of the foremost tasks in the left-turn related studies.

- An empirical statistical model, based either on field measurements or simulation experiments, should be developed to project the distribution of arriving flows during each subphase of a PT/PM phase under various conditions.

- Most importantly, a standard procedure for measuring the saturation flow rate should be developed and provided for use by the transportation community.

Chapter 5 RECOMMENDATIONS

This chapter presents some preliminary concepts to be followed by the research team in conducting the second task of this project - Development of a Traffic Initial Model. The discussion will be divided into three parts: key model parameters to be calibrated directly from field measurements or simulation experiments are identified first. This is followed by a description of **some** vital functional relations to be tackled with analytical approaches. Finally, critical issues to be resolved with an integrated method (i.e., empirical and mathematical formulations) are presented in the last section.

5.1 Some Key Variables or Parameters to be Obtained Directly from Empirical Studies

With well-designed procedures approved by FHWA, the research team suggests that the following variables be collected directly from field measurements.

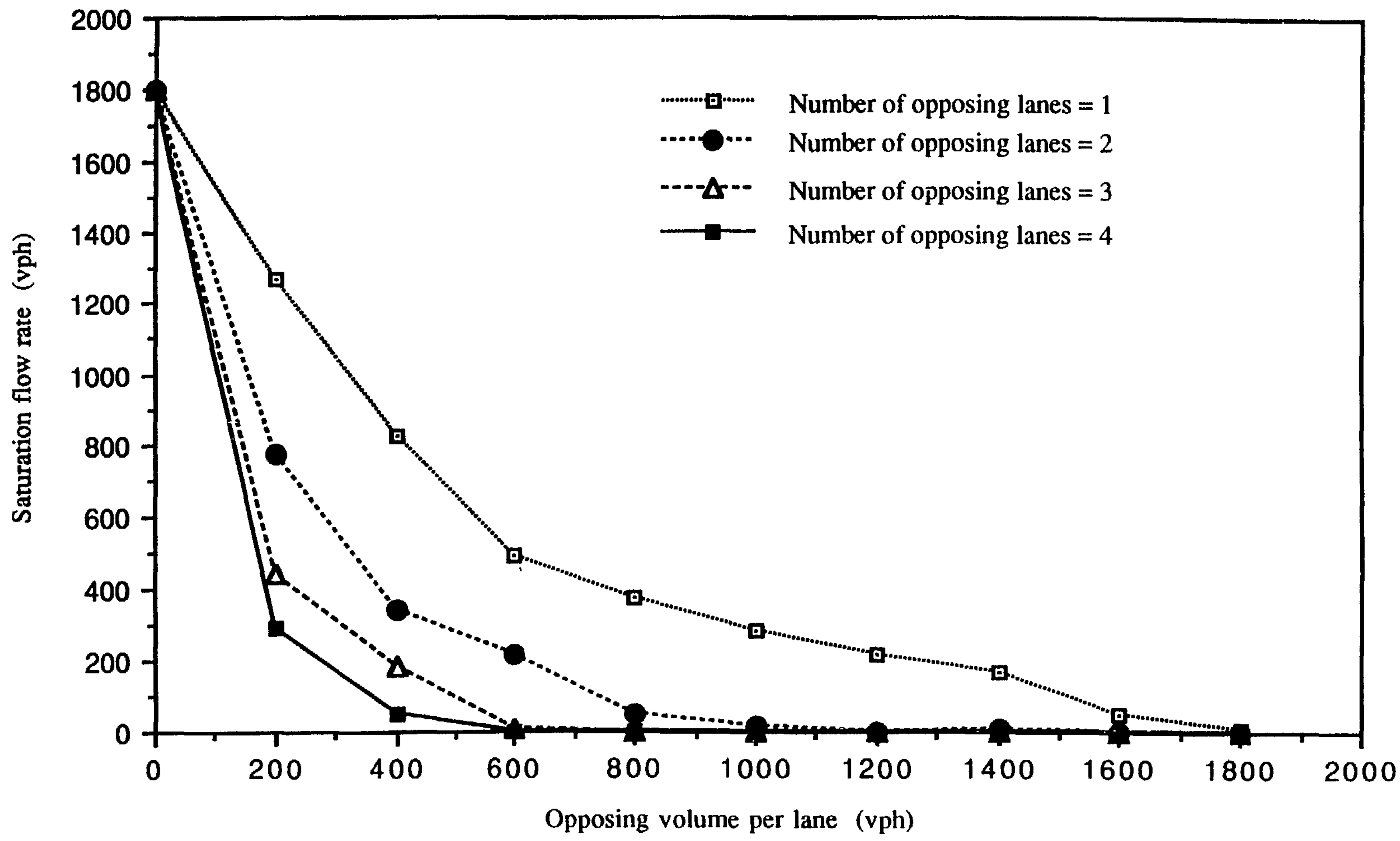
- The basic saturation flow rate for both through and left-turn movements.
- The left-turn processing time for heavy vehicles under various conditions for computing the equivalent factors.
- The average "lost time" under various LT phasings and geometric conditions
- The average queue length per cycle as a function of G/C ratio.

Note that the above variables or parameters by no means represent the only information to be collected in the field studies. It simply indicates that these variables or parameters will be viewed as given in the process of model development.

With a set of well calibrated parameters, the simulation program TRAF-NETSIM will be used to develop the following empirical relations:

- The relationships between the opposing volume and permitted saturation flow rate under a different number of opposing lanes (see an example in Figure 5.1).
- The distribution of opposing flow patterns under various signal control strategies.
- The relation between the average queue length and G/C ratio under various left-turn signal control strategies.

Figure 5.1 Left-Turn Saturation Flow rate vs. Opposing volume (per lane)



- Through-car equivalent for permitted left turns under various opposing volumes and lanes as reported by Messer and Fambro (1977).

- The effectiveness of using the adjusted opposing flow rate in estimating the saturation flow rate of permitted left-turns.

- An empirical relation between an individual exclusive left-turn lane and lane-group capacities (e.g., two exclusive left-turn lanes)

- The minimal left-turn bay length under a given left-turn capacity.

- The effect of actuated control on the left-turn capacity.

- The distribution of arriving flow patterns between the protected and permitted subphases in PT/PM phasing under various conditions.

Note that prior to the calibration of parameters in TRAF-NETSIM, extensive simulation experiments will be conducted to determine if the aforementioned relations can be represented with statistical models, charts, or tables. Such information will be very useful for the research team to best integrate empirical results into analytical formulations.

5.2 Some Relations to be Captured with Analytical Models

To facilitate the development of computerized procedures, the research team intends to maximize the use of analytical models to capture key relations associated with the left-turn capacity, but not at the cost of sacrificing accuracy. Some critical issues most suitable to be tackled with mathematical techniques are summarized below:

- The effect of the left-turn bay length, if it is less than the critical length, on the left-turn capacity.

- The effect of lane utilization ratio on the lane-group capacity.

- The interaction between the saturation flow rates in an exclusive left-turn lane and adjacent lanes.

- An analytical model to estimate the left-turn filtering rate under various opposing volumes and lanes.

- The effect of pedestrian flow rate on the saturation flow rate of an exclusive left-turn lane.

5.3 Some Issues to be Represented with Integrated Models

Due to the well recognized difficulty in capturing the complex relations between the left-turn capacity and all related factors, some recent studies have explored the integration of analytical methods with empirical models (e.g., Prassas and Roess, 1992). Such a model enables the researchers to take advantages of well accepted relations with empirical findings, and thus offers a promising research approach. The research team believes that the following research issues can actually be best tackled with integrated models:

- **A** prediction model for protected left-turn saturation flow and capacity.
- A prediction model for opposing queue discharging time.
- **A** prediction model for permitted left-turn saturation flow rate and capacity.
- A prediction model for estimating the effective green time, unsaturated green time for each subphase of a PT/PM or PM/PT phase.

Simulation-based Method: Note that the aforementioned studies as well as possible methods are based on the conventional capacity estimation concept, i.e., saturation flow approach. In review of literature, it is clear that the simulation-based method along with the "Transparency" concept is quite unique and seems to produce very promising results in Texas. This method is not only creative in a sense that it takes full advantage of simulation capabilities, but also very convenient and flexible for incorporation of all capacity related factors. However, there are two critical issues associated with this method to be overcome, prior to a full-scale development of essential models along this line:

- The assumption of a "constant" processing time for each individual left-turn vehicle under various opposing flows (during the unsaturated range) and lanes needs to be validated with both extensive simulation and empirical experiments; and
- The proposed methodology differs significantly from the conventional methods which mainly employ the saturation flow rate and adjustment factors. It may thus be difficult for traffic engineers and researchers to develop the required level of confidence, especially for those always having some doubts on the simulation results.

References

[1] Adams, C.J. & Humner, E.J. (1993). ***"The Effects of U-Turns on Left Turn Saturation Flow Rates,"*** presented at the Transportation Research Board 72nd Annual Meeting

[2] Akcelik, R. (1989). ***"Traffic Signal: Capacity and Timing Analysis,"*** 1981. Reprint. Research Report ARR No. 123. Nunawading, Australia: Australian Road Research Board, 1989.

[3] Akcelik, R. (1990). ***"User notes for the Highway Capacity Manual Option in Sidra (version*** 3.2)," Australian Road Research Board, May 1990

[4] Assmus, W.E. (1970). ***"Operational Performance of Exclusive Double Left Turn*** Lanes," M.S. thesis. Northwestern University, Evanston, Ill., 1970.

[5] Burrow, I.J. (1987). "OSCADY: ***A computer program to model capacities, queues and delays at isolated traffic signal junctions,"*** Research Report 105

[6] Bonneson, J.A., ***&*** P.T. ***McCoy.*** (1987). ***"Operational Analysis of Exclusive Left-Turn Lanes with Protected/Permitted Phasing,"*** Transportation Research Record 1114, pp 74-85, 1987

[7] Capelle, D.G. & Pinnell, C. (1961). ***"Capacity Study of Signalized Diamond Interchanges,"*** Bulletin 291, HRB, National Research Council, Washington, D.C., 1961, pp. l-25

[8] Fambro, B.D., Gaston, D.G. and Hoff, M.C. (1991). ***"Comparison of two Protected-Permitted Lead-Lag Left-Turn Phasing Arrangements,"*** Research Report 989-lF, Texas Transportation Institute

[9] Gipps, P.G. (1982). ***"Some Modified Gap Acceptance Formula,"*** Proceedings of the 11 th ARRB conference ll(4): 71-75

[10] Guell, D.L. (1987). ***"Left-Turn Adjustment Factor for Permitted Phasing,"*** Journal of Transportation Engineering Vol. 113, No. 6, Nov. 1987, pp 689-695

Hansson, A. & Bergh, T. (1988). ***"A New Swedish Capacity Manual/CAPCAL 2,"*** 14th ARRB conference

[11] Hoppe, L.M. & Krystek, T. (1988). ***"Some Factors Affecting Left-Turn Capacity and Delay at Unsignalized Intersections,"*** Intersections without Traffic Signals, Proceedings of an International Workshop, 16-18 March 1988, Bochum, West Germany

[12] Kell, James H. & Fullerton, Iris J. ***"Manual of traffic signal design,"*** ITE

[13] Kell, J.H. (1984). ***"Highway Capacity in North America - The New Approach,"*** Australian Road Research Board, Vol. 12, No. 1, 1984, pp 33-43

[14] Kimber, R.M., McDonald, M. and Hounsell, N.B. ***"The Predication of saturation flows for road junctions controlled by traffic signals,"*** Research Report 67

[15] Kikuchi, S., Chakroborty, P., and Vukadinovic, ***K.*** (1993). ***"Lengths of Left-Turn Lanes at Signalized Intersections,"*** presented at the Transportation research Board 72nd Annual Meeting

[16] Kunzman, W. (1978). ***"Another Look at Signalized Intersection Capacity,"*** ITE Journal, Vo1.48, 1978, pp. 12-15

[17] Lee, C.J., Wortman, H.R., and Poppe, (1991). ***"Comparative Analysis of Leading and Lagging Left-turns,"*** FHWA-AZ91-321, Arizona Department of Transportation

[18] Leisch, J.E. (1967). ***"Capacity Analysis Techniques for Design of Signalized Intersections,"*** Public Roads, Vol. 34, 1967, pp. 171-209

[19] Lin, Feng-Bor (199 1). ***"Left-Turn Signal Phasing for Full-Actuated Signal Control,"*** TRB, 70th Annual Meeting

[20] Lin, Han-Jei, Randy B. Machemehl, Clyde E. Lee and Robert Herman (1984). ***"Guidelines for use of left-turn lanes and signal phases,"*** Research report 258-l

[21] Lin, F. (1992). ***"Left-Turn Adjustment Factor for Saturation Flow Rates of Shared permissive Left-Turn Lanes,"*** Transportation Research Board National Research Washington DC, Jan. 1992, 10 pp, English

[22] Lee, J.C., Wortman, R.H., Hook, D.J.P., and Poppe, M.J. (1991). ***"Comparative Analysis of leading and Lagging Left Turns. Final Report,"*** Aug., 1991, 209p, Report No: FHWA-AZ-91-321

[23] Lin, H.J. & R.B. Machemehl (1983). ***"Developmental Study of Implementation Guidelinesfor Left-Turn Treatments,"*** TRB research Record 905, 1983, pp 96-115

[24] Lu, Y-J (1984). ***"Study of Left-Turn Maneuver Time for Signalized Intersections,"*** ITE Journal, Vol. 54, No. 10, Oct 1984, pp 42-47

[25] Lu, Y-J, Hsu, Y-H and Tan, G.C. (1988). ***"Application of the Image Analysis Technique to Detect Left-Turning Vehicles at Intersections"*** Transportation Research Record N1194, 1988, pp 120-128

[26] Mayers, Barry B. & Taylor, Gordon W. (1984). ***"The testing and development of a high capacity highway coach,"*** 9th Annual Conference proceedings, ITE, District 7, Canada

[27] Machemehl, R.B. (1986). ***"An Evaluation of Left-Turn Analysis Procedures,"*** ITE Journal, Vol. 56, No. 11, Nov. 1986, pp 37-41

[28] Machemehl, R.B. & Mechler, A.M. (1984). ***"Comparative Analysis of Left-Turn Phase Sequencing (Abridgment), "*** Transnortation Research Record N956, 1984, pp 37-40

[29] McCoy, Patrick T. & Navarro, Ulises R. (1991). ***"Position of Lefi-Turn Lanes,"*** Final Report, Research Report No: TRP-02-24-90

[30] Messer, C. & Fambro, D. (1977). ***"Critical Lane Analysis for Intersection Design,"*** Transnortation Research Record 644

[31] Michalopoulos P., J.O. Connor, and ***S.*** Novoa (1978). ***"Estimation of Left-Turn Saturation Flows,"*** TRB, 1978

[32] Mousa, R.M. & Rouphail, N.M. (1989). ***"Effect of Platoons on Permissive Lefi-Turn Capacity: Pilot Study,"*** Journal of Transportation Engineering, vol. 115, No.2, Mar. 1989, pp 208-215

[33] Moussavi, M., & Tarawneh, M. (1990). ***"Investigation of Saturation Flow Rates in Nebraska. Final Report,"*** June 1990, 150p, Report No: Proj RES1 (0099) p427

[34] Moussavi, M. & Tarawneh, M. (1990). ***"investigation of saturation flow rates in NEBRASKA. Final report,"*** Report No: Proj RES1(0099)p427, Jun 1990 150p

[35] Peterson, B.E., A. Hansson, and K.L. Bang. (1978). ***"Swedish Capacity Manaual,"*** Transportation Research Record 667, pp l-28, 1978

[36] Pitsiava-Latinopoulou, M., & Mustafa, MAS (1992). ***"The Accuracy of Estimating Delays at Signalized Intersection: A Comparison between Two Methods,"*** Traffic Engineering and Control VOL.33, No.5, May 1992, pp 306-3 11

[37] Pitsiava-Latinopoulou, M. & Mustafa, M.A.S. (1991). ***"A Comparison between the 1985 Highway Capacity Manual and Sidra for Signalized Intersection Analysis,"*** Traffic Engineering and Control, Vol. 32, No. 9, Sept., 1991, pp 406-411

[38] Prassas, S.E. & Roess, P.R. (1993). ***"The Left-Turn Adjustment for Permitted Turns from Shared Lane Groups: Another Look,"*** presented at the Transportation Research Board 72nd Annual Meeting

[39] Prevedouros, P.D. & Jovanis, P.P. (1988). ***"Validation of Saturation Flows and Progression Factors for Traffic-Actuated Signals,"*** Transportation Research Record N1194, 1988, pp 147-159

[40] Ray, J.C. (1965). ***"Two Lane Left-Turns Studied at Signalized Intersections,"*** Traffic Engineering, Vol. 35, 1965, pp. 17-19, 58.

[41] Rouphail, N., Magnuson, M., and Sisiopiku, V. (1991). ***"Validation of 1985 HCM Procedures for Capacity and Loss of Left Turn Lanes in Illinois. Final Report,"*** May 1991, 170p, Report No: FHWA/IL/RC-012

[42] Roess, R.P., Ulerio, J.M., and Papayannoulis, V.N. (1990). ***"Modeling the Left-Turn Adjustment Factor for Permitted Left Turns Made from Shared Lane Groups,"*** Transnortation Research Record, N1287, 1990, pp 138-150

[43] Roess, R.P., Papayannoulis, V.N., Ulerio, J.M., and Levinson, H.S. (1989). ***"Levels of Service*** in ***Shared Permissive Left-Turn Lane Groups at Signalized Intersections. Final Report,"*** Sept. 1989, 158p, Report No: FHWA-RD-89-228

[44] Roess, R.P. (1987). ***"Development of Analysis Procedures for Signalized Intersections in the 1985 Highway Capacity Manual,"*** Transportation Research Record 1112, pp 10-16, 1987

[45] Roess, R.P. & McShane, W.R. (1987). ***"Capacity and Level-of-Service Concepts in the Highway Capacity Manual,"*** ITE Journal, Vol. 57, No. 4, Apr 1987, pp 27-30

[46] Schorr, S.M. & P.P. Jovanis. (1984) ***"Accuracy of Capacity Models for Permissive Left Turns From an Exclusive Lane,"*** In Compendium of Technical Papers. ITE Journal, pp 71-76, 1984

[47] Stokes, R.W., Messer, C.J., and Stover, V.G. (1986). ***"Use and Effectiveness of Simple Linear Regression to Estimate Saturation Flows at Signalized Intersections,"*** Transportation Research Record, N1091, 1986, p 95-101

[48] Stokes, R.W., Messer, C.J., and Stover, V.G. (1986). ***"Saturation Flows of Exclusive Double Left-Turn Lanes,"*** Transnortation Research Record, N1091, 1986, p 86-95

[49] Takeshi Chishaki & Youichi Tamura (1984). ***"Headway distribution Model based on the distinction between leaders and followers,"*** Ninth International Symposium on Transportation and Traffic Theory. 1984 VNU Science Press, pp 43-63

[50] Tanner, J.C. (1967). ***"The capacity of an Uncontrolled Intersection,"*** Biometrika 54, 1967, p 657-658

[51] Teply, ***S.*** (1984). ***"Canadian Capacity Guide for Signalized Intersections: An overview,"*** 9th Annual Conference proceedings, ITE, District 7, Canada

[52] Teply, S. & A.M. Jones (1991). ***"Saturation Flow: Do we speak the same language?"*** TRB Research Record 1320, 1991, pp 144-153

[53] Wong, S.Y (1991). ***"Capacity and Level of Service by Simulation -- A Case Study of TRAF-NETSIM. Highway Capacity and Level of Service,"*** Proceedings of the International Symposium on Highway Capacity, 24-27 July, 1991, pp 467-483

[54] Wortman, R.H. (1987). ***"Capacity of Dual Left-Turn Lanes. State of the Art. Final Report,"*** Oct. 1987, 32p, Report No: FHWA-AZ87-829

[55] Institute of Traffic Engineers (1975). ***"The Use and Effectiveness of Double Left-Turn Movements,"*** Traffic Engineering, Vo1.45, 1975, pp.52-57

[56] ***"Guidelines for Signalized Left Turn Treatments,"*** Report No: FHWA-TP-81-4, 1981

[57] ***"A study of clearance intervals, flashing operations, and left-turn phasing at traffic signals,"*** Final Report, May 1980. Report No. FHWA-RD-78-48

[58] ***"Japanese Highway Capacity Manual,"*** (1978) translated by Taiwan Institute of Transportation

[59] ***"Swedish Highway Capacity Manual,"*** (1977) National Swedish Road Administration

[60] ***"Traffic Flow, Capacity, and Measurements,"*** Transportation Research Record N905, 1983, 174p

[61] ***"A Program of Research in Highway Capacity,"*** ***TRB*** Circular 371, 1991