Empirical Investigation of Intersection Traffic Patterns Under Maryland Driving Populations

--- Analysis of the Critical-Lane Volume Method at Signalized Intersections---

(Phase-I: Final Report)

by

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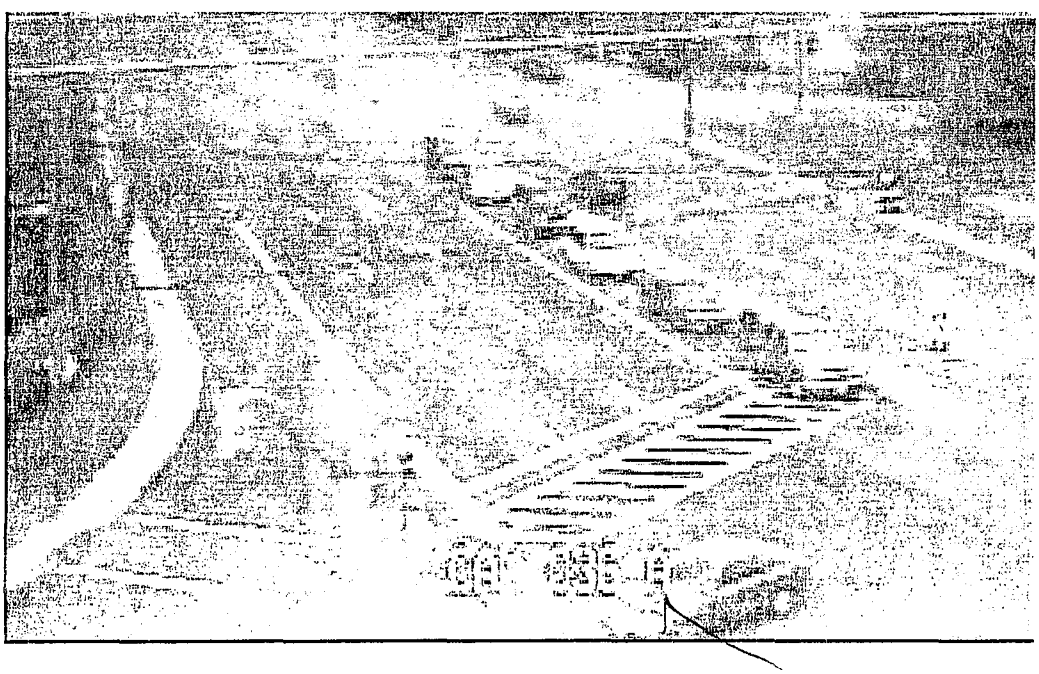

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Executive Summary

. Background

As is well recognized in the transportation community, key parameters for saturation flow and capacity estimation in both the Critical-Lane Volume method (CLV) method and the operational analysis approach in the Highway Capacity Manual (HCM) are based on national averages of limited sample observations. Consequently, *the level of service of an intersection*, computed with either the CLV or HCM methods using default parameters, may not truthfully reflect the actual traffic condition. This is due understandably to the variation of driving populations and their behavior discrepancies between and within states.

A brief review of the literature has also revealed that most studies (e. g., Stokes, 1988; Ruehr, 1988) recommend that some field observations be performed at each local jurisdiction so as to capture its traffic behavior with respect to key characteristics such as discharging headways and saturation flow rate. The Highway Capacity Manual (HCM) has also urged States and local municipalities to conduct field validation of its default saturation flow rate.

However, this essential task of observing traffic conditions and updating key traffic characteristics parameters seems to have been long over due in the State of Maryland. Some key parameters such as saturation flow rate or discharging headways have not been collected and calibrated over the past several years. Thus, county traffic planners and SHA staffs tend to employ their preferred parameter values in the analyses, and very often head to significantly different conclusions.

This research is proposed in response to such discrepancies, and to the well-recognized need for each state to perform its own field observations and key traffic parameter calibration. It is expected that the calibrated local traffic parameters, such as saturation flow rate, may offer the common ground for each local traffic agency in its traffic impact analysis and design of effective strategies.

. Research Scope and Procedures

This study has focused primarily on empirical and simulation analyses of driving characteristics at signalized local intersections with emphases on their discharging headways, start-up delay, and saturation flow rates. The research procedures have been divided into the following sequence of stages:

- Using field observation data from 11 representative intersections (selected by SHA from different counties) to compute the key traffic characteristics such as discharging headways, start-up delays, saturation headways, and the average time loss per cycle at each selected intersection;

- Computing the ideal saturation flow rate at each sampled intersection based on the estimated average headway;

- Conducting an additional set of field observations to identify key factors, such as loss time during signal phase transition and truck percentage in the traffic stream, that may contribute to the reduction of the MCLV;

- Estimate the MCLV based on the observed average saturation flow rate, average loss time per cycle, and a given truck percentage in the traffic stream;

- Compare the proposed MCLV with the critical lane volume collected from a very congested intersection that is sure to be close or over its capacity; and

- Modeling each intersection included in the field observations with a commonly used simulation program, CORSIM, and computing its maximum critical lane volume by incorporating observed key traffic characteristics (e. g., startup delay, discharging and average headways) and an artificially increased volume in each intersection approach

. Research Findings and Recommendations

Through the above investigation process, both the analytical and simulation methods have consistently yielded the following findings:

- The MCLV of 1600vph, proposed by the original CLV document in more than three decades ago, is below the actual critical lane capacity of local signalized intersections; and

- Although the actual MCLV for a local intersection may vary with a variety of factors such as geometry and driving characteristics, it most likely lies between 1700 vph and 1800 vph.

Thus, with respect to the most appropriate MCLV for statewide traffic impact analysis, it is recommended that:

- the value of *1800 vph* be used as the MCLV for intersections that have major arterials on all four approaches or are known to have aggressive driving patterns;

- the value of 1700 vph be used as the MCLV for intersections having mainly minor roads and/or community roads;

- the average value of 1750 vph be adopted as the MCLV for local intersections that have both major and minor roads in their approaches, or are at the planning stage having only limited traffic as well as geometric related information available.

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Chapter 1: Introduction

1.1 Background

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A brief review of the literature has also revealed that most studies (e. g., Stokes, 1988; Ruehr, 1988) recommend that some field observations be performed at each local jurisdiction so as to capture its traffic behavior with respect to key characteristics such as discharging headways and saturation flow rate. The Highway Capacity Manual (HCM) has also urged States and local municipalities to conduct field validation of its default saturation flow rate.

However, this essential task of observing traffic conditions and updating key traffic characteristics parameters seems to have been long over due in the State of Maryland. Some key parameters such as saturation flow rate or discharging headways have not been collected and calibrated over the past several years. Thus, county traffic planners and SHA staffs tend to employ their preferred parameter values in the analyses, and very often head to significantly different conclusions.

For instance, it is likely that the traffic condition at an intersection may have been ranked as at the "*Level of Service E*" by the county engineers, but classified by SHA staffs as at the "*Level of Service D*". Thus, whether or not to allocate budget for improving such intersections has often emerged as a difficult issue among transportation professionals in both state and counties.

This research is proposed in response to such discrepancies, and to the well-recognized need for each state to perform its own field observations and key traffic parameter calibration. It is expected that the calibrated local traffic parameters, such as saturation flow rate, may offer the common ground for each local traffic agency in its traffic impact analysis and design of effective strategies.

1.2 Research Objectives:

This study is aimed to observe the behavior of Maryland driving population at signalized intersections, and to calibrate key traffic characteristics parameters for use in the CLV method by SHA and county engineers. The empirical results are expected to better capture the interrelations between the driving behavior of local population and their resulting impacts on traffic congestion. This study also intends to produce a set of parameters that truthfully reflect the local traffic patterns, and offers the common ground for SHA and county engineers in performing their traffic impact analysis. More specifically, the primary objectives of this study are to:

- empirically investigate the driving behavior of various local populations at signalized intersections;

- rigorously characterize traffic flow parameters with respect to discharging headways, saturation flow rate, and intersection critical lane volume based on empirical data observed at local intersections; and

- recommend key traffic parameter values in the CLV method for use by SHA, county and local engineers in their traffic impact analyses.

1.3: Report Organization

This report is organized as follows: A brief description of the entire research methodology along with principal tasks is presented in the next chapter. This is followed by a detailed presentation of the field data collection and analysis procedures, including statistical test results and computation of discharging headways as well as saturation flow rate in Chapter 3. The application of computed empirical results for intersection capacity estimation and critical lane volume approximation constitute the core of Chapter 4.

Also included in Chapter 4 are the results of empirical and simulation validations with respect to the proposed critical lane volume for the CLV. Concluding comments and recommendations regarding the use of the CLV method for traffic impact assessment are reported in the last chapter.

Chapter 2: Research Methodology and Procedures

2.1: Description of research procedures

As the main focus of this study is to derive key CLV parameters to realistically reflect the driving behavior of local populations and their impacts on traffic conditions, it is apparent that all analyses and calibration ought to ground on the data from field observations. Thus, our research procedures, as shown in Figure 2-1, start with a comprehensive field study of traffic flow patterns at representative local intersections, and then followed by a sequence of statistical analysis and simulation evaluations. The entire study, after review of available literature, has been divided into the following principal tasks:

- Field observations of intersection discharging headways, start-up delays, and saturation flow rate;
- Data filtering and quality assessment with respect to field observations at 11 representative local intersections (selected by SHA from different counties);
- Data analysis and computation of average headways as well as the saturation flow rate at each field site;
- Estimation of the critical lane volume for each intersection from saturation flow rate, start-up loss time, and all associated factors;
- Field observations and to validate the proposed intersection critical lane volume;
- Simulation analysis of the critical lane volume at each of those 11 observed local intersections;
- Recommendation of the maximum critical lane volume for use in the CLV method.

The logic underlying each of the above principal tasks along with its key steps are presented below:

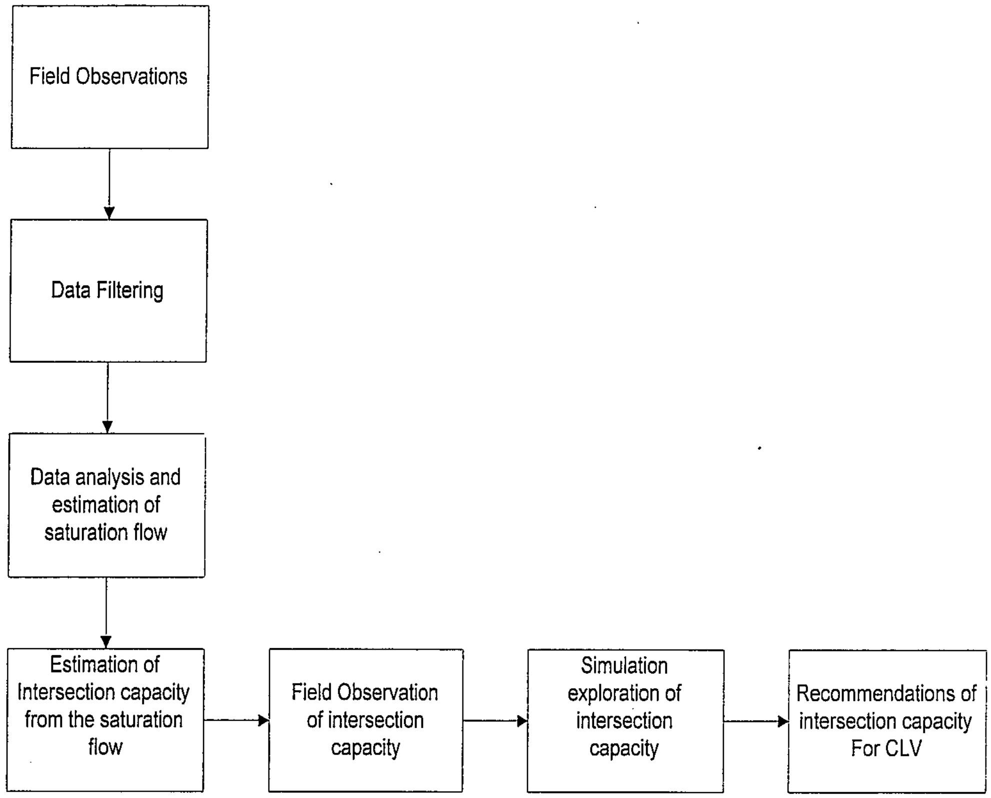

Figure 2. 1 Flowchart for the method used to estimate the Maximum Critical Lane Volume for signalized intersections

2.2: Field observations

Regardless of the specific procedure used in the intersection capacity analysis, saturation flow rate is always used as a base flow rate. Various capacity related factors are then used to modify this base flow to reflect prevailing traffic conditions at each target site The saturation flow can be defined as the maximum flow that can occur during the "go" period of a signal cycle. Notably, such a flow rate depends mainly on how drivers react to signals and follow each other under the given traffic environment. Thus, it is essential for a study of local intersection saturation flow rate and critical lane volume to start from direct field observations of driving characteristics.

In this task, eleven of the 22 intersections suggested by the State Highway Administration have been surveyed to obtain the average of their headways under prevailing traffic conditions. Those field observations have been focused on measuring the time at which a vehicle crosses the stop line when the signal is green for that movement. These times are then used to compute the headway between the vehicles.

To facilitate the data analysis and ensure the data quality, two camcorders were placed at each intersections to monitor the movement of traffic over the stop lines in all directions. These recorded tapes were reviewed later and the series of times at which a vehicle crosses the stop line was recorded for computation of the headway, start-up delay, and critical lane volume.

2.3: Data filtering and quality control

As is well recognized, a variety of potential data errors may occur in field observations, especially those tasks involving a large amount of data recording, counting, and input. Also, some observed data, though recorded accurately, might not be good samples for use in computing the average as those may represent some extreme values of the target subject.

For instance, it has been observed in our field data that some intersections also had a few drivers who were overly aggressive and a few were overly conservative. Those drivers do not represent the average driving populations under consideration, especially in computing the resulting saturation flow and critical lane volume. Thus, the following steps have been taken to ensure the quality of available data:

- Discarding all those observations where vehicles were not in platoons and moved independently from the traffic flow;

- Computing the mean and standard deviation of observed headways at each intersection;

- Using the mean and standard deviation to construct the standard quality control thresholds; and

- Performing the data analysis from those observations falling within the quality control thresholds.

2.4: Data analysis and estimation of saturation flow rate

Note that theoretically one may measure the maximum number of vehicles passing the intersection per lane per hour as the saturation flow rate. But it is practically difficult to do so as most intersections may not be under their saturation state during the entire observation period. The alternative is thus to compute the average headway and their distribution of vehicles within a platoon or in a continuous and compacted traffic stream.

Since the discharging headways of vehicles from each green phase that contain the start-up delay are not a representative of typical headways under prevailing traffic conditions, one has to compute the number of vehicles in the queue at each cycle and estimate their start-up delays from the video record. This enables us to have a better idea regarding local drivers' response to signal phase changes, and to distinguish the discharging headways from the saturation flow headways.

2.5: Estimation of the maximum intersection critical lane volume

Given the saturation flow rate and startup delays, this step is to compute the maximum critical lane volume for typical local intersections. It should be noted that the critical lane volume, correlated directly with the intersection capacity, may vary with a variety of factors associated with traffic characteristics, geometric conditions, and signal design. Among those, the all red time and the percentage of trucks have the most significant impact on the intersection capacity and therefore the maximum critical lane volume in the CLV applications.

This due to the fact that the all red time duration will reduce the total time available for passing of vehicles thus reducing the capacity of the intersection. Similarly, trucks often move slowly and usually maintain relatively long headways in the traffic stream which in turn result in a reduction of the intersection capacity. Thus, to propose an average value for the maximum critical lane volume for local intersections, it is essential to estimate the average time loss per cycle at typical local intersections, and the impacts under various truck percentages.

Note that while the formal factor was observed from field observations, the impact of the latter factor on the maximum critical lane volume and capacity was conducted through extensive simulation experiments, as field observation data may not contain a wide range of variation in truck percentage.

2.6: Field observations for the maximum critical lane volume (MCLV)

Following the computation of an average MCLV for typical local intersections, it is essential for the research team to investigate if any of existing intersections may exhibit a critical lane volume higher than the proposed MCLV. Thus, the following two commonly used approach were taken to assess the applicability of the proposed MCLV:

- *Field observations at saturated or over saturated intersections:*

The intersection between US 1 and Cherry Hill Road has been chosen to find out the approximate MCLV. The traffic counts cover all lanes in each approach at an interval of 5 minutes.

- *Simulation evaluations for all 11 intersections recorded in the field data*

To assess if the computed MCLV can be applied to all local intersections, the research team has further performed a series of simulation analyses to find out the approximate MCLV at each of those 11 intersections. The simulation investigation with respect to each intersection was based on the discharging headways, signal plans, and startup delay data collected from field observations. All those eleven intersections that were surveyed to compute the saturation flow are simulated with CORSIM. The capacity and MCLV of each of those intersections are computed from the volume data counted with detectors placed during the simulation

2.7: Recommendations for the CLV application

Upon the completion of both the empirical and simulation analyses, we have made some preliminary recommendations regarding the most reasonable value for MCLV at local intersections and presented to SHA staffs. Further revisions have also been conducted according the comments and suggestions from several technical discussions between the research team and SHA staffs.

Chapter 3: Field observations of intersection saturation flow

3.1: Introduction:

This chapter details the field observation procedures and data analysis process with respect to those 11 signalized intersections selected by MSHA for empirical investigation of the maximum critical lane volume (CLV). The data filtering and statistical methods used are also described in this chapter, along with a recommendation of the intersection saturation flow rate that reflects the behavior of Maryland driving populations.

3.2: Description of field data collection procedures

Based on the suggestions from MSHA staff, the field observations were conducted at 11 eleven sites selected from the preliminary list of 22 intersections. To ensure a representative coverage of Maryland driving populations, all those 11 selected intersections are sampled from different counties. The list of the intersections surveyed is shown in Table 3-1.

	Intersection
1	US 1 / MD 212
2	MD 97 / MD 192
3	MD 648 / MD 177
4	MD 216 / All Saint's Road
5	US 301 / MD 197
6	US 40 / Rolling Road
7	MD 85 / Crestwood Village Blvd
8	MD 152 / US 1
9	US 50 / MD 213
10	US 40 / MD 272
11	US 50 / Mill street

Table 3. 1: List of intersections for which data was collected

The data collection is focused on the headway between vehicles when they leave the target intersection approach. The startup delay at the beginning of each green phase has also been observed and computed.

To facilitate the data recording and analysis, all field observations were performed with camcorders that record the movement of traffic streams at the intersection. As shown in Figure 3-1, the field observation study at each site has employed two camcorders, each placed parallel to the traffic movement that is being monitored. The two camcorders were placed diagonally opposite to each other. This is to ensure that one can not only see the stop line of the movement that has the highest volume, but also record vehicles crossing the stop line of the link perpendicular to the line of sight. With such a field configuration, one can also count the number of vehicles that cross the stop line.

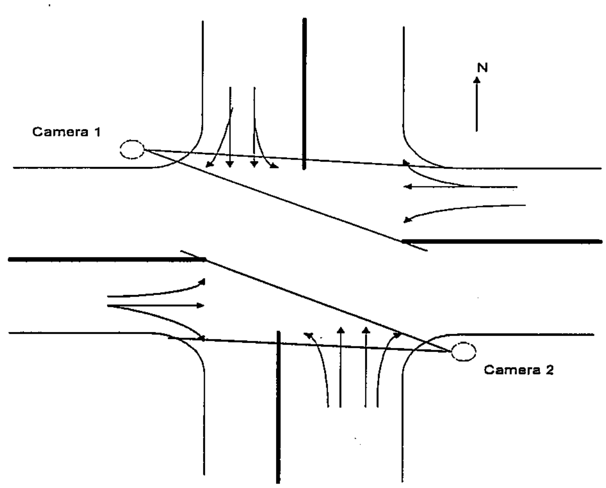

Figure 3. 1 A graphical illustration of the position of the cameras for the data collection.

3.3: Data Acquisition and field observation results

The field traffic stream data collected with those camcorders were first transferred to the normal VHS tapes for future references. The most popularly used computer language; C++ was selected to develop the data reduction and analysis program. This is to take advantage of the strengths of C++ as it has predefined functions to determine that start time and end time of a particular event.

An event can be defined as a click of the mouse or the enter key or the space bar. For recording the series of observed headways, a program was written to ensure that each time the spacebar or enter button is pressed the time would be stored in the memory. This array will show the time at which a vehicle passed the stop line. Also, the index of the array will indicate the total number of vehicles being counted. This array can be used as an input to another program that was designed to categorize all observed headways into saturate and undersaturated conditions.

To remove some field data that may reflect some extreme types of drivers or contain some input errors, the following three steps have been taken in computation of the saturation flow rate:

Step-1: Removing all extreme data based on the quality control principle

In this step, the average and standard deviation of headways across all cycles for each vehicle were first determined. This is then followed by the removal of all data points that do not lie in the range $[\mu-\sigma,\mu+\sigma]$. The main purpose is to makes sure that very large headways or very small headways that are mostly caused by human errors during computation or by very aggressive drivers will not cause any significant bias in the final results.

This elimination process also removes the headway values that represent the time interval between two consecutive platoons, as traffic flow under such conditions certainly has not reached its saturation level yet. To calculate the saturation headway, only the headways between the vehicles in a platoon or compacted traffic stream ought to be considered.

Step-2: Computing the startup delay:

This step computes the start up delay based on the video record results as it only incur among those vehicles arriving during the red signal phase. To ensure that the collected headways represent those vehicles under a saturated state, a comparison between successive headways has also been made in the automated data filtering process

Note that after the filtering process only those consecutive headways over 1.8 seconds were included in the data set for further analysis and computation of start up delays. The threshold of 1.8 seconds was estimated based on the observation results from recorded tapes at different intersections, as it was noticed that the start-up delays are always above 1.8s. One can also view the tapes to verify which of those vehicles actually experienced the start-up delay.

Step-3: Saturation flow calculation:

The final step consists of finding the saturation flow at each observed intersection for that target lane. Prior to computation of the average for observed saturation flow headways, a similar data quality check as with the start up delay was also performed. Basically, each data point is compared with the next subsequent data point in the observed time series to see if the difference between these two data points reflects the existence of a gap between platoons. Thus, only those valid data points were include in the final set for computing the average of the saturation flow rate for that cycle.

Note that for each intersection the analysis was focused on its through movement as it generally has a large volume to yield sufficient sample observations. In the case that the left turns have large platoons, the same analysis was conducted for the left turns as well. At the same time the number of vehicles that cross the stopline in on hour on that target lane was recorded in the database.

The procedures for analyzing startup delays and saturation flow headways are divided into the following three parts as shown in Figure 3.2:

. The startup delays;
. The saturation headways without considering the startup delay; and
. The saturation headways taking the startup delay into account.

The results of this analysis are shown in Tables 3-2 and 3-3.

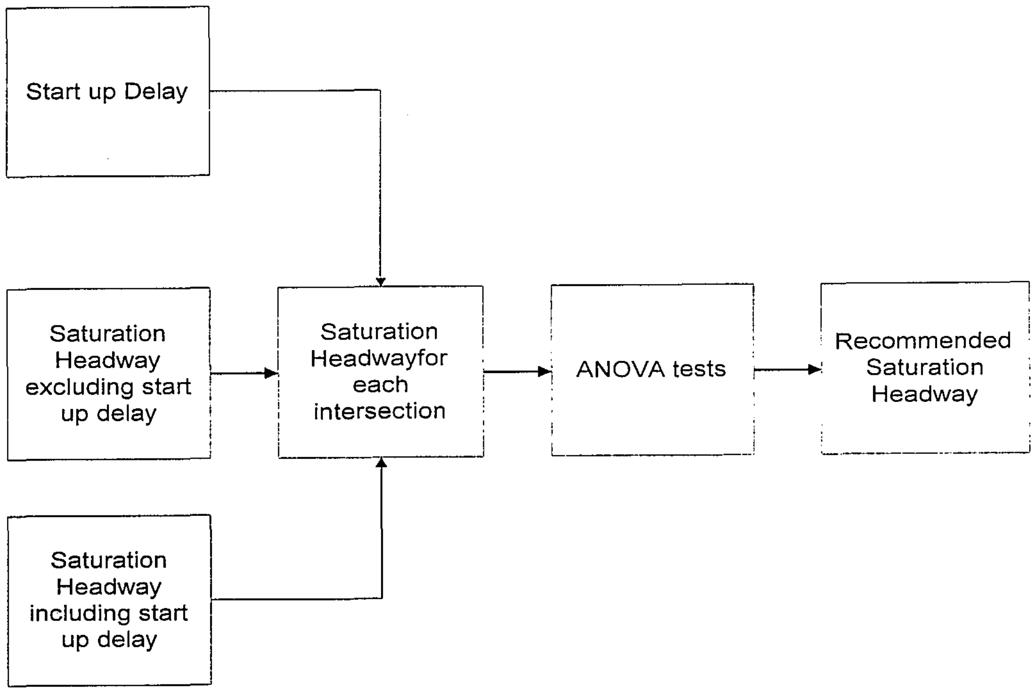

Figure 3. 2 A graphical illustration of the computation Process for saturation flow rate

	Intersection	Lane Volume (veh/hr)	Start up delay (s) (SD, Count)	Average excluding startup vehicles (s) (SD, Count)	Average including startup vehicles (s) (SD, Count)
US 1 / MD 212	US 1 NB through	502	2.28 (0.084,89)	1.79 (0.211,198)	1.94 (0.152,287)
	US 1 SB through	423	2.42 (0.045,56)	1.74 (0.234,117)	1.96 (0.2,174)
MD 97 / MD 192	MD 97 NB Through	800	2.36 (0.15,59)	1.76 (0.2,127)	1.89 (0.235,195)
	MD 97 SB Through	992	2.46 (0.05,37)	1.74 (0.18,174)	1.8 (0.182,195)
MD 648 / MD 177	MD 177 EB Through	645	2.1 (0.141,65)	1.76 (0.18,197)	1.86 0.1,262)
MD 216 / All saints Rd	MD 216 EB left turn	745	2.07 (0.207,129)	1.68 (0.118,166)	1.84 (0.162,276)
	MD 216 EB Through	577	2.14 (0.081,77)	1.60 (0.221,200)	1.77 (0.241,288)
US 301 / MD 197	US 301 NB Through	730	2.33 (0.103,63)	1.73 (0.267,118)	1.95 (0.233,181)
	US 301 SB Through	700	2.26 (0.089,60)	1.63 (0.235,170)	1.77 (0.241,233)
US 40 / Rolling Road	US 40 EB through	349	2.45 (0.07,25)	1.72 (0.168,180)	1.82 (0.16,205)
	US 40 WB through	336	2.4 (0.08,49)	1.75 (0.186,158)	1.89 (0.23,209)

Table 3. 2: Summary of analysis results

	Intersection	Lane Volume (veh/hr)	Start up delay (s) (SD, Count)	Average excluding startup vehicles (s) (SD, Count)	Average including startup vehicles (s) (SD, Count)
MD 85 / Crest Vlg Blvd	MD 85 SB Through	884	2.2 (0.04,91)	1.70 (0.09,240)	1.87 (0.18,331)
US 1 / MD 152	MD 152 EB Through	253	2.133 (0.05,43)	1.74 (0.126,87)	1.89 (0.153,130)
US 50 / MD 213	US 50 EB Through	817	2.2 (0.1,16)	1.8 (0.17,62)	1.87 (0.169,78)
	US 50 WB Through	782	2.25 (0.173,21)	1.79 (0.094,89)	1.88 (0.11,107)
US 40 / MD 272	MD 272 NB Through	527	2.25 (0.058,39)	1.86 (0.106,140)	1.95 (0.117,179)
	US 40 WB Through	541	2.35 (0.17,51)	1.91 (0.127,141)	2.04 (0.17,192)
US 50 / Mill Street	US 50 WB Through	613	2.1 (0.1,34)	1.7 (0.116,126)	1.78 (0.126,160)

SD = Standard Deviation
Count = Total number of observations

Table 3. 3 Summary of results (continued)

3.4: ANOVA tests

After the results were tabulated, the single-factor ANOVA test was performed to determine whether the average of the headways are statistically equal or whether we should use different saturation headways for each intersection. The hypothesis to be tested is presented below:

H_o: Are all the means statistically equal i.e. is $\mu_1=\mu_2=\mu_3=\ldots=\mu_{11}$. The rejection range is given by the equation:

$$f \geq F_{\alpha,I-1,n-I}$$

	Start up delay	Saturation headway without start up vehicles	Saturation headway with start up vehicles
f	0.213	0.044	0.043
$F_{\alpha,I-1,n-I}$	1.88	1.67	1.67
Grand Mean	2.2	1.73	1.88

Table 3. 4 ANOVA test results.

Table 3-4 shows the results of the ANOVA tests. It clearly indicates that one ought not to employ different average saturation headways for different intersections in the CLV application. Instead, the use of a common value for the saturation flow rate, and subsequently the MCLV, is a statistically valid alternative.

Note that for both the comparison and sensitivity analyses, we have further divided all computed saturation headways into two categories based on their grand average value. Table 3-5 shows all the movements with their saturation headway (saturation headway including the startup delay) above the grand mean of 1.88s, whereas Table 3-6 presents those intersections with their means below the grand mean.

Intersection	Movement	Mean (s)
US 1 / MD 212	US 1 NB through	1.94
US 1 / MD 212	US 1 SB through	1.96
MD 97 / MD 192	MD 97 NB Through	1.89
US 301 / MD 197	US 301 NB Through	1.95
US 40 / Rolling Road	US 40 WB through	1.89
US 1 /MD 152	MD 152 EB Through	1.89
US 50 /MD 213	US 50WB Through	1.88
US 40 /MD 272	MD 272 NB Through	1.95
US 40 /MD 272	US 40 WB Through	2.04
Mean		**1.93**

Table 3. 5 Intersections with saturation flow higher than the grand mean.

	Movement	Mean (s)
MD 97 / MD 192	MD 97 SB Through	1.8
MD 648 / MD 177	MD 177 EB Through	1.86
MD 216 / All saints Rd	MD 216 EB left turn	1.84
MD 216 / All saints Rd	MD 216 EB Through	1.77
US 301 / MD 197	US 301 SB Through	1.77
US 40 / Rolling Road	US 40 EB through	1.82
MD 85 /Crestwood Vlg Blvd	MD 85 SB Through	1.87
US 50 /MD 213	US 50 EB Through	1.87
US 50 /Mill Street	US 50 WB Through	1.78
Mean		**1.82**

Table 3. 6: Intersections with saturation flow lower than the grand mean.

Notably, with results form the above two tables, one can calculate the ideal saturation flow value for each category of intersections under the assumption that all factors contributing to time loss are negligible.

The saturation flow value from Table 3.5 is

$$s_1 = \frac{3600}{1.93} \approx 1850 \; vphpl$$

And the saturation flow value from Table 3.6 is

$$s_2 = \frac{3600}{1.82} \approx 1950 \; vphpl$$

Using these saturation flow values, the MCLV at those signalized intersections can be calculated. A detailed description of the computation procedures for the MCLV is discussed in the next chapter.

Chapter 4: Estimation of the Maximum Critical Lane Volume for Signalized Intersections

4.1: Introduction:

This chapter describes the method and procedures used to calculate the critical lane capacity of signalized intersections. As shown in Figure 4-1, the computation starts with the estimation of the ideal saturation flow rate from field observations, and then proceeds to the analysis of loss time due to transitions of signal phases in each cycle. This is followed by an extensive simulation investigation of MCLV under various geometric conditions, signal design plans, and truck percentages in the traffic stream. A detailed description of each step in Figure 4-1 is presented below:

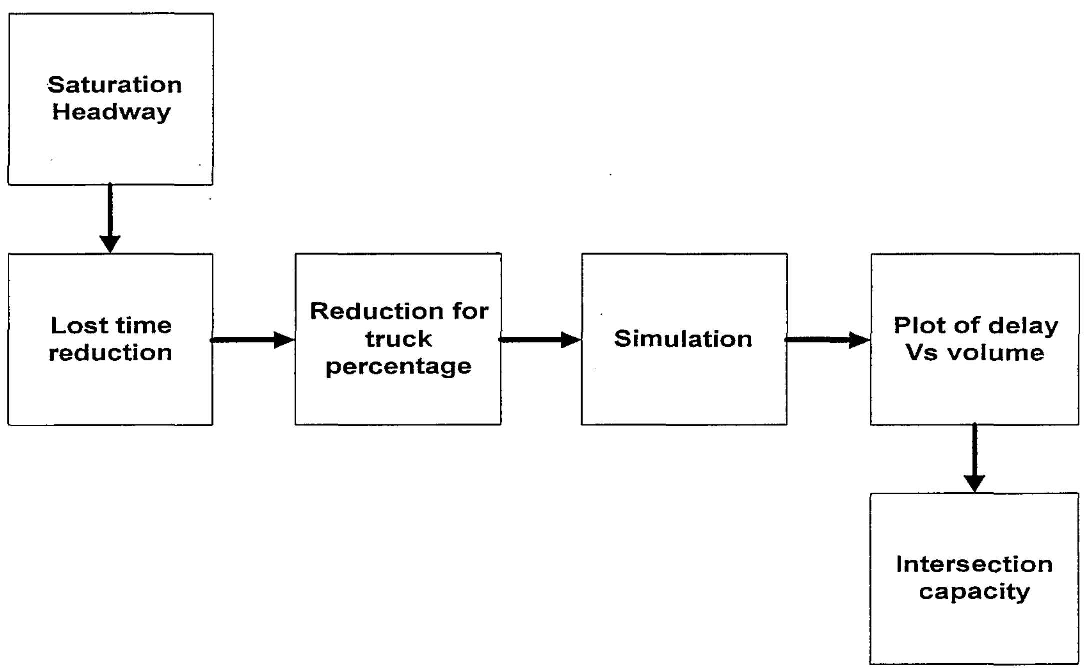

Figure 4. 1 Procedures for computing the maximum critical lane volume at signalized intersections

- Estimation of the average lost time per cycle

It is notable that the saturation headway gives the saturation flow rate under the ideal condition, which is the maximum number of vehicles that can cross the stop line given a constant green. The MCLV, or critical lane capacity of an intersection, on the other hand depends on a variety of other factors. One of such critical factors is the lost time or the clearance time during the signal phase transition.

For instance, during an all-red phase, no movement is allowed though the intersection. Also, drivers may not fully use all provided amber duration either. All such factors will certainly reduce the number of vehicles that may cross the intersection, and hence affect the MCLV or critical lane capacity of the intersection. The average all-red time and amber duration typically used at local intersections were computed form additional field studies and described in Section 4.2

- The MCLV Reduction under various truck percentages

As mentioned above, the critical lane capacity of an intersection is affected by several factors. Aside from the all-red time, the percentage of trucks at the intersection is also one of those critical factors. Conceivably, an increase in the truck percentage will reduce the capacity, as they tend to move in relatively slow pace at intersections, causing a long headway in the traffic flow and consequently the time taken to cross the intersection.

- Simulation analysis

To circumvent the difficulty in collecting traffic data that can contain a wide range of truck percentage, this study has employed the simulation approach to investigate the interrelations between truck percentage and the resulting MCLV at signalized intersections.

In all simulation experiments, truck percentages ranging from 0 to 15 percent are used as input to the simulation model, and the results are shown in Section 4.4. In addition, an average value of 5% is used as the standard input in simulating all those eleven intersections that were surveyed in the field data collection. The simulation results are presented in Section 4.5

- Computing the MCLV or the critical lane capacity

Note that the MCLV can be approximated directly from the average saturation flow rate computed from field observations, and adjusted with the reductions due to potential loss time per cycle and the presence of truck volume.

4.2: Lost time and the resulting critical lane capacity:

To determine the average all-red duration at local intersections, additional field data were collected at the following ten intersections shown in Table 4.1.

	INTERSECTION
1	US 1 / MD 410
2	MD 410 / MD 212
3	MD 410 /MD 650
4	US 1 / MD 212
5	US 1 / MD 198
6	MD 198 / US 29
7	US 29 / Cherry Hill Road
8	US 29 / MD 193
9	MD 193 / MD 212
10	MD 193 / MD 650

Table 4. 1: Intersections surveyed for lost time calculations.

The average lost time from these observations was found to be 1.5 seconds. This average value is used to estimate the reduction in the MCLV of the intersection. The average cycle length of those 10 intersections was found to be about 150 seconds. Given the average cycle length of 150 seconds, each intersection will have 24 cycles over one hour.

Apart from the all red time, there is some time lost in the yellow phase, as drivers may not use some portion of the provided amber phase. The average yellow time per cycle at those 10 intersections is about 3.6 seconds.

Note that the average start up delay is about 2.2 seconds as reported in the previous chapter for saturation flow estimation. Assuming that most signalized intersections, on average, have 4 phases in every cycle, then the time loss due to startup delay amounts to 8.8 seconds. Thus, the total time lost per cycle can be approximated as follows:

The total time lost per cycle = the all red time +
half of the average yellow time +
the startup delay

$$= 1.5 + 0.5*3.6+8.8 = 12.1 \text{ seconds}$$

Hence, the MCLV of an intersection using 1.82 seconds as the saturation headway will be closed o 1800 vph [note: *(3600 – 12.1*24)/1.82 = 1819 vph*]

In contrast, Using 1.93 seconds as the average saturation headway for intersections having less aggressive drivers, the MCLV will be around 1700 vph [note: *(3600 – 12.1*24)/1.93 = 1715 vph*]

To check the validity of the results computed above, it was decided that an independent field data observation at a saturated intersection would be the most appropriate in this case. The site chosen for the partial validation was the intersection between US-1and Cherry Hill Road. The reason for choosing this intersection was that it is always very congested during its peak period in all approaches. Thus, it is likely that the intersection may have reached its capacity level.

4.3: Field data collection at US1/ Cherry Hill intersection

Traffic count data was collected at the intersection US 1 / Cherry Hill road, including the volume data for all intersection legs. The collection was done at 5-minute intervals and the data recorded are shown in Table 4-2.

Interval	US 1SB thru Vol / 5 min interval	US 1NB thru Vol / 5 min interval	US 1NB left Vol / 5 min interval	Cherry Hill EB right Vol / 5 min interval	Cherry Hill EB left Vol / 5 min interval
1	111	137	47	45	36
2	99	117	39	43	39
3	137	114	32	59	31
4	158	91	37	47	39
5	98	122	40	54	47
6	96	135	48	47	24
7	105	140	44	42	48
8	121	133	36	58	38
9	127	113	31	55	37
10	119	128	38	49	40
11	157	135	39	51	41
12	142	141	34	52	42
13	152	141	35	55	35
14	148	129	43	49	46
15	142	137	41	44	35
16	156	142	47	45	37
17	173	121	31	51	36
18	141	119	34	44	35
Max 5 min volume	173	142	48	59	48
Max possible Volume per hour	2076	1704	576	708	576
No of lanes	3	2	1	2	2
Volume per lane per hour taking the maximum 5 min volume	2076/3 = 692	1704/2 = 852	576/1 = 576	708/2 = 354	576/2 = 288

Table 4. 2: Data collected at the US 1 / Cherry Hill intersection

Of these data collected, the maximum value of the volume is found and used to determine the maximum volume per hour for that lane. This volume is used in calculating the MCLV of the intersection. A graphical illustration of the intersection with the maximum possible volumes is shown in Figure 4-2. It was observed that though the green time allocated to the lanes for the US 1 north bound approach were utilized to their maximum, the southbound approach was less congested. Hence the volume for the southbound approach was not close to its capacity.

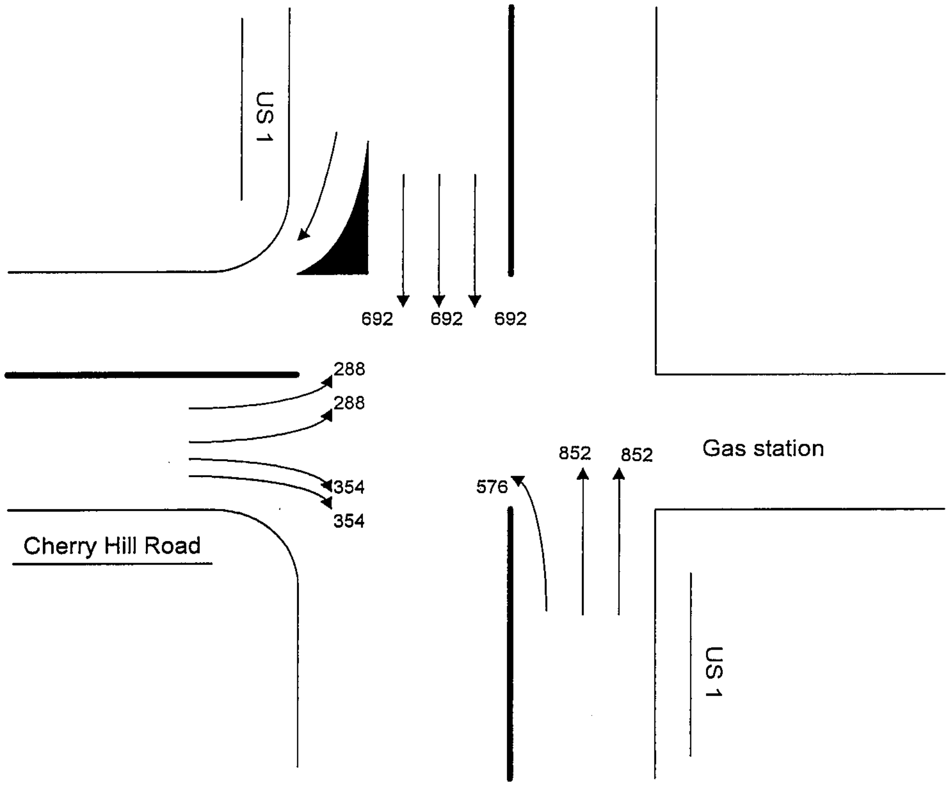

Figure 4. 2 US 1 / Cherry Hill intersection Lane configuration and maximum hourly Volume.

Using the phase diagram for the intersection and the maximum volumes for the lanes, the critical lane volume can be calculated. The phase diagram and the volumes are shown in Figure 4-3.

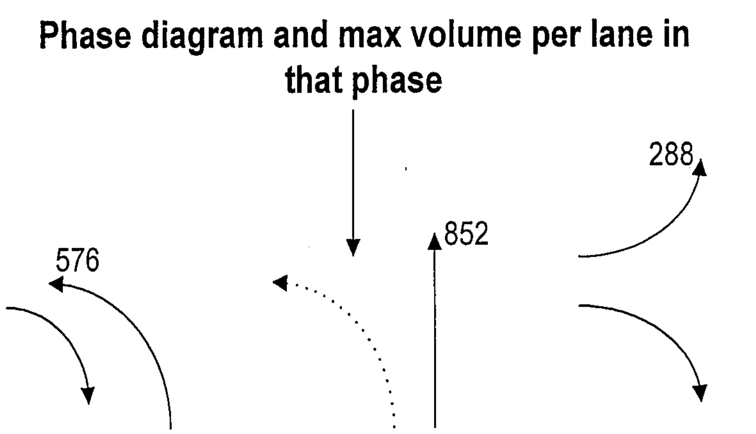

Figure 4. 3 Phasing plan and maximum volume per lane in that phase.

The critical lane volume at this intersection thus equals 1716 vph (= 576 + 852 + 288). As mentioned previously, the intersection has not yet reached its capacity and the actual MCLV could lie between 1750 vph and 1800 vph.

4.4: Simulation analyses for the effects of higher truck percentage on the MCLV

To explore the impacts of truck percentage on the MCLV, the intersection of US1 / Cherry Hill was selected as the candidate site and modeled with the simulation program CORSIM. In performing the simulation, a maximum possible volume was input at all the legs of the intersection so that the intersection could be operated very close to its capacity. Different percentages of trucks were then used as an input variable to simulate the effect of trucks on the MCLV of the intersection.

The results of the simulation are shown in Figure 4-4. It can be seen that the capacity of the intersection reduces by 3% when the truck percentage is increased to 5% from 0% and reduces further by 5% when the truck percentage is up to 10%.

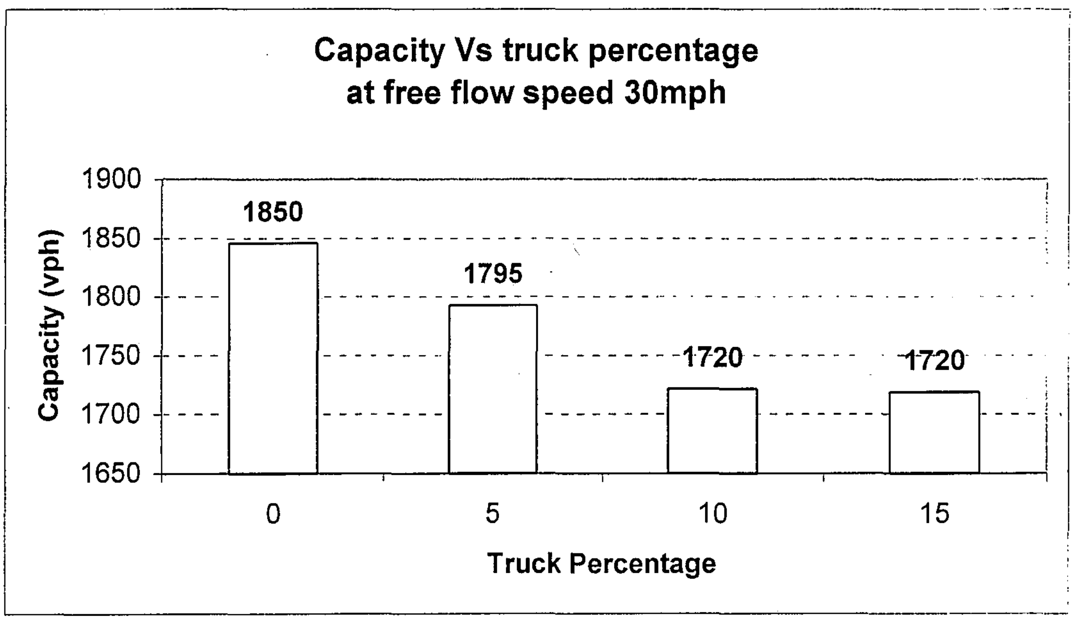

Figure 4. 4: Impacts of truck percentage on the MCLV.

4.5: Simulation of all 11 intersections being observed in the field data collection

From the above analysis, it seems that the MCLV for signalized intersection may lie between 1700 and 1800 vph. To further verify the applicability of this proposed range for MCLV, a rigorous simulation experiment was also conducted with respect to all those eleven intersections observed in the field data collection. This is due to the representative status of each intersection in its county, and also the availability of driver behavioral data such as startup delay and discharging headways that are essential input for a reliable simulation analysis.

Table 4-3 lists the MCLV at each of those intersections computed from extensive simulation experiments. The method used to simulate this set of intersections is the same as the one for the US1/Cherry Hill intersection, that is, oversaturated volumes were used as input to all legs of each intersection, and multiple replications of simulation runs with different random numbers were executed to obtain a statistically reliable average value.

	Intersection	**MCLV (vph)**
1	US 1 / MD 212	1700
2	MD 97 / MD 192	1750
3	MD 216 / All Saint's Road	1730
4	US 301 / MD 197	1750
5	MD 648 / MD 177	1700
6	US 40 / Rolling Road	1690
7	MD 85 / Crestwood Village Blvd	1710
8	MD 152 / US 1	1680
9	US 40 / MD 272	1700
10	US 50 / MD 213	1650
11	US 50 / Mill street	1770

Table 4. 3: Results of each intersection MCLV from Simulation analyses

Considering the variation of geometric as well as traffic conditions in the above set of intersections, the results in Table 4-3 seem to offer a solid support to the finding that the MCLV for most local intersections is higher than 1600 vph used in the current CLV applications, and most likely lies between 1700 vph and 1800 vph, depending on the type of intersections and the aggressiveness of drivers.

Chapter 5: Conclusions and Recommendations.

5.1: Research Finding and Conclusions

In response to the increasing concern of the MCLV value currently used by state and county engineers for traffic impact analysis, this study has focused on the following two critical issues: "*Is the 1600 vph a reasonable approximate of the MCLV for local signalized intersections?*" and "*What is the most appropriate value for the MCLV that can realistically reflect the driving behavior of Maryland populations*?"

To answer these two interdependent issues, the research team has grounded both the analytical and simulation approaches on traffic characteristic data collected from 11 representative local intersections, and divided the entire investigation into the following sequence of stages:

- Using field observation data to compute the key traffic characteristics such as discharging headways, start-up delays, saturation headways, and the average time loss per cycle at each selected intersection;

- Computing the ideal saturation flow rate at each sampled intersection based on the estimated average headway;

- Conducting an additional set of field observations to identify key factors, such as loss time during signal phase transition and truck percentage in the traffic stream, that may contribute to the reduction of the MCLV;

- Estimate the MCLV based on the observed average saturation flow rate, average loss time per cycle, and a given truck percentage in the traffic stream;

- Compare the proposed MCLV with the critical lane volume collected from a very congested intersection that is sure to be close or over its capacity; and

- Modeling each intersection included in the field observations with a commonly used simulation program, CORSIM, and computing its maximum critical lane volume by incorporating observed key traffic characteristics (e. g., startup delay, discharging and average headways) and an artificially increased volume in each intersection approach.

Through the above investigation process, both the analytical and simulation methods have consistently yielded the following findings:

- The MCLV of 1600vph, proposed by the original CLV document in more than three decades ago, is below the actual critical lane capacity of local signalized intersections; and

- Although the actual MCLV for a local intersection may vary with a variety of factors such as geometry and driving characteristics, it most likely lies between 1700 vph and 1800 vph.

Thus, with respect to the most appropriate MCLV for statewide traffic impact analysis, it is recommended that:

- the value of *1800 vph* be used as the MCLV for intersections that have major arterials on all four approaches or are known to have aggressive driving patterns;

- the value of 1700 vph be used as the MCLV for intersections having mainly minor roads and/or community roads;

- the average value of 1750 vph be adopted as the MCLV for local intersections that have both major and minor roads in their approaches, or are at the planning stage having only limited traffic as well as geometric related information available.

5.2: Recommendations of the CLV for each level of service (LOS)

Regarding the appropriate CLV for each level of service (LOS), it is worth noting that the boundaries between different LOSs are created artificially by traffic professionals for convenience of analysis, not grounded on any fundamental traffic nature or theoretical results. Traffic conditions at a given intersection due to their random nature are likely to fall in the boundaries between two neighboring LOSs, or actually vary between them, depending on the evolution of traffic patterns. Under such scenarios, it will be more appropriate to classify the given intersection as, for instance, between LOS C and D, rather than strictly defining it as just in one particular LOS. Thus, one ought not to view the MCLV for each LOS as a precise indicator, but an approximate for reference, especially for planning applications.

With this notion in mind, the following two methods may be used to identify an appropriate CLV for its corresponding LOS:

- Using the average vehicle delay per 15 minutes defined in the HCM for each LOS to estimate the corresponding CLV; and

- Simply scaling up the CLV for each LOS in the current application (i.e., 1600 vph for MCLV) to the selected MCLV.

For convenience of using the first method, Figure 5-1 has presented the interrelation between the critical lane volume and corresponding delay at the interval of 15 minutes. Based on the average delay per 15 minutes specified in the HCM for each LOS, one may redefine the corresponding CLV for the MCLV of 1700 vph and 1750vph, respectively, as shown in Tables 5-1 and 5-2.

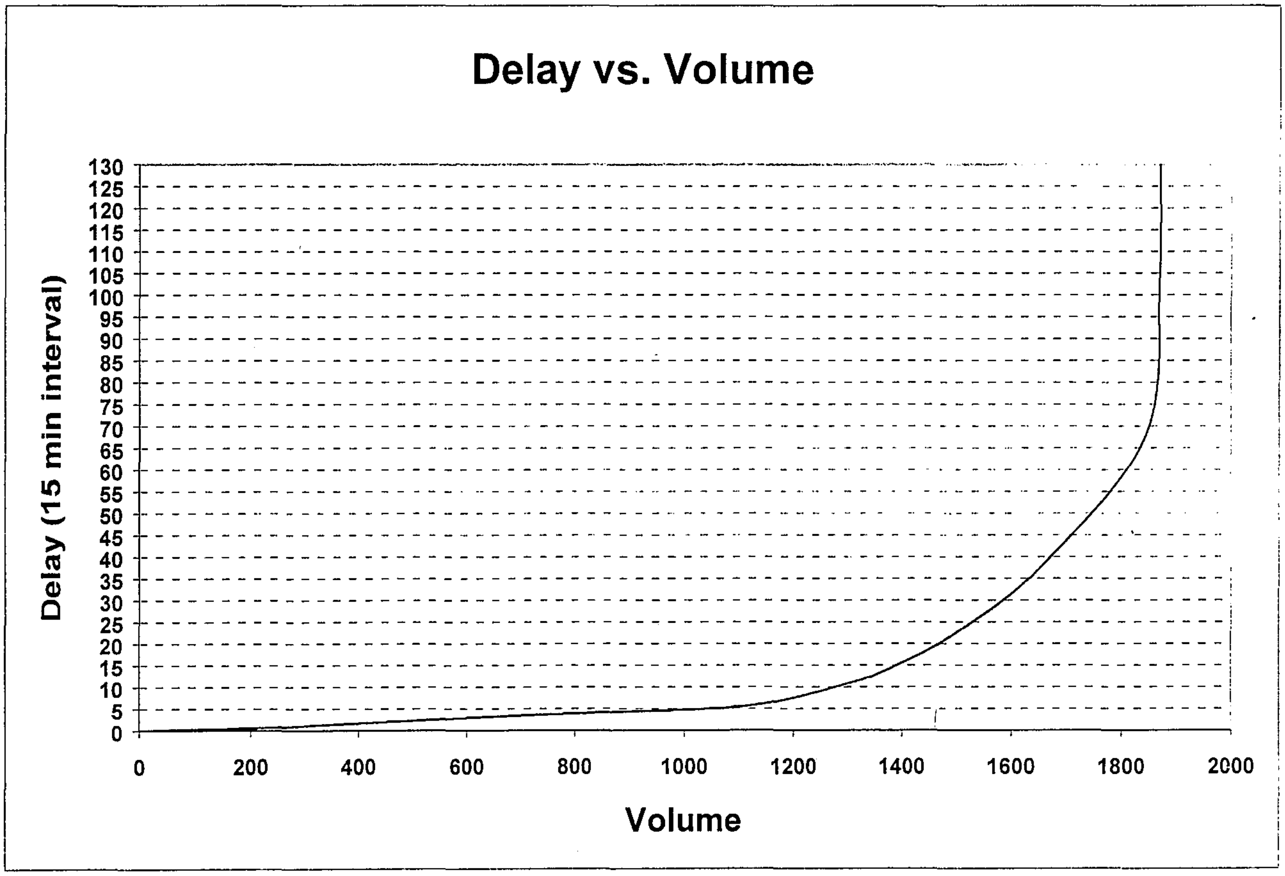

Figure 5. 1: Plot of delay versus the critical lane volume from simulation results.

Delay (15 min interval)	Critical lane Volume (vph)	LOS
≤10	≤1200	A
>10 and ≤20	≤1450	B
>20 and ≤35	≤1600	C
>35 and ≤55	≤1750	D
>55 and ≤80	≤1800	E
>80	>1800	F

Table 5. 1 The CLV for each LOS based on the HCM definitions and the MCLV of 1800 vph

Delay (15 min interval)	Critical lane Volume (vph)	LOS
≤10	≤ 1150	A
>10 and ≤20	≤ 1400	B
>20 and ≤35	≤ 1550	C
>35 and ≤55	≤ 1700	D
>55 and ≤80	≤ 1750	E
>80	>1750	F

Table 5. 2 The CLV for each LOS based on the HCM definitions and the MCLV of 1750 vph

To be consistent with current applications and also to minimize the efforts on reaching a consensus among all potential users, SHA may adopt the second method in estimation of the CLV for each LOS.

To facilitate the comparison, the list of CLVs in current applications and their corresponding values after being scaled up, based on the MCLV of 1700 vph, 1750 vph, and 1800 vph are presented in the following sequence of tables:

Critical lane Volume (vph)	LOS
≤ 1000	A
≤ 1150	B
≤ 1300	C
≤ 1450	D
≤ 1600	E
> 1600	F

Table 5. 3: Critical Lane volumes currently used by the SHA (MCLV = 1600 vph)

Critical lane Volume (vph)	LOS
≤1100	A
≤1200	B
≤1400	C
≤1550	D
≤1700	E
□1700	F

Table 5. 4: Critical Lane Volumes scaled up to 1700 based on Table 5-3

Critical lane Volume (vph)	LOS
≤ 1100	A
≤ 1250	B
≤ 1400	C
≤ 1600	D
≤ 1750	E
> 1750	F

Table 5. 5 Critical Lane Volumes scaled up to 1750 vph based on Table 5-3

Critical lane Volume (vph)	LOS
≤ 1100	A
≤ 1300	B
≤ 1450	C
≤ 1650	D
≤ 1800	E
> 1800	F

Table 5. 6: Critical Lane Volumes scaled up to 1800 vph based on Table 5-3

5.3: Future research needs

Despite the use of rigorous methods and high quality field data for this study, it should be recognized that all research findings are certainly most applicable to intersections that share the similar traffic characteristics and geometric features as those selected for field observations. The research results, however, may not be sufficient for use at locations that are subjected to the impacts of some additional critical factors not observed during the field investigation of this study, such as a high volume of transit vehicles, the presence of bus stations, and double or triple left-turn lanes. Thus, a supplemental study to address potential interrelations between those critical factors and the resulting MCLV as well as intersection capacity will be essential.

Some of such issues to be addressed in the future study are briefly stated below:

- The impacts of different geometric features such as T intersections, double or triple turning lanes on the MCLV;

- The interrelation between the percentage of transit vehicles and the resulting MCLV at some typical downtown intersections;

- The potential reduction in the MCLV due to the presence of bus stations at either the near-side or far-side of an intersection;

- The likely impact of the speed limit on the approaching traffic flow speed and the resulting saturation flow rate as well as the MCLV, especially for those intersections on high-speed expressways such as Route-29;

- The guidelines for design of cycle length and phasing plans that may optimize the MCLV at signalized intersections; and

- The potential need to set different MCLVs for both signalized and non-signalized intersections at residential communities that often have very slow traffic.

References:

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5. Pervedouros, P. and Koga, C., "*Comparison of 1985 and 1994 HCM: Signalized intersection delay estimates*" ITE July (1996) p 26-30.