9-1-1999

Impact of Non-Local Drivers on the Capacity at Signalized Intersections

Yanhu Zhou

J. John Lu

Follow this and additional works at: https://scholarcommons.usf.edu/cutr_reports

Scholar Commons Citation
Zhou, Yanhu and Lu, J. John, "Impact of Non-Local Drivers on the Capacity at Signalized Intersections" (1999). CUTR Research Reports. 75.
https://scholarcommons.usf.edu/cutr_reports/75

This Technical Report is brought to you for free and open access by the CUTR Publications at Scholar Commons. It has been accepted for inclusion in CUTR Research Reports by an authorized administrator of Scholar Commons. For more information, please contact scholarcommons@usf.edu.
IMPACT OF NON-LOCAL DRIVERS ON
THE CAPACITY AT SIGNALIZED INTERSECTIONS

Submitted to:
Florida Department of Transportation

Center for Urban Transportation Research
College of Engineering, University of South Florida
4202 E. Fowler Avenue, CUT 100
Tampa, Florida 33620-5675

Principal Authors:

Yanhu Zhou
J. John Lu, Ph.D., P.E.
Edward A. Mierzejewski, Ph.D., P.E.
Xuewen Le

September 1999
ACKNOWLEDGEMENTS

The authors express their sincere appreciation to the Florida Department of Transportation for the support of this project. Specially, the assistance of the following people is recognized: Douglas McLeod and Kurt Eichin of the FDOT Systems Planning Office, and Keith Crawford of the FDOT District VII. It was through their support and assistance that this project became a reality.

The opinions, findings and conclusions expressed in this publication are those of the authors and not necessarily those of the Department of transportation at the U.S. Department of Transportation.

Prepared in cooperation with the State of Florida Department of Transportation and the U.S. Department of Transportation.
## TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>LIST OF TABLES</td>
<td>iii</td>
</tr>
<tr>
<td>LIST OF FIGURES</td>
<td>v</td>
</tr>
<tr>
<td><strong>CHAPTER 1: INTRODUCTION</strong></td>
<td>1</td>
</tr>
<tr>
<td>Research Background</td>
<td>1</td>
</tr>
<tr>
<td>Problem Statements</td>
<td>3</td>
</tr>
<tr>
<td>Study Performed</td>
<td>3</td>
</tr>
<tr>
<td>Technical Considerations</td>
<td>4</td>
</tr>
<tr>
<td>Scope of Report</td>
<td>6</td>
</tr>
<tr>
<td><strong>CHAPTER 2: REVIEW OF PAST STUDIES</strong></td>
<td>7</td>
</tr>
<tr>
<td>HCM Capacity Analysis Procedures</td>
<td>7</td>
</tr>
<tr>
<td>Other Capacity Estimation Methods</td>
<td>11</td>
</tr>
<tr>
<td>Studies of Adjustment Factors</td>
<td>16</td>
</tr>
<tr>
<td>Driver Population Studies</td>
<td>26</td>
</tr>
<tr>
<td><strong>CHAPTER 3: METHODOLOGY</strong></td>
<td>29</td>
</tr>
<tr>
<td>Capacity Estimation</td>
<td>29</td>
</tr>
<tr>
<td>Determination of Non-Local Driver Population Level</td>
<td>32</td>
</tr>
<tr>
<td>Development of Driver Population Factor Tables</td>
<td>34</td>
</tr>
<tr>
<td>Other Adjustment Factors</td>
<td>37</td>
</tr>
<tr>
<td><strong>CHAPTER 4: DATA COLLECTION</strong></td>
<td>38</td>
</tr>
<tr>
<td>Development of Software for Data Collection</td>
<td>38</td>
</tr>
<tr>
<td>Sites Selection</td>
<td>42</td>
</tr>
<tr>
<td>Data Collection</td>
<td>44</td>
</tr>
<tr>
<td><strong>CHAPTER 5: DATA ANALYSIS AND RESULTS</strong></td>
<td>47</td>
</tr>
<tr>
<td>Analysis of Capacity Parameters</td>
<td>47</td>
</tr>
<tr>
<td>Development of Statistical Models</td>
<td>53</td>
</tr>
<tr>
<td>Development of Driver Population Factors</td>
<td>55</td>
</tr>
<tr>
<td>Development of Population Factors Based on Area Types</td>
<td>56</td>
</tr>
<tr>
<td>Case Study</td>
<td>59</td>
</tr>
<tr>
<td><strong>CHAPTER 6: SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS</strong></td>
<td>62</td>
</tr>
<tr>
<td>Summary</td>
<td>62</td>
</tr>
<tr>
<td>Conclusions</td>
<td>64</td>
</tr>
<tr>
<td>Recommendations</td>
<td>66</td>
</tr>
</tbody>
</table>
TABLE OF CONTENTS (CONTINUED)

REFERENCES ......................................................................................... 67

BIBLIOGRAPHY ..................................................................................... 71

APPENDIX A: PROGRAM FOR TRAFFIC DATA COLLECTION AT SIGNALIZED INTERSECTIONS

APPENDIX B: SURVEY FORM FOR THE ZIPCODE INFORMATION

APPENDIX C: REDUCED DATASET
LIST OF TABLES

Table 2-1: Discharge Headway by Queue Position ........................................... 13
Table 2-2: Summary of Saturation Flow Rate in Other Countries ...................... 14
Table 2-3: Comparison of Past Studies of Start-Up Lost Time ......................... 15
Table 2-4: Effect of Speed Limit on Saturation Flows ..................................... 21
Table 2-5: Effect of Speed Limit on Start-Up Lost Time .................................. 22
Table 2-6: Effect of Speed Limit on Clearance Lost Time ............................... 22
Table 2-7: Effect of Cycle Length on Start-Up Lost Time ................................ 23
Table 2-8: Effect of Cycle Length on Clearance Lost Time ............................. 23
Table 2-9: Effect of Green Time on Start-Up Lost Time .................................. 23
Table 2-10: Effect of Green Time on Clearance Lost Time .............................. 24
Table 2-11: Dry-Weather, Peak-Period Straight-Through Saturation Flows ....... 24
Table 2-12: Driver Population Factor f_p Recommended by Sharma .................. 27
Table 4-1: List of Studied Signalized Intersections ......................................... 43
Table 5-1: Summary of Capacity Parameters by Movements ............................ 47
Table 5-2: t-tests of Capacity Parameters Between the Movements .................. 48
Table 5-3: Mean of Capacity Parameters by Land Use Characteristics ............... 50
Table 5-4: One-way ANOVA tests of Capacity Parameters among Driver Population Groups ................................................................. 52
Table 5-5: Results of Linear Models ............................................................... 54
Table 5-6: Saturation Flow Rate under a Given Population Level ...................... 55
Table 5-7: Driver Population Factors Based on Non-Local Driver Population
LIST OF TABLES (CONTINUED)

Table 5-8: Range of Non-Local Driver Population Levels of Each Area Type......57

Table 5-9: Distribution of Non-Local Driver Population Level......................58

Table 5-10: Driver Population Factors $f_{\text{pop}}$ Based on Area Types.............59

Table 5-11: Peak-Hour Traffic Volume (vph) at Intersection S.R.192 & S.R.535..60

Table 5-12: Traffic Signal Phase Plan at Intersection S.R.192 & S.R.535.............60

Table 5-13: Capacity and LOS Before and After Driver Population Adjustment…61

Table 6-1: Driver Population Factors $f_{\text{pop}}$ in Terms of Non-Local Driver Population Level..................................................................................................................65

Table 6-2: Driver Population Factor $f_{\text{pop}}$ in Terms of Area Type....................65
LIST OF FIGURES

Figure 2-1: Operational Analysis Modules for Signalized Intersections .............. 7

Figure 2-2: Comparison of Past Studies of Queue Discharge Headway .............. 12

Figure 2-3: Effect of Lane Width on Saturation Flow Values .......................... 17

Figure 3-1: Raw Database to Estimate Driver Population Factor at Signalized Intersections ..................................................................... 35

Figure 3-2: Driver Population Factors in terms of Non-Local Driver Population Level ................................................................. 36

Figure 3-3: Driver Population Factors in terms of Area Type .......................... 36

Figure 4-1: Physical Information of Data Collection .................................... 39

Figure 4-2: Interface of Traffic Data Collection Window .............................. 39

Figure 5-1: Left-Turn Saturation Headway vs. Non-Local Population Level....... 53

Figure 5-2: Through Saturation Headway vs. Non-Local Population Level ....... 53

Figure 5-3: Distribution of Non-Local Drivers of Each Area Type ................. 57

Figure 5-4: Geometric Condition of the Intersection S.R. 192 & S.R. 535 ......... 60
CHAPTER 1: INTRODUCTION

Research Background

The capacity of a transportation facility reflects its ability to accommodate a moving stream of people or vehicles. Capacity estimation is essential in traffic operation analysis, transportation planning and decision-making because it assists transportation practitioners to evaluate and improve the performance of existing facilities, and to plan and design new facilities.

Studies of highway capacities date back to the 1920s. Since then, many capacity analysis procedures have been developed and applied. A survey conducted by Virginia Department of Transportation (VDOT) indicated that procedures included in the Highway Capacity Manual (HCM) were the most prevalent and acceptable capacity estimation methods among the USDOT (US Department of Transportation) [Arnold and McGhee 1995].

The latest version of HCM, the 1997 update, was published and is applied worldwide. All the capacity analysis procedures in this update are based on the extensive nationwide studies since the 1920s. It includes current speed-flow relationships, revised capacity values, and new analytical procedures. The basic principle of HCM capacity analysis procedures is to take the maximum flow rate that can be achieved by a highway facility under ideal conditions as a base value. Capacity under prevailing conditions is estimated by applying adjustment factors to this base value. General ideal conditions in HCM assume good weather, all passenger cars in the traffic stream, good pavement, users familiar with facility, and no incidents impeding traffic flow. For each specified ideal condition, there is an adjustment factor to account for any non-ideal occurrence. Adjustment factors are classified in HCM as geometric factors, traffic factors, operating factors, environmental factors and driver population factors [Stokes 1989]. Geometric factors are developed to adjust the impacts of number of lanes, lane width, approach grades, turning radii, or lane configuration (including parking lanes). Traffic factors are developed to adjust the impacts of traffic composition and vehicle mix. Operating factors
are developed to adjust the impacts of speed limit, signal timing, phasing arrangement, peak activities, or bus stop operations. Environmental factors are developed to adjust for weather, area population, roadway surface conditions, or adjacent land uses. Driver population factors reflect driver familiarity with the facilities, driving experience and knowledge. Except for driver population factors, other adjustment factors are well addressed in the HCM, and clear procedures are provided to calculate these factors.

The driver population condition is ideal when most users in the traffic stream are weekday commuters or local drivers who are familiar with road facilities. Previous studies indicated that any driver population groups other than weekday commuters would utilize freeway facilities less efficiently, and freeway capacity was reduced significantly due to the existence of non-local drivers in the traffic stream [CUTR 1997]. Non-local drivers or non-commuters might cause capacity reduction in several ways, including perception-reaction time, car-following behavior, lane change, gap acceptance behavior, and operating speed. As the level of non-local drivers or non-commuters increases, these factors together would contribute to the expected reduction in highway capacity. However, very little attention has been paid in the HCM capacity analysis procedures to adjust the impacts of the non-local drivers. Driver population factors are only applied to adjust the impacts of non-local drivers on the capacity of basic freeway sections in HCM, not for the other facilities. Even for basic freeway sections, driver population factors are concluded to be in the range of 0.85-1.00. No detailed guidelines were given on how to select a value from this range. One reason that driver population factors are not accurately described in HCM is because of the difficulty in quantifying the mix of drivers in some understandable and predictable fashion. Driver population could be defined by a number of factors, such as trip purpose, trip duration, driver age and driver familiarity with the facility. There may be some inter-relationships between these elements, and therefore, it is impossible to determine the effect of any single element on the capacity. The other reason is that driver population factors varied greatly among the states. In order to overcome these problems, it is recommended to perform driver population factor studies in those states with large number of visitors, and focus on quantifying major elements of driver population factors [McShane, Roess and Prassas 1998].
Problem Statements

Florida attracts millions of domestic and international tourists each year. In 1998, more than 48 million people visited Florida, of which more than 43 million people visited by car [Tampa Tribune]. The research conducted by CUTR in 1996 indicated that the capacity of the basic freeway section could be reduced as much as 15% at highly recreational areas in Florida [CUTR 1997]. Since there are also many visitors on local streets, reduction of capacity at signalized intersections in Florida may also be expected due to the driving behavior of these non-local drivers.

In order to perform capacity analysis more adequately, it is necessary to thoroughly examine the impacts of these non-local drivers on the capacity at signalized intersections in Florida, and then to quantify those impacts by introducing the driver population factors into the capacity analysis procedure. Such factors may help to better estimate capacities at signalized intersections in Florida, which in turn assists transportation practitioners to evaluate existing facilities and to design and plan future facilities more cost-effectively.

Study Performed

In 1997, the Florida Department of Transportation (FDOT) contracted with the Center for Urban Transportation Research (CUTR) at the University of South Florida (USF) to conduct a research project, “Impact of Non-Local Drivers on the Capacity at Signalized Intersections.” The purpose of the project was to examine and quantify the impacts of non-local drivers on the capacity of signalized intersections in Florida, and to develop driver population factors for the capacity analysis procedure.

In this project, driver population factors were deduced by examining the relationship of capacity and its corresponding non-local driver population levels. Capacity and driver population level were computed based on field data. With the consideration of technical accuracy, practical feasibility, and available resources, traffic data were collected by a laptop computer in the field, and driver population information was obtained by performing roadside interviews of the signalized intersections.
The main goal of the project was to quantify the impacts of non-local drivers on the capacity at signalized intersections, and to develop driver population factors for the capacity analysis procedures of signalized intersections. This goal was achieved through the accomplishment of the following specific objectives:

- To conduct traffic data collection and roadside interviews in the field;
- To estimate the capacity and the corresponding non-local driver population level based on the field data;
- To examine the relationship between the capacity and the non-local driver population level; and
- To develop driver population adjustment factor tables for the capacity analysis procedure of signalized intersections.

Technical Considerations

Since the purpose of this research was to develop driver population adjustment factors for the capacity analysis procedure at signalized intersections, definitions, background and considerations about the studies of the driver population and the capacity at signalized intersections are presented as follows.

Driver populations can be defined based on different criteria, such as trip purpose (commuter vs. recreational), familiarity with the facilities (local vs. non-local), time of the day (weekend vs. weekday), driver age (average vs. the elderly), and/or trip length (short distance vs. long distance) [Lu, Huang, and Mierzejewski 1997]. There might be some inter-relationships between these elements, and thus, it is impossible to determine the impact of any single element on highway capacities. However, a major element could be singled out to represent driver population. For a state like Florida with a significant number of out-of-state visitors, it is reasonable to suggest that familiarity with facilities be the major element and non-local driver population levels in a traffic stream be used as a descriptor of the driver population. Thus, this research concentrated on analyzing the impacts of non-local driver population levels on the capacity at signalized intersections.
The impacts of non-local drivers on the capacity at signalized intersections may be reflected by their different perception-reaction time to the phase change, different usage of yellow interval, and/or their different car following behavior. Non-local driver population levels at signalized intersections can be estimated by performing roadside interviews, using video camera, or conducting mail-in postcard surveys.

Capacity at a signalized intersection is defined for each lane group. The lane group capacity in HCM 1997 update is defined as the maximum flow rate for the subject lane group that may pass through the intersection under prevailing traffic, roadway, and signalization conditions. The capacity of a given lane group is stated in HCM as:

\[ c_i = s_i \times \frac{g_i}{C} = s_i \times \frac{(G + Y + AR - t_s - t_c)}{C} \]

where:

- \( c_i \) = capacity of lane group i, vph;
- \( s_i \) = saturation flow rate for lane group i, vphg;
- \( C \) = cycle length, sec.;
- \( G \) = green time for each cycle, sec.;
- \( Y \) = yellow change interval, sec.;
- \( AR \) = all-red interval, sec.;
- \( t_s \) = start-up lost time, sec.;
- \( t_c \) = clearance lost time, sec.; and
- \( g_i \) = effective green time, \( g_i = G + Y + AR - t_s - t_c \), sec.

The above equation indicates that the capacity at a signalized intersection depends on saturation flow rate \( s_i \), start-up lost time \( t_s \) and clearance lost time \( t_c \) for a given timing allocation. Saturation flow rate is defined in 1997 HCM update as the maximum flow rate that can pass through a given lane group under prevailing traffic and roadway conditions, assuming that the lane group has 100 percent of real time available as effective green time. Start-up lost time and clearance lost time are the time during which the intersection is not effectively used by any movement. Start-up lost time occurs at the beginning of each phase due to the delay of response time to the phasing change for the first few
vehicles, and clearance lost time occurs at the end of each phase due to the partial usage of yellow change interval and all-red interval.

**Scope of Report**

Six chapters were included in this report. Chapter 1 was the introduction of the proposed research study. Chapter 2 reviewed past studies on the capacity analysis at signalized intersections. Previous studies related to the driver population were also reviewed in this chapter. In Chapter 3, the detailed methodology of developing driver population factors for the capacity analysis procedure at signalized intersections was explained. Experimental design, description of sites and data requirements, and preliminary data processing were presented in the Chapter 4. Model development and establishment of driver population factor tables for signalized intersections were given in Chapter 5. A case study was conducted to examine how the driver population factors affected the capacity and level of service at a signalized intersection and the results were presented in Chapter 5. Chapter 6 discussed the summaries, conclusions and recommendations of this study.
CHAPTER 2: REVIEW OF PAST STUDIES

HCM Capacity Analysis Procedures

The *Highway Capacity Manual* (HCM) provides a resource for technical information that is used by transportation planners, designers and operators. The results included in the latest 1997 update are based on extensive research studies in the last 40 years, which represent the best available techniques for determining capacity. The procedures and parameters in this manual provide a systematic and consistent basis for assessing the capacity and quality of service for various types of transportation facilities.

The operational modules of the capacity analysis procedure for signalized intersections in HCM are shown in Figure 2-1 [Roess 1987].

![Figure 2-1 Operational Analysis Modules for Signalized Intersections](Source: Roess, 1987)
As indicated in HCM, capacity at signalized intersections is defined for each lane group. A lane group is defined as one or more lanes that accommodate traffic and have a common stop line and capacity shared by all vehicles. The lane group capacity is the maximum rate of flow for the subject lane group that may pass through the intersection under prevailing traffic, roadway, and signalization conditions. The capacity of a given lane group is expressed by the following equation:

\[ c_i = s_i \left( \frac{g_i}{C} \right) \]

where:

\[ c_i \] = capacity of lane group i, vph;  
\[ s_i \] = saturation flow rate for lane group i, vphg; and  
\[ \frac{g_i}{C} \] = effective green ratio for lane group i.

Saturation flow rate \( s_i \) is the maximum flow rate that can pass through a given lane group under prevailing traffic and roadway conditions, assuming that the lane group has 100 percent of real time available as effective green time. Computations of saturation flow rate under prevailing conditions are based on the saturation flow rate under ideal conditions and adjustment factors for the prevailing conditions. The ideal conditions at a signalized intersection approach are specified in HCM as follows:

- 12-foot lane width;  
- level approach grade;  
- only passenger cars in the traffic stream;  
- no left- or right-turning vehicles in the traffic stream;  
- no parking adjacent to a travel lane within 250 ft of the stop line; and  
- intersections located in non-CBD area.

In HCM, computations of saturation flow rate begin with the selection of an “ideal” saturation flow rate, usually 1900 passenger cars per hour of green time per lane (pcphgpl), and this value is adjusted for a variety of prevailing conditions that are not ideal. Saturation flow rate under prevailing conditions then can be estimated by the following equation:

\[ s = s_0 N f_w f_{HV} f_g f_p f_{bb} f_a f_{LU} f_{RT} f_{LT} \]
where:

\[ s = \text{saturation flow rate for the subject lane group, expressed as a total for all lanes in the lane group under prevailing conditions, vphg;} \]
\[ s_0 = \text{ideal saturation flow rate per lane, usually 1900 pcphgpl;} \]
\[ N = \text{number of lanes in the lane group;} \]
\[ f_w = \text{adjustment factor for lane width;} \]
\[ f_{HV} = \text{adjustment factor for heavy vehicles in the traffic stream;} \]
\[ f_g = \text{adjustment factor for approach grade;} \]
\[ f_p = \text{adjustment factor for the existence of a parking lane adjacent to the lane group and the parking activity in that lane;} \]
\[ f_{bb} = \text{adjustment factor for the blocking effect of local buses that stop within the intersection area;} \]
\[ f_a = \text{adjustment factor for area type;} \]
\[ f_{LU} = \text{adjustment factor for lane utilization;} \]
\[ f_{RT} = \text{adjustment factor for right-turns in the lane group;} \]
\[ f_{LT} = \text{adjustment factor for left-turns in the lane group.} \]

As indicated in the above equation, conditions, such as lane width, heavy vehicle composition, approach lane grade, parking, bus blockage, area type, lane utilization, right turn and left turn movements, are justified if they do not meet the definition of ideal conditions.

Saturation flow rate can also be estimated by field measurement. It is indicated in HCM that measured values of prevailing saturation flow rate in the field will produce more accurate results than the estimation procedure described above. As stated in the procedure of field measurement, saturation flow rate is usually achieved after the fourth to seventh vehicle has entered the intersection from a standing queue. Vehicles are recorded when their rear axles cross the stop line. Since other reference points may yield different saturation flow rates, in order to maintain consistency and to allow for information exchange, it is essential to maintain the roadway and vehicle reference points identical.
In field measurement, saturation flow rate is determined by the saturation headway. Saturation flow rate is the reciprocal of saturation headway at signalized intersections. Saturation headway is measured by averaging the total time-lapse between the rear axle of the fifth vehicle and of the last queued vehicle crossing the stop line. Saturation flow rate then can be calculated by the following equation:

\[ s = \frac{3600}{h_s} \]

where:

- \( s \) = saturation flow rate under prevailing conditions, in pcphgpl; and
- \( h_s \) = saturation headway in seconds.

Another variable affecting the capacity of a signalized intersection is the effective green time, \( g_i \), which is calculated by the following equation:

\[ g_i = G - t_{sl} - t_{cl} \]

where:

- \( G \) = green time, sec.;
- \( t_{sl} \) = start-up lost time, sec.; and
- \( t_{cl} \) = clearance lost time, sec.

Lost time is defined in the Chapter 9 of HCM 1997 as the time during which the intersection is not effectively used by any movement. Lost time occurs at the beginning of each phase and during the change-and-clearance intervals. The lost time occurred at the beginning of each phase is called start-up lost time, and the lost time occurred at the change-and-clearance interval (yellow change plus all-red interval) is called clearance lost time.

The 1997 HCM update specifies that saturation headway should be estimated based on the headways of all the queued vehicles except the first four vehicles. This approach implies that the first four vehicles incur most, if not all, of the start-up lost time. Thus, start-up lost time is calculated by the HCM procedure as:
\[ t_{sl} = t_4 - 4 \times h_s \]

where:

- \( t_{sl} \) = start-up lost time, sec.;
- \( t_4 \) = total time from the signal turning green to the rear axle of the fourth vehicle crossing the stop line, sec.; and
- \( h_s \) = saturation headway, sec.

Clearance lost time is the sum of the unused yellow change interval and all-red interval occurred in the same signal cycle. The clearance lost time is usually estimated from field studies. No specific procedures are suggested in HCM to estimate clearance lost time.

It is indicated in HCM that the start-up lost time \((t_{sl})\) is normally about 2 second. It is also shown that the extension of effective green is about 2 second. The remainder of the change-and-clearance time is the clearance lost time. Thus, the total lost time for a movement approximately equals to the sum of yellow and all-red interval in HCM.

**Other Capacity Estimation Methods**

Studies of highway capacity date back to the 1920s. Since then, many analytical procedures have been developed. HCM is just one of the most popular procedures, which reflects the most recent results in the United States. Since saturation flow rate and lost time are the two critical elements in estimating the capacity at a signalized intersection, other methods used to estimate these elements are presented in the next section.

In general, two alternatives have been applied in the past studies to estimate saturation flow rate. One is the queue discharge model, and the other is the discharge headway model. The most widely applied queue discharge model is Webster’s model [Webster 1958]. It was indicated in that model that when a vehicle queue was released by a traffic signal turning green, the departure headways of the vehicles quickly reached a steady condition. The flow remained at about this value until the light changed to amber, when it fell steadily to zero. This uniform departure rate was termed the saturation flow rate, \( s \). This method was only applicable when the cycle was loaded or saturated.
Discharge headway models were more popular in estimating saturation flow rate at a signalized intersection. The saturation flow rate was estimated based upon the minimum discharge headway (saturation headway). Saturation headway was defined as the average discharge headway between passenger cars in a stable-moving queue as they passed through a signalized intersection in seconds. Discharge headway was the time interval between the rear axle of the two consecutive vehicles passing the stop line at an intersection.

All of the previous studies indicated that the discharge headway converged to a constant after a few seconds of green signal indication. Studies conducted by Greenshields et al. in New York City and New Haven [Moussavi 1990] indicated that after five vehicles, the departure headway was constant and stable. Bonneson [1992] summarized previous studies on average vehicle headways by queue position as shown in the Figure 2-2. It indicated that the discharge headway converged to a relatively constant value in each study, which stood for the minimum discharge headway (saturation headway). However, the minimum discharge headways were different from studies. It was also indicated in the Figure 2-2 that the discharge headway varied during the initial portion of the green interval. Such variation reflected the reaction time of the first driver responding to the change in signal indication and the steady acceleration of the first few vehicles in queue.

![Figure 2-2 Comparison of Past Studies of Queue Discharge Headway](source: Bonneson, 1992)
Table 2-1 summarized the results of discharge headway for each queue position based on the previous studies. The discharge headway of each queue position varied among the studies. The reason for the high degree of variability may be because of the difference in traffic, geometric, and/or driver population conditions of the intersections.

Table 2-1 Discharge Headway by Queue Position

<table>
<thead>
<tr>
<th>Queue</th>
<th>Range of Discharge Headway (Sec.)</th>
<th>Average Discharge Headway (Sec.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.04-3.74</td>
<td>2.90</td>
</tr>
<tr>
<td>2</td>
<td>1.87-2.44</td>
<td>2.04</td>
</tr>
<tr>
<td>3</td>
<td>2.00-2.33</td>
<td>2.10</td>
</tr>
<tr>
<td>4</td>
<td>1.96-2.36</td>
<td>2.04</td>
</tr>
<tr>
<td>5</td>
<td>1.64-2.10</td>
<td>1.87</td>
</tr>
<tr>
<td>6</td>
<td>1.87-1.95</td>
<td>1.91</td>
</tr>
<tr>
<td>7</td>
<td>1.69-1.82</td>
<td>1.75</td>
</tr>
</tbody>
</table>

(Source: Moussavi and Tarawneh, 1990)

The variation of saturation flow rate was also found in the capacity studies of other countries. Table 2-2 presented the summary of saturation flow rates in other countries [Niittymaeki 1997]. Due to such variation, field studies were more reliable to reflect prevailing conditions.

As indicated above, the discharge headways of the queued vehicles converged to a relatively constant value, which was the saturation headway, after the first few vehicles. Usually the first few vehicles in a traffic queue have headways in excess of saturation headway. This excess time is commonly referred as start-up lost time because it represents the time that is inefficiently used by the discharge traffic queue. The general equation for start-up lost time is expressed by:

\[ t_{sl} = \sum_{n=1}^{N} (h_n - h_s) \]

where:

\[ t_{sl} = \text{start-up lost time (sec/phase);} \]
\[ h_n = \text{discharge headway of } n^{th} \text{ queued vehicle (sec)}; \]
\[ h_s = \text{saturation headway (sec)}; \text{ and} \]
\[ N = \text{number of queue positions having headways larger than } h_s. \]

Table 2-2 Summary of Saturation Flow Rate in Other Countries

<table>
<thead>
<tr>
<th>Country</th>
<th>Saturation Flow Rate (phgpl)</th>
</tr>
</thead>
<tbody>
<tr>
<td>United Kingdom</td>
<td>Ideal 2080 pcu</td>
</tr>
<tr>
<td>Canada</td>
<td>Max 1900 veh</td>
</tr>
<tr>
<td>Australia</td>
<td>Max 2475 veh</td>
</tr>
<tr>
<td>Israel</td>
<td>Avg. 2176 veh</td>
</tr>
<tr>
<td>Poland</td>
<td>Ideal 1890 veh</td>
</tr>
<tr>
<td>Yugoslav</td>
<td>Ideal 2290 pcu</td>
</tr>
<tr>
<td>South Africa</td>
<td>Ideal 1928 veh</td>
</tr>
<tr>
<td>Germany</td>
<td>Ideal 2000 veh</td>
</tr>
<tr>
<td>Hong Kong</td>
<td>Ideal 1895 veh</td>
</tr>
<tr>
<td>Lithuania</td>
<td>Max 2045 veh</td>
</tr>
<tr>
<td>Japan</td>
<td>Ideal 2000 pcu</td>
</tr>
<tr>
<td>United States</td>
<td>Ideal 1900 pcu</td>
</tr>
<tr>
<td>Finland</td>
<td>Avg. 1940 veh</td>
</tr>
</tbody>
</table>

(Source: Niittymaeki, 1997)

The above equation indicates that the start-up lost time directly depends on the \( h_s \) and \( N \). The HCM procedure implies that \( N \) equals to 4, start-up lost time is 2.0 sec and total lost time equals to the change-plus-clearance interval. However, Shanteau’s study [1988] indicated that actual lost time was less than the value suggested by the HCM (3-5 seconds) and was usually less than the yellow time. Occasionally he observed that at some locations there was even negative start-up lost time, which might be caused by the aggressive driving behavior.

Over the past 20 years, many studies of start-up lost time have been conducted at conventional signalized intersections [Bonneson 1990]. Agent [1983] analyzed 1,428 headways for queued vehicles positioned first, second, or third in Kentucky, and found that the average start-up lost time was 1.40 sec. Zegeer [1986] measured 3,687 vehicles
in Chicago, Houston, and Los Angeles metropolitan areas, indicating the average start-up lost time was 1.31 sec. Results of past studies were summarized in Table 2-3.

Table 2-3 Comparison of Past Studies of Start-Up Lost Time

<table>
<thead>
<tr>
<th>Source</th>
<th>Date of Study</th>
<th>Number of Queued Vehicles</th>
<th>Start-Up Lost Time (sec/phase)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gerlough</td>
<td>1967</td>
<td>4</td>
<td>2.05</td>
</tr>
<tr>
<td>Carstens</td>
<td>1971</td>
<td>3</td>
<td>0.75</td>
</tr>
<tr>
<td>Agent</td>
<td>1983</td>
<td>3</td>
<td>1.40</td>
</tr>
<tr>
<td>Lee</td>
<td>1986</td>
<td>4</td>
<td>3.04</td>
</tr>
<tr>
<td>Zegeer</td>
<td>1986</td>
<td>4</td>
<td>1.31</td>
</tr>
</tbody>
</table>

(source: Bonneson 1990)

Bonneson [1992] also concluded that the start-up lost time of the left-turn movement was larger than that of through-movement at a SPUI (Single-Point Urban Interchange). The start-up lost time of left-turn movement ranged from 2.19 to 2.96 sec/phase, whereas that of through movement ranged from 1.98 to 2.44 sec/phase.

Clearance lost time has been measured in past studies by one of two methods. The first method is based on direct measurement of the unused time at the end of a phase. The criterion for this measurement is that the phase is fully saturated (i.e., a “loaded” cycle). Agent [1983] applied the direct measurement at several signalized intersections and found that an average clearance lost time for through movement was 1.67 seconds. He concluded that the factors affecting clearance lost time included length of yellow, cycle length, city population, gradient, speed limit, and type of lane. It was also concluded in his article that the average clearance lost time for exclusive left-turn lanes was 1.27 seconds. The second method assumes that the initial portion of every yellow interval is available to drivers. This method is to measure the amount of the yellow interval used by the clearing drivers. Clearance lost time is determined by subtracting this end use from the total yellow plus all-red interval. Since all-red interval is not allowed to be used by any vehicle, the clearance lost time could be stated as being equal to the sum of the all-red clearance interval and the unused portion of the yellow change interval:

\[ t_{cl} = AR + Y - EU \]
where:

\[ t_{cl} = \text{clearance lost time, sec.;} \]
\[ \text{AR} = \text{all-red time, sec.;} \]
\[ Y = \text{yellow change interval, sec.; and} \]
\[ \text{EU} = \text{average usage of yellow interval, sec.} \]

The formula indicates that the clearance lost time relies on the usage of yellow interval. The usage of yellow interval by clearing vehicles implies that drivers do not have sufficient distance to stop comfortably once the signal change is perceived. Williams [1977] indicated that the probability of vehicles to stop was a function of distance from the stop line and approach velocity when signal turned yellow.

Due to the difficulties in measuring clearance lost time, very few attempts were made to measure this variable in the past. The recent study by Hook et. al. [1992] reported that the mean clearance lost time was in the range of 2.78 to 4.22 seconds. Maini [1997] reported that half of the yellow interval was not used for the saturated flow conditions, and 75% of the yellow interval was not used for the unsaturated flow conditions.

Carstens' study [Bonneson 1992] found that drivers used more of the yellow interval on higher speed approaches. Bonneson's study [1992] indicated that traffic demands also had significant impact on the end use of left-turn phase. This was because left-turn drivers may be expected to use more of the yellow interval as traffic demands increased to avoid a perceived lengthy delay.

**Studies of Adjustment Factors**

As stated in the HCM, capacity under prevailing conditions is estimated by adjusting the ideal capacity with adjustment factors. In the past, extensive studies were conducted to address the impacts of the prevailing conditions on the saturation flow rate. The adjustment factors of lane width, heavy vehicles, approach grades, curb parking, local bus stop activity, area type, lane utilization, turning movement are explained very well in the HCM 1997 update. The following section briefly reviews these factors as well as those not covered by HCM.
(1) Lane Width

In the HCM, the impact of lane width on the saturation flow rate is formulated as:

\[ f_w = 1 + \frac{(w - 12)}{30} \]

where:

- \( f_w \) = adjustment factor for average lane width; and
- \( w \) = average lane width (ft), \( w \geq 8 \).

Other studies also indicated that lane width had significant impacts on saturation flow rate of a signalized intersection. The effect of lane width on saturation flow rate in the previous studies was summarized in Figure 2-3 [Niittymaeki 1997].

![Figure 2-3 Effect of Lane Width on Saturation Flow Values](Source: Niittymaeki, 1997)

(2) Heavy Vehicles

In HCM, the heavy vehicle adjustment factor is calculated by the following equation:

\[ f_{HV} = \frac{100}{100 + \%HV(E_T - 1)} \]
where:

\[ f_{HV} = \text{adjustment factor for heavy vehicles in the traffic stream}; \]
\[ \%HV = \text{percentage of heavy vehicles}; \]
\[ E_T = \text{heavy vehicle passenger car equivalent}. \]

The results included in HCM were based on the study by the JHK & Associates [Zegeer 1986]. A heavy vehicle in their study was defined as any truck or bus having six or more tires on the pavement. The average headway between preceding passenger vehicles and trucks was 2.06 sec. The average headway between trucks and following passenger vehicles was 2.61 sec. The average headways between all vehicles preceding or following a bus were 3.10 sec. This implied a heavy vehicle passenger car equivalent of between 1.3 and 1.6.

Carstens’ survey [1971] indicated that a heavy vehicle passenger car equivalent for through movement was 1.6. Zegeer’s study [1986] indicated that the average headway between passenger cars was 1.92 sec. The average headway for the condition of truck following passenger cars was 5.16 sec., of passenger cars following trucks was 2.22 sec., and of truck following trucks was 3.76 sec. He suggested a heavy vehicle passenger car equivalent of 1.92.

(3) Grades

The impact of grades on the capacity at signalized intersections is complex. Usually, a combination of heavy vehicles and grades creates impact. Heavy vehicles cannot maintain the same speed as passenger cars on grades of appropriate length and severity, and therefore create large gaps in the traffic stream that cannot be effectively filled by normal passing maneuvers. In HCM, both passenger cars and heavy vehicles are affected by grades at signalized intersections. The results included in the HCM show that capacity or service flow increases at downhill and decreases at uphill.

The results in the NCHRP report showed reductions in saturation flow rate for upgrades [Zegeer 1986]. The reduction for upgrades were as follows: 3.1% for 3% upgrades, 4.6% for upgrades of between 4% and 5%, 23.9% for upgrades of between 6% and 7%.
Zegeer’s study [1986] also indicated a reduction in flow rates for downgrades. The reason was that drivers on the relatively steep downgrades proceeded through the intersections in a manner that was somewhat more tentative than that used on intersection approaches with no discernable grade.

(4) Curb Parking

Parking adjustment factor accounts for the frictional effect of a parking lane on flow in an adjacent lane group, as well as for the occasional blocking of an adjacent lane by vehicles moving into and out of parking spaces. The results included in the latest HCM were derived from FHWA films taken in 1975 of vehicle parking movement. It was indicated in this research that saturation flow rates were 9.2 percent lower in lanes adjacent to curb parking than in similar lanes not adjacent to curb parking.

In Transportation Research Circular 212 [1980], it was stated that, “Most North American techniques do not explicitly consider a reduction in capacity due to parking, if the parking ends 250 feet before the intersection stops…”

Zegeer’s study [1986] indicated that the saturation flow rate was reduced by 11 percent for vehicles traveling in lanes adjacent to curb parking. The average parking and unparking maneuver blocked the adjacent lane for about 7 seconds.

(5) Left-Turn movement

Turning movement factor is another important characteristic affecting saturation flow rate of a signalized intersection. The left-turn factor is calculated by comparing left-turn saturation flow rate with that of through movement with the other factors fixed.

The study by the JHK & Associates showed that the saturation flow rates for exclusive left-turn lanes with protected signal phases was reduced by 15 percent [Zegeer 1986]. In Transportation Research Circular 212, the recommended adjustment factor for an exclusive left-turn lane with a protected turn phase is 0.95. Zegeer’s study [1986] indicated an average 3 percent reduction in saturation flow rate when compared with through-lane headways at comparable locations. Thus, a 0.97 factor for single left-turn
lanes with protected phase was proposed. Ackeret [1996] conducted a study of left-turn factors in Las Vegas, Nevada. He concluded that the left-turn factor for single left-turn lane with protected left-turn signal phasing was 1.0, for dual left-turn lanes was 0.98, and for triple left-turn lanes was 0.95. However, the studies by ITE subcommittee concluded that the left-turn saturation flow rate may not be substantially lower than through lane flow rates. A left-turn factor of 1.0 may be more appropriate for certain intersection geometric conditions [ITE 1995 Compendium].

(6) Right-Turn Movement

The HCM 1997 update includes a right-turn adjustment factor of 0.85 for exclusive right-turn lanes controlled by protected signal phase. Zegeer’s study [1986] found when the curb radii was between 10 and 30 ft, the average saturation headway for right-turn movement was 19 percent longer than that for through movement at comparable locations. Chandra et. al. [1994] concluded that right-turn factor was a function of turning radius. Equivalency factor for right-turners was defined as the ratio of through saturation flow rate to right-turn saturation flow rate at a given radius of turn. By regression method, he deduced the equivalency factor by the following equation:

$\text{Equivalency Factor} = 1 + \frac{5729.7}{r^{2.9}}$

where:

$r = \text{turning radius in meters.}$

(7) Area Type

The adjustment factor of area type in the latest HCM was based on the study conducted by the JHK & Associates in 1982. In that study, the intersection locations within a metropolitan area were classified as residential (RES), outlying commercial district (OCD), or central business district (CBD) [Zegeer 1986]. The average saturation headway for RES and OCD were 2.09 and 2.11 sec., respectively. The average saturation headway for CBD was 2.35 sec. As a result, an adjustment factor of 0.9 for intersections within CBDs was suggested. However, an analysis by Zegeer indicated that there was not
significant difference among those area types [Zegeer 1986]. Thus, no adjustment factors were proposed for area type in his study.

(8) Lane Utilization

Lane utilization factor takes into account that vehicles in exclusive through lanes do not distribute equally among the lanes available on an approach. A review of FHWA films identified a mean value for the high volume in two through lanes as 54 percent and for the high volume in three through lanes as 39 percent, which resulted in lane utilization factors of 1.08 and 1.17 for two and three through lanes, respectively [Zegeer 1986].

In Transportation Research Circular 212 [1980], a lane utilization factor of 1.05 for a two-lane approach (which represented 52.5 percent of the approach volume in the heavier lane), and 1.10 for a three-lane approach (which represented 37 percent of the approach volume in the heaviest lane) was recommended.

(9) Speed Limit

Lee’s study [1995] indicated that lower speed limits in general produced higher entering headways. The impacts of speed limit on saturation flow rate, start-up lost time and clearance lost time were summarized in Table 2-4, 2-5 and 2-6.

<table>
<thead>
<tr>
<th>Speed Limit</th>
<th>Total Headways</th>
<th>Average Headway (sec.)</th>
<th>Saturation Flow (vphg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>35</td>
<td>4614</td>
<td>2.19</td>
<td>1644</td>
</tr>
<tr>
<td>40</td>
<td>1042</td>
<td>2.15</td>
<td>1674</td>
</tr>
<tr>
<td>45</td>
<td>10726</td>
<td>2.17</td>
<td>1659</td>
</tr>
<tr>
<td>50</td>
<td>489</td>
<td>2.13</td>
<td>1690</td>
</tr>
<tr>
<td>55</td>
<td>391</td>
<td>2.11</td>
<td>1706</td>
</tr>
</tbody>
</table>
Table 2-5 Effect of Speed Limit on Start-Up Lost Time

<table>
<thead>
<tr>
<th>Speed Limit (mph)</th>
<th>Average Start-Up Lost Time (sec.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>35</td>
<td>0.60</td>
</tr>
<tr>
<td>45</td>
<td>1.16</td>
</tr>
<tr>
<td>55</td>
<td>1.55</td>
</tr>
</tbody>
</table>

Table 2-6 Effect of Speed Limit on Clearance Lost Time

<table>
<thead>
<tr>
<th>Speed Limit (mph)</th>
<th>Average Clearance Lost Time (second)</th>
</tr>
</thead>
<tbody>
<tr>
<td>35</td>
<td>1.81</td>
</tr>
<tr>
<td>45</td>
<td>1.64</td>
</tr>
<tr>
<td>55</td>
<td>0.88</td>
</tr>
</tbody>
</table>

(source: Lee 1995)

(10) Metropolitan Area Size

Transportation Research Circular 212 and the HCM did not take into account variations in driver characteristics based on city size. However, JHK's study indicated that saturation flow rates in the medium-size cities (of population from 250,000 to 400,000) were 11 percent higher than those in the largest cities (of population more than 1,000,000) [Zegeer 1986]. Surveys in Kentucky indicated that saturation flow rates in the cities with 20,000 to 50,000 persons were 8 percent lower than those in the largest cities (of population more than 100,000). Saturation flow rates in the cities with populations less than 20,000 were 17 percent lower than those in the largest cities [Agent 1982].

Zegeer studied six communities with different population level and concluded that saturation flow rates in the communities with 300,000 to 800,000 persons were identical to those in larger communities (of population more than 1,000,000). Saturation flow rates in the communities with 50,000 to 100,000 persons were 9 percent lower than in other communities [Zegeer 1986].

22
(11) *Signal Timing*

Traffic signals dramatically affect the capacity on the intersection approaches. The signal effectively regulates how much time vehicles on a given approach or set of lanes can legally move through the intersection. Previous studies indicated that cycle length and green time had impacts on start-up lost time as well as clearance lost time as shown in Table 2-7, 2-8 and 2-9 and 2-10 [Lee 1995].

**Table 2-7 Effect of Cycle Length on Start-Up Lost Time**

<table>
<thead>
<tr>
<th>Cycle Length (sec.)</th>
<th>Average Start-Up Lost Time (sec.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>60 to 90</td>
<td>1.42</td>
</tr>
<tr>
<td>90 to 120</td>
<td>1.28</td>
</tr>
<tr>
<td>120 or more</td>
<td>0.85</td>
</tr>
</tbody>
</table>

**Table 2-8 Effect of Cycle Length on Clearance Lost Time**

<table>
<thead>
<tr>
<th>Cycle Length (sec.)</th>
<th>Average Clearance Lost Time (sec.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>less than 90</td>
<td>2.09</td>
</tr>
<tr>
<td>90 to 120</td>
<td>1.62</td>
</tr>
<tr>
<td>120 to 180</td>
<td>1.62</td>
</tr>
<tr>
<td>greater than 180</td>
<td>1.29</td>
</tr>
</tbody>
</table>

**Table 2-9 Effect of Green Time on Start-Up Lost Time**

<table>
<thead>
<tr>
<th>Green Time (sec.)</th>
<th>Average Start-Up Lost Time (sec.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>30 or less</td>
<td>1.72</td>
</tr>
<tr>
<td>30 to 60</td>
<td>1.20</td>
</tr>
<tr>
<td>60 or more</td>
<td>1.05</td>
</tr>
</tbody>
</table>
Table 2-10 Effect of Green Time on Clearance Lost Time

<table>
<thead>
<tr>
<th>Green Time (sec.)</th>
<th>Average Clearance Lost Time (sec.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>less than 30</td>
<td>2.09</td>
</tr>
<tr>
<td>30 to 60</td>
<td>1.62</td>
</tr>
<tr>
<td>greater than 60</td>
<td>1.35</td>
</tr>
</tbody>
</table>

(source: Lee, 1995)

(12) Environmental Conditions

Previous studies indicated that light and weather conditions had impacts on capacity at signalized intersections. Branston’s results concluded that light condition had impacts on saturation flows at signalized intersections and were listed in the Table 2-11 [Branston 1979].

Table 2-11 Dry-Weather, Peak-Period Straight-Through Saturation Flows

<table>
<thead>
<tr>
<th>Site</th>
<th>Lane Width (ft)</th>
<th>Straight-through Saturation Flow (pcphgpl)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Daylight</td>
</tr>
<tr>
<td>1</td>
<td>11.8</td>
<td>1771</td>
</tr>
<tr>
<td>2</td>
<td>9.8</td>
<td>1757</td>
</tr>
<tr>
<td>3</td>
<td>10.8</td>
<td>1767</td>
</tr>
<tr>
<td>4</td>
<td>14.1</td>
<td>2007</td>
</tr>
<tr>
<td>5</td>
<td>14.1</td>
<td>2092</td>
</tr>
</tbody>
</table>

(source: Branston, 1979)

(13) Other Adjustment Factors

The impacts of other factors on the capacity of signalized intersections were also examined in the past. These factors included length of left-turn storage lane, link-distance to downstream intersection, one-way or two way control, time of day, pedestrian activity, driveway traffic, and queue length.

Length of left-turn storage lane was found to have effect on the capacity of left-turn movement [Johnson and Matthias]. The storage lane should be long enough to prevent
blockage of a through movement. Also the turning lane should be long enough to allow entry of turning vehicles past a line of stopped, through vehicles.

Distance to the downstream intersection also affected the saturation flow rate. When the distance to the downstream intersection was short, saturation flow rate was found to be significantly reduced. A mostly unnoticed assumption in signalized intersection analysis method was that vehicles leaving the stop line must not be impeded by a downstream queue, which would cause a drop in the speed and flow rate at which these vehicles discharged. While this was not a concern when traffic flows were light or when intersections were far apart, it began to present a problem at combinations of short links and heavy traffic [Rouphail 1996].

JHK’s study found that saturation flow rates decreased by 7.5 percent on one-way streets compared with saturation flow rates on two-way streets [Zegeer 1986]. Zegeer’s study [1986] found that the one-way saturation flow rates were 9 percent lower than those on the comparable two-way streets.

Time of day usually was signified by a.m. and p.m. peak hours. The results from Lee’s study indicated that there was not significant difference of entering headways between a.m. and p.m. peak hours [Lee 1986].

Pedestrians in the opposing street crosswalk conflicted with both right-turning and left-turning vehicles when they must turn without a protected phase. Pedestrian activities were taken into account in the HCM only to adjust the right-turn factor. In Transportation Circular 212, passenger car equivalent values for the right-turn vehicles were proposed based on pedestrian volume ranges in the conflicting crosswalk. Carstens [1971] developed a relationship between additional vehicle delay and the number of pedestrians per cycle in the crosswalk. Zegeer developed right-turn adjustment factors for pedestrian flows by deriving an equivalent percent reduction in green time available for right-turn vehicles due to the pedestrian activities [Zegeer 1986].

McCoy et. al. [1990] studied the effect of driveway traffic on saturation flow rates and developed driveway factors. The studies involved the measurement of the departure
headways between the queued vehicles in the curb lanes immediately ahead of and behind vehicles making right turns into and out of the driveways. These departure headways were compared with the prevailing headways of queues in curb lanes that were not interrupted by driveway traffic. The results indicated that driveway traffic could reduce the saturation flow rate. The reduction depended on the corner clearance of the driveway and the proportions of curb-lane volume that entered and exited the driveway. Driveway adjustment factors were presented in a table format.

Queue length also had impacts on the entering headways. The studies conducted by Agent [1982] and Lee [1986] indicated that in general, the entering headway decreased with the increasing queue length. This might have been expected because longer queues were generally associated with heavy volumes, which in turn might be associated with higher types of streets, higher speed limits, more phases in a signal setting, and a great likelihood for drivers to be in a hurry. It was recommended that vehicles after the 10th queue position should not be included in calculating saturation headway, because those drivers were relatively more aggressive which resulted in a smaller value of saturation headway.

**Driver Population Studies**

The Transportation Research Information Services (TRIS) is the Transportation Research Board’s bibliographic database. It is the most comprehensive and current source for transportation information retrievals in the nation. In this study, the TRIS database was searched to find past studies related to driver population factors for the highway capacities. No article was found related to the driver population studies at signalized intersections. A few articles were found related to the driver population studies on basic freeway sections.

The earliest study of driver population factor on freeways dated back to the early 1970s. At that time, research performed by the California Department of Transportation indicated that the capacity was much less during weekend compared to weekday due to the large number of recreational drivers in the weekend traffic. The capacity levels of 1500 vphpl were observed in the traffic streams with large number of recreational drivers.
during weekend periods where the normal capacity in the range of 2000 vphpl should be expected. Unfortunately, no concrete and precise conclusions on driver population factor were achieved at that time [CUTR 1997].

Driver population factors were developed by Sharma and are presented in the Table 2-12 [1987]. It was assumed in his research that the driver population factor for urban commuter traffic was 1.0 and for highly recreational traffic was 0.75. Five additional categories were identified between two extreme situations as shown in Table 2-12. Although he classified traffic composition based on the variation of seasonal traffic and daily traffic, the value of driver population factors for those categories were still based on purely judgment, not analytical procedure. Sharma [1994] also indicated that the monthly factor for different roads varied greatly.

<table>
<thead>
<tr>
<th>Traffic Stream Type</th>
<th>Driver Population Factor $f_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Urban Commuter</td>
<td>1.0</td>
</tr>
<tr>
<td>Regional Commuter</td>
<td>0.95</td>
</tr>
<tr>
<td>Regional Recreational &amp; Commuter</td>
<td>0.90</td>
</tr>
<tr>
<td>Inter-Regional</td>
<td>0.85</td>
</tr>
<tr>
<td>Long Distance</td>
<td>0.85</td>
</tr>
<tr>
<td>Long Distance and Recreational</td>
<td>0.80</td>
</tr>
<tr>
<td>Highly Recreational</td>
<td>0.75</td>
</tr>
</tbody>
</table>

(source: Sharma, 1987)

Brilon [1996] studied the speed-flow relationship on German autobahns and found that the lowest average speed occurred in summer with predominantly leisure traffic, whereas the highest speeds occurred during winter and early spring. Speed and capacity were also found lower during Sundays and holidays.

A research study was performed by CUTR in 1996 to estimate the capacity reduction on basic freeway sections due to the behavior of unfamiliar drivers [CUTR 1997]. Driver population factors for basic freeway sections were developed based on the monthly factor and daily factor. Monthly factor and daily factor indicated the seasonal variation of traffic.
and were able to represent driver population indirectly. Another study [Muranyi] also indicated hourly factor, daily factor, weekly factor exhibited a high degree of regularity. Based on his study, it was reasonable to assume that the variations among these factors were due to the existence of non-regular drivers.

In the HCM 1997 update, it is indicated that for recreational traffic streams, capacities are observed to be as much as 20 percent lower than for commuter traffic traveling on the same section of freeways. Free-flow speed is not significantly affected by non-local drivers. Driver population adjustment factors for basic freeway sections in HCM range from 1.0 to 0.85. No detailed information is given in HCM about how to choose the factors from this range. Furthermore, there are no adjustment factors to account for the impacts of non-local drivers on the capacity of other facilities.

As indicated previously, the driving behavior of non-local drivers are different from local drivers. When a significant number of non-local drivers are in the traffic, the capacity of freeways is reduced significantly. For a state like Florida, at intersections located near major tourist attraction sites, the level of non-local drivers is very high. These non-local drivers may also cause the capacity reduction at the signalized intersections. Therefore, it is advisable to examine and quantify the impacts of non-local drivers on the capacity of signalized intersections, and include driver population adjustment factor for the capacity analysis at signalized intersections.
CHAPTER 3: METHODOLOGY

Capacity Estimation

In order to make our results compatible and applicable to the HCM capacity analysis procedures, the analytical methodology in the HCM was adopted in this research. In the HCM, the capacity at a signalized intersection is estimated by the following equation:

\[ c_i = s_i \times \frac{g_i}{C} = s_i \times \frac{(G + Y + AR - t_a - t_c)}{C} \]

where:

- \( c_i \) = capacity of lane group i, vph;
- \( s_i \) = saturation flow rate for lane group i, vphg;
- \( C \) = cycle length, sec.;
- \( G \) = green time for each cycle, sec.;
- \( Y \) = yellow change interval, sec.;
- \( AR \) = all-red interval, sec.;
- \( t_a \) = start-up lost time, sec.;
- \( t_c \) = clearance lost time, sec.; and
- \( g_i \) = effective green time, \( g_i = G + Y + AR - t_a - t_c \), sec.

Of all the variables, signal timing is not affected by the non-local driver population level. Only saturation flow rate, start-up lost time and clearance lost time may be affected by the non-local driver population level. Saturation flow rate, start-up lost time and clearance lost time were determined based on the discharge headway data collected in the field. The discharge headway of the first vehicle is the time interval between the start of green signal and the time when the rear axle of the first vehicle crosses the stop line. The discharge headway of vehicles in other queue positions is measured by the elapsed time between the rear axle of the two consecutive vehicles passing stop line.
Saturation flow rate is the reciprocal of saturation headway. In HCM, saturation headway is calculated by averaging the discharge headway of the 5\textsuperscript{th} queued vehicle to the last queued vehicle. It is expressed as:

\[ h_s = \frac{\sum_{i=1}^{m} \sum_{j=1}^{n_i} h_{ij}}{\sum_{i=1}^{m} (n_i - 4)} \]

where:

- \( h_s \) = saturation headway, sec.;
- \( h_{ij} \) = discharge headway of \( j \textsuperscript{th} \) queued vehicle in the cycle \( i \), sec.;
- \( n_i \) = vehicle queue position in the cycle \( i \), \( n_i > 4 \); and
- \( m \) = total number of cycles during an observation period.

The above equation implies that the stable queue starts from the 5\textsuperscript{th} queued vehicle.

Start-up lost time is a function of the total discharge headways of the first four queued vehicles (HFF) and saturation headway. Start-up lost time is calculated by the following equation:

\[ t_{stl} = \sum_{i=1}^{4} h_i - 4 \times h_s = \text{HFF} - 4 \times h_s \]

where:

- \( t_{stl} \) = start-up lost time, sec.;
- \( h_i \) = discharge headway of the \( i \textsuperscript{th} \) vehicle in the queue, sec.; and
- \( \text{HFF} \) = total discharge headways of the first four queued vehicles, sec.

Clearance lost time is the unused portion of change-and-clearance interval. Since the clearance (red) interval cannot be used by any vehicle in Florida, the clearance lost time actually is the unused yellow interval plus all-red interval. The average clearance lost time for the studied movement is computed by the following equation:
It is assumed in the above equation that all-red interval is constant for a given movement and is not affected by the non-local driver population levels. Only the unused portion of yellow change interval might be affected by the non-local driver population levels. The unused portion of the yellow interval is defined as yellow lost time. By definition, yellow lost time is calculated by the following equation:

\[ t_{yl} = \frac{1}{m} \sum_{i=1}^{m} (Y - EU_{i}) \]

where:
- \( t_{yl} \) = average yellow lost time for the observed movement, sec.
- \( Y \) = yellow change interval, sec.;
- \( EU_{i} \) = the portion of yellow interval being used, sec.;
- \( AR \) = all-red interval, sec.;
- \( i \) = the sequence of the cycle; and
- \( m \) = total number of cycles during an observation period.

Since yellow intervals are not always the same for all the studied sites, it will be meaningless to compare yellow lost time at two locations with different yellow interval. To avoid such impact, a new variable \( PYU \), the percentage of yellow interval unused during the observation period, was introduced in this research to reflect the non-local drivers' impact on the usage of yellow interval. \( PYU \) is calculated by the following equation:

\[ PYU = \frac{t_{yl}}{Y} \times 100\% \]

In this research, for the purpose of the simplicity, saturation headway, HFF, start-up lost time and \( PYU \) were stated as capacity parameters. As indicated early, capacity parameters were calculated based on the discharge headway data collected in the field.
Since driver population factors were derived by quantifying the impacts of non-local drivers on these capacity parameters, it would be very critical to accurately determine the non-local driver population levels at studied signalized intersections.

**Determination of Non-Local Driver Population Level**

Non-local drivers refer to those who are not familiar with the studied facilities. However, it is hard to define the familiarity with the facility by an objective criterion. One possible way is to use the average distance from home to the study site to distinguish between local drivers and non-local drivers. Average distance from home to the studied site could be computed based on the survey information collected in the field. For example, if the average distance is 15 miles, the people living inside 15-mile of the studied site are considered local drivers. One limitation is that such a concept is not easily applied in practice since the information of non-local driver population level based on such an arbitrary boundary usually is not available to the transportation practitioners. In order to make the conclusion of this research applicable to the practical engineers, county boundary was used as a criterion to define non-local drivers. Non-local drivers were defined as those living outside the county boundary of the study site.

Driver's home location could be determined either by the license plate, or by zipcode information. License plates could be collected by video cameras installed in the field, and zipcode information could be obtained by the roadside interviews. The advantage of video camera method is that driver population information and discharge headway data could be collected simultaneously without disturbing the traffic. However, this method is very costly and time-consuming. In addition, it is difficult to identify residential locations from license plates. Previous CUTR experience revealed that many autos are registered to locations different from the driver residence. Consequently, the estimation of non-local driver population levels will be misrepresented. The other limitation is that the accuracy of discharge headway data collected by video cameras rely on many factors such as installation position and angle of the camera. With the consideration of practical feasibility and available resources, the home location of a driver was determined in this research by the zipcode information, which was obtained from roadside interviews.
However, one limitation of roadside interview method is that the discharge headway data and zipcode information have to be collected separately since drivers’ behavior might be disturbed by the interviews. For each observation, there is a trade-off of the time between the collection of discharge headway data and the zipcode survey. The more time spent to collect headway data, the less time for the zipcode survey. Thus, a well planned time schedule for the data collection would be necessary.

In this research, the minimum time required for the collection of discharge headway was determined based on the past studies. Research by Stokes [1988] indicated that a sample size of fifteen saturated phases per approach was adequate to provide a reliable saturation headway at 95% confidence level. Thus, if the cycle length is 150 seconds, then, at least 40-minute should be spent in the field to collect discharge headway data in order to obtain a reliable saturation headway.

The minimum time required for the roadside interviews in the project was determined by the sample test. A 2-hour sample survey was performed at one study site. The cycle length of this site was 2-min. A total of 56 effective cycles were collected during the 2-hour interval. These 56 cycles were divided into 8 groups with 15-minute a group. Based on the county boundary definition, the percentage of non-local drivers of each cycle was computed from zipcode information. The one-way ANOVA test was performed to examine whether the mean value of each group was the same. The null hypothesis $H_0$ was that the mean of non-local driver population level in each group was the same. A confidence level of 95% ($\alpha=0.05$) was chosen for this test. The principal of ANOVA table is to compare the $F$ and $F_{crit}$. If $F>F_{crit}$, the null hypothesis should be rejected. Otherwise, the hypothesis cannot be rejected. For this sample test, $F$ ratio was computed as 1.08 and $F_{crit}$ at 5% level of significance was 2.21. Since $F<F_{crit}$, it was concluded that the distribution of non-local driver population level (NPL) in 15-min was not significantly different from that in the 2-hour peak period at 5% level of significance. Therefore, the population level deduced from 15-min survey were sufficient to represent the population level of the peak two-hour traffic.
According to the above analysis, two-hour field study was planned for each observation in order to obtain enough saturated phases. Of each hour, the mid 20-min was used for roadside interviews, and the remaining 40-min for the collection of discharge headways. By splitting the interviews into two parts, any possible variation of non-local population levels between the two hours could be minimized.

The zipcodes collected by roadside interviews were stored in a database file and geocoded through Maptitude, a GIS (Geographic Information System) software. The location of surveyed drivers could be easily identified on the map based on the geocoded zipcodes. The county boundary map of each study site was also prepared by Maptitude. As indicated earlier, drivers living inside the county boundary of the study site were defined as local drivers in this research. By overlaying the zipcode maps on the county boundary maps in Maptitude, the number of drivers coming within the county $n_{\text{local}}$ and the total number of surveyed drivers $n_{\text{total}}$ were computed. Then, the non-local driver population level NPL was calculated by the following equation:

$$
\text{NPL} = 1 - \frac{n_{\text{local}}}{n_{\text{total}}}
$$

Once the capacity parameters and NPL were calculated from field data, driver population factors were developed by examining the relationships between the capacity parameters and the corresponding NPL.

**Development of Driver Population Factor Tables**

For each observation period (2 hours), capacity parameters, including saturation headway, HFF (total discharge headways of the first four queued vehicles), start-up lost time and PYU (percentage of yellow lost time unused), and their corresponding non-local driver population level (NPL) were calculated. A raw database with the information derived from each observation period was used to analyze the relationships between the capacity parameters and their corresponding NPL. An example of the format of raw database was presented in Figure 3-1.
Figure 3-1 Raw Database to Estimate Driver Population Factor at Signalized Intersections

Statistical tests, such as ANOVA test and t-test, were performed to examine if the capacity parameters under different non-local driver population levels were significantly different. With the assumption that the results were statistically significant, statistical models were developed to quantify such impacts. In these models, the capacity parameters were expressed as a function of NPL:

\[ CP_{NPL} = f(NPL) \]

where:

- \( CP_{NPL} \) = capacity parameters under a given non-local driver population level;
- \( f() \) = function; and
- \( NPL \) = non-local driver population level.

Driver population factors were calculated by the following equation:

\[ f_{\text{pop}(NPL)} = \frac{CP_{NPL}}{CP_0} \]

where:

- \( f_{\text{pop}(NPL)} \) = driver population factor under a given non-local driver population level;
- and
- \( CP_0 \) = capacity parameters calculated based on the data of all local drivers.
Two types of population factor tables were established in this research and the formats are presented in Figure 3-2 and Figure 3-3. Driver population factors in Figure 3-2 are recommended in terms of non-local driver population level. Driver population factors in Figure 3-3 are recommended in terms of area type.

<table>
<thead>
<tr>
<th>Non-Local Driver Population Level</th>
<th>Driver Population Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
</tr>
<tr>
<td>10%</td>
<td></td>
</tr>
<tr>
<td>20%</td>
<td></td>
</tr>
<tr>
<td>...</td>
<td></td>
</tr>
<tr>
<td>100%</td>
<td></td>
</tr>
</tbody>
</table>

*Figure 3-2 Driver Population Factors in terms of Non-Local Driver Population Level*

<table>
<thead>
<tr>
<th>Area Type</th>
<th>Driver Population Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residential Area</td>
<td></td>
</tr>
<tr>
<td>Business Area</td>
<td></td>
</tr>
<tr>
<td>Shopping Area</td>
<td></td>
</tr>
<tr>
<td>Recreational Area</td>
<td></td>
</tr>
</tbody>
</table>

*Figure 3-3 Driver Population Factors in terms of Area Type*

Once the driver population factor tables were developed, capacity sensitivity analysis was performed at a typical intersection. The purpose of such analysis was to examine how the capacity and Level of Service (LOS) varied due to the change of non-local driver population levels. HCM analysis procedure for signalized intersections was applied for this purpose.
Other Adjustment Factors

In addition to driver population adjustment factors, other factors, such as geometric factors, traffic composition factors, operation factors and environmental factors, also had significant impacts on the capacity of signalized intersections. In the field studies, the impacts of these other factors may obscure the non-local drivers’ impacts on the capacity estimation. Therefore, necessary measurements should be taken to minimize or eliminate these other impacts.

One possible solution to minimize the impacts of the other factors was to avoid those "non-ideal" conditions as much as possible. This could be accomplished by carefully selecting study sites. For example, all the candidate intersections should have wide lanes and no lateral obstructions to eliminate the impacts of narrow lane width; vehicles in the traffic stream should be mainly passenger cars to minimize the impacts of heavy vehicles; field observation should be conducted in good weather conditions and during the daytime; and no bus stops or parking were near the studied intersections.

However, if those "non-ideal" conditions could not be eliminated by the experimental design, adjustment factors included in the HCM capacity analysis procedure were adopted to adjust those impacts before the relationship of capacity parameters and non-local driver population level was analyzed. Under such situation, saturation flow rate estimated from the field data should be adjusted by applying adjustment factors to meet all the requirements of ideal conditions except for the driver population conditions. Adjusted saturation flow rate $S_{\text{adjust}}$ can be calculated as:

$$S_{\text{adjust}} = \frac{S_{\text{field}}}{F}$$

where:

- $S_{\text{field}}$ = saturation flow rate estimated by field data, vphgpl;
- $S_{\text{adjust}}$ = saturation flow rate under ideal conditions except driver population conditions, pcpvhgpl;
- $F$ = adjustment factors to adjust those “non-ideal” conditions.

Such adjusted saturation flow rate were used to develop driver population factor tables.
CHAPTER 4: DATA COLLECTION

As indicated earlier, driver population factors were developed by examining the relationships between the capacity parameters and corresponding non-local driver population level. Capacity parameters were estimated based on the discharge headway data, and non-local driver population level was estimated based on the zipcode information. Discharge headway data and zipcode information were collected in the field. Therefore, data collection was the key to this research study. Several important aspects of the experimental design of data collection were described below.

The first one was the study purpose. The main objective of this study was to evaluate the impacts of non-local drivers on the capacity at signalized intersections. Mathematical models were developed to quantify such impacts. In order to establish reliable mathematical models, the study sites should cover a wide range of non-local driver population levels, and have high traffic demand to make capacity study possible. The second one was the methodology for data collection, a key factor that would affect data quality. Discharge headway data could be collected either by manual methods, such as using laptop computer, sound recorder, and digital time recorder, or by automatic methods, such as using video recorder, digital video recorder, and computerized data system. Based on the data requirements and the available resources, a laptop computer method was adopted to collect discharge headway data.

Development of Software for Data Collection

Since discharge headway data were recorded by a laptop computer in this project, proper software should be applied. The candidate software should have the ability of providing the information that is necessary in the estimation of capacity parameters. More specifically, the following information should be recorded: signal timing, vehicle discharging headway, queue length of each cycle, physical information of observed intersections, and observation period. Some abnormal situations, such as the first queued vehicle passing the stop line when it stopped, the left-turn vehicle obstructing the ongoing through movement, vehicles passing the intersection during the red interval, and
left-turn vehicles making U-turn, should also be identified. Since existing commercial software packages do not contain all the information needed for this study, a program developed by the research team was applied to fulfill the tasks. This program was written in Visual Basic version 5.0 and the code of this program is given in Appendix A. This program was designed to execute under the Windows 95/98 environment. The interface of this program was presented in Figures 4-1 and 4-2.

![Figure 4-1 Physical Information of Data Collection](image1)

![Figure 4-2 Interface of Traffic Data Collection Window](image2)
All the physical information, such as geometric and movement information, was input in Figure 4-1. Specifically, the following record items were included:

- Observation Time—AM or PM;
- Observed Street—The name of the street that is being studied;
- Intersected Street—The name of the crossing street;
- Movement—Studied movement, left-turn or through movement;
- Approach—Northbound, southbound, eastbound or westbound.

After the input of all the information is complete, clicking OK will direct the program to the next window. The second window depends on which movement has been selected. If left-turn movement is selected, the left-turn movement window will be popped up. Otherwise, the through movement window will be shown. The keyboard operation required by the left-turn movement is almost the same as through movement. The interface of data collection for both movements is presented in Figure 4-2. In Figure 4-2, function keys are designed to record the change of phase and the action of each individual vehicle. These function keys are defined as:

- Key J: Pressed when the signal turns green, and the start time of the green interval will be automatically recorded;
- Key K: Pressed when the signal turns yellow, and the start time of the yellow interval will be automatically recorded;
- Key L: Pressed when the signal turns red, and the start time of the red interval will be automatically recorded;
- Key D: Pressed when the rear axle of a passenger car crosses the stop line;
- Key S: Pressed when the rear axle of a heavy vehicle crosses the stop line. Heavy vehicles are defined as vehicles with more than two axles;
- Key A: Pressed when the rear axle of a RV (recreational vehicle) crosses the stop line; and
- Key U: Pressed after one of the keys D, S, or A is pressed to record a U-turn movement. This function key is only available for left-turn movement. Vehicles making U-turn are excluded from the data analysis.

The reason of selecting keys J, K, L, D, S, A and U for input is that they are convenient keys in the keyboard. When the input procedure of each cycle is completed (after Key L is pressed), two questions will be asked for both left-turn movement and through movement:

- Is the first vehicle passing the stop line? If yes, the discharge headway of the first vehicle of this cycle is eliminated from the data analysis.

- Are there any vehicles using the red interval? If yes, the yellow lost time of this cycle will be equal to the yellow interval.

Besides these two questions, another question is designed exclusively for the through movement:

- Are there any left-turn vehicles blocking the on-going through movement at the start of the green phase? If yes, the data of this cycle will be eliminated from the data analysis.

At last, the input of the queue length will be requested. This information is very important since only queued vehicles are included in the estimation of saturation flow rate and start-up lost time of a signalized intersection. Whenever the abnormal situation occurs during the survey, the key “Esc” on the keyboard or “False Cycle” shown in Figure 4-2 will be pressed to eliminate the abnormal cycle from the database. “Another Approach” function in Figure 4-2 is designed to perform another field study at other approaches without restarting the program.

Of each observation, three data files are created: origin.txt, headway.txt and sneaker.txt. The origin.txt file has all the raw information, such as location, observation duration, movement, approach, the computer-clock time of each vehicle passing the stop line, the discharge headway data, vehicle type, vehicle queue position, signal timing, etc. The
headway.txt file only includes the discharge headways of the queued vehicles and their associated vehicle types. The sneaker.txt file has the information of the green and yellow interval as well as the time interval between the start of yellow signal and the rear axle of last vehicle passing stop line. The information in all the files is recorded and processed on a cycle basis. In this project, capacity parameters were estimated based on the following two files: headway.txt and sneaker.txt.

A brief summary of how to use this program is given as follows:

- Double click the icon of the executive file headway.exe to run the program;
- Provide all the information required in the first Window. Then click OK into the second Window. Get ready to record traffic data;
- Press a corresponding key (J, K, L) when the indication of the signal changed. Press a corresponding key (D, S, or A) when the rear axle of a vehicle passing the stop line;
- Answer all the questions by pressing the Key “Y” (Yes) or Key “N” (No); and
- Input of the queue length. Get ready for the next cycle.

Sites Selection

In order to make data collection of this research feasible and successful, the following factors were considered in the selection of study locations:

- Heavy peak hour volumes were necessary at all the study sites since the saturated condition was achieved after the 4th queued vehicle in HCM;
- Exclusive left-turn lanes with protected left-turn phase were required to conduct the study of the left-turn movement;
- Except driver population conditions, other conditions should match the ideal requirements in HCM as much as possible, such as 12-ft lane width, level approach grade, very few heavy vehicles in the traffic, no parking or bus stop within the intersection, very few pedestrian conflicts, and good weather;
- Study sites should cover a wide range of non-local driver population level;
- Only right-angle signalized intersections were considered; and
All the sites had an ideal spot for surveyors to conduct the survey. The purpose was to minimize the disturbance of surveyors on the traffic flow while recording the discharge headway data.

More than 50 candidate intersections in Hillsborough, Pinellas, Osceola and Orange Counties, Florida were examined. Based on the above criteria, twelve signalized intersections were selected. These intersections covered four different land use characteristics: residential, business, shopping, and recreational. Sixteen movements from the 12 intersections were singled out to perform the proposed study. The selected intersections and movements are listed in Table 4-1.

Table 4-1: List of Studied Signalized Intersections

<table>
<thead>
<tr>
<th>SITES</th>
<th>AREA TYPE</th>
<th>APPROACH</th>
<th>MOVE</th>
<th>COUNTY</th>
<th>LANE WIDTH (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bruce B. Downs Blvd. @ Tampa Palms</td>
<td>Residential</td>
<td>Southbound</td>
<td>Through</td>
<td>Hillsborough</td>
<td>12</td>
</tr>
<tr>
<td>Sheldon Rd. @ Waters Ave.</td>
<td>Residential</td>
<td>Northbound</td>
<td>Through</td>
<td>Hillsborough</td>
<td>12</td>
</tr>
<tr>
<td>Bruce B. Downs Blvd. @ Tampa Palms</td>
<td>Residential</td>
<td>Northbound</td>
<td>Left-Turn</td>
<td>Hillsborough</td>
<td>12</td>
</tr>
<tr>
<td>Sheldon Rd. @ Waters Ave.</td>
<td>Residential</td>
<td>Southbound</td>
<td>Left-Turn</td>
<td>Hillsborough</td>
<td>11</td>
</tr>
<tr>
<td>Cypress St. @ West Shore Blvd.</td>
<td>Business</td>
<td>Eastbound</td>
<td>Through</td>
<td>Hillsborough</td>
<td>12</td>
</tr>
<tr>
<td>Dale Mabry Hwy. @ Kennedy Blvd.</td>
<td>Business</td>
<td>Southbound</td>
<td>Through</td>
<td>Hillsborough</td>
<td>12</td>
</tr>
<tr>
<td>West Shore Blvd. @ Boy Scout</td>
<td>Business</td>
<td>Northbound</td>
<td>Left-Turn</td>
<td>Hillsborough</td>
<td>12</td>
</tr>
<tr>
<td>Columbus @ Dale Mabry Hwy.</td>
<td>Business</td>
<td>Westbound</td>
<td>Left-Turn</td>
<td>Hillsborough</td>
<td>12</td>
</tr>
<tr>
<td>SR 580 @ US 19</td>
<td>Shopping</td>
<td>Eastbound</td>
<td>Through</td>
<td>Pinellas</td>
<td>12</td>
</tr>
<tr>
<td>SR 580 @ US 19</td>
<td>Shopping</td>
<td>Westbound</td>
<td>Left-Turn</td>
<td>Pinellas</td>
<td>12</td>
</tr>
<tr>
<td>SR 60 @ Brandon Mall</td>
<td>Shopping</td>
<td>Westbound</td>
<td>Left-Turn</td>
<td>Hillsborough</td>
<td>11</td>
</tr>
<tr>
<td>SR 192 @ SR 535</td>
<td>Recreational</td>
<td>Westbound</td>
<td>Through</td>
<td>Osceola</td>
<td>12</td>
</tr>
<tr>
<td>International Dr. @ Kirkman Rd.</td>
<td>Recreational</td>
<td>Eastbound</td>
<td>Through</td>
<td>Orange</td>
<td>12</td>
</tr>
<tr>
<td>Sand Lake Rd. @ John Young Pkwy</td>
<td>Recreational</td>
<td>Eastbound</td>
<td>Through</td>
<td>Orange</td>
<td>12</td>
</tr>
<tr>
<td>Central FL Pkwy @ International Dr.</td>
<td>Recreational</td>
<td>Westbound</td>
<td>Left-Turn</td>
<td>Orange</td>
<td>12</td>
</tr>
<tr>
<td>SR 535 @ SR 192</td>
<td>Recreational</td>
<td>Southbound</td>
<td>Left-Turn</td>
<td>Osceola</td>
<td>12</td>
</tr>
</tbody>
</table>
Data Collection

Two types of data were collected in this research: discharge headways and driver zipcodes. Capacity parameters were obtained from discharge headway data, and non-local driver population level was determined by the zipcode information. As mentioned before, discharge headways were recorded by a laptop computer, and zipcode information was collected by roadside interviews.

Data collection was performed during the peak hours of a day. The morning peak hour was from 7:00 a.m. to 9:00 a.m., and the afternoon peak hour was from 4:00 p.m. to 6:00 p.m. for the residential and business areas. For the shopping and recreational areas, the morning peak hour was from 10:00 a.m. to 12:00 p.m., and the afternoon peak hour was from 3:00 p.m. to 5:00 p.m. The weekdays, including Monday afternoon, Tuesday, Wednesday, and Thursday, were used to collect data at residential and business locations. Most of data collection at shopping and recreational areas were conducted on Saturday and Sunday because the heaviest traffic situation usually occurred on these days. However, in order to cover a wide range of non-local driver population level, data were also collected at recreational areas during weekdays. Monday morning and Friday were excluded from the data collection due to the fluctuation in traffic patterns and atypical traffic demand on these days.

As described in the chapter of methodology, in order to get enough saturated phases, a 2-hour field survey was conducted for each observation. Within each hour, the mid 20-minute was assigned for roadside interviews, and the remaining 40-minute was for the collection of discharge headways. Each observation was conducted by a two-person team. During the collection of discharge headways, one person was in charge of operating the computer program to record discharge headway of each vehicle. The other person recorded queue length and reported it to his/her partner at the end of each cycle. The information of queue length was very important since the saturation headway and start-up lost time were related only to the queued vehicles. The other person was also responsible to remind the partner about the start of yellow and red interval in case the partner missed the signal change when recording the headways. Any abnormal situation occurred during the observation was also noted for further analysis.
Discharge headways were collected at sixteen lanes from twelve intersections, including 8 exclusive left-turn lanes and 8 exclusive through lanes. Each lane was studied for at least 16 hours. A total of 174 hours were spent in the field for the collection of discharge headway. Over 36,660 passenger car discharge headways were processed to estimate the capacity parameters. Of all the capacity parameters, saturation headway, HFF (total headways of the first four queued vehicles) and start-up lost time were calculated based on the discharge headways in the file of headway.txt. PYU was computed from the file of sneaker.txt. The analytical methodology presented in Chapter 3 was applied to calculate the capacity parameters.

Both surveyors participated in the roadside interviews that were conducted during the red interval of each cycle. Interviews stopped before the signal turned green. A “SURVEY CREW AHEAD” sign was posted at the place close to the average queue length. Such sign was taken away from the field when the discharge headways were collected to avoid disturbing the normal driving behavior. Each queued driver was inquired about his/her zipcode information during the red interval. Usually 8 to 10 people were surveyed during a red interval. In order to improve the efficiency, one surveyor interviewed with the first half of the queued drivers, and the other interviewed the remaining half of queued drivers. The survey form that was used for the study is presented in Appendix B. During the red interval, each surveyor, who had a clipboard and pen in hand, went to the stopped driver and asked about the zipcode information. Sometimes, the surveyors had to explain the study purpose to the questioned drivers. For those having zipcodes, their zipcodes were recorded in the rows of zipcode. For those who did not respond to the surveyors, they were classified as FL (Florida) or Non-FL by their license plates. The information of the non-response drivers was used to compute the response rate. Moreover, all Non-FL records were considered non-local drivers in this study. The FL non-response records were excluded from the calculation of non-local driver population level. The foreigners were recorded with an ‘F’ instead of with zipcode in the separate rows.

A total of 86 hours were used for roadside interviews to obtain driver zipcode information. During the 86 hours, 12,219 drivers were interviewed, among whom 1,040 individuals did not respond to the surveyors. The response rate of the survey was 91.5%.
All the zipcodes were saved as a database file and geocoded by the GIS software of Maptitude. By displaying the geocoded zipcodes on the U.S. map, the driver home locations could be easily identified. County boundary maps of the study sites were also prepared by Maptitude. As indicated in Chapter 3, non-local drivers were defined as those who lived outside the county boundary of the study site. By overlaying the zipcode maps over the county boundary maps, the number of drivers coming within the county $n_{\text{local}}$ and the total surveyed people $n_{\text{total}}$ were calculated. Then, the non-local driver population level (NPL) was computed by the equation described in Chapter 3.

The reduced capacity parameters and their corresponding non-local driver population level were presented in Appendix C. By examining the relationships between the capacity parameters and the corresponding non-local driver population level, driver population adjustment factors were developed for the capacity analysis at signalized intersections and the results were presented in Chapter 5.
CHAPTER 5: DATA ANALYSIS AND RESULTS

Data analyses were performed in this study to examine the effects of non-local driver population levels (NPL) on capacity parameters, including saturation headway, HFF, start-up lost time and PYU. One-way ANOVA tests and t-tests were applied to examine whether or not the impacts of non-local drivers on the capacity parameters were significant, and statistical models were developed to quantify the relationships between capacity parameters and NPL. Driver population factors were calculated based on the model results. Two type of driver population factor tables were recommended for the practical application. One table was based on the non-local driver population level, and the other was based on area types surrounding the intersections.

Analysis of Capacity Parameters

Capacity parameters were analyzed and summarized in three aspects: (1) capacity parameters by movements; (2) capacity parameters by land use characteristics; and (3) capacity parameters by non-local driver population levels.

(1) Capacity Parameters by Movements

Capacity parameters analyzed in this research included saturation headway, HFF, start-up lost time and PYU. Results of these capacity parameters for left-turn movements and through movements were presented in Appendix C. The summarized statistics of capacity parameters were listed in Table 5-1.

Table 5-1 Summary of Capacity Parameters by Movements

<table>
<thead>
<tr>
<th>Capacity Parameters</th>
<th>Left-Turn Movement</th>
<th>Through Movement</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Range</td>
<td>Average</td>
</tr>
<tr>
<td>Saturation Headway (sec.)</td>
<td>1.67 – 2.10</td>
<td>1.85</td>
</tr>
<tr>
<td>HFF (sec.)</td>
<td>8.41 – 10.60</td>
<td>9.36</td>
</tr>
<tr>
<td>Start-Up Lost Time (sec.)</td>
<td>1.05 – 2.80</td>
<td>1.90</td>
</tr>
<tr>
<td>PYU (%)</td>
<td>19 - 100</td>
<td>71</td>
</tr>
</tbody>
</table>
According to Table 5-1, the capacity parameters appeared to be different between the movements. The t-test was performed in this study to examine whether or not the differences of capacity parameters between the movements were statistically significant. The null hypothesis $H_0$ was that both groups had the same mean of capacity parameters. More specifically, the null hypotheses were expressed as:

$H_{01}$: The average saturation headway of left-turn movement = The average saturation headway of through movement;

$H_{02}$: The average HFF of left-turn movement = The average HFF of through movement;

$H_{03}$: The average start-up lost time of left-turn movement = The average start-up lost time of through movement; and

$H_{04}$: The average PYU of left-turn movement = The average PYU of through movement.

A confidence level of 95% ($\alpha=0.05$) was chosen for the test. The principal of t-test is to compare the t statistics and $t_{crit}$. t statistics is calculated by the following equation:

$$t = \frac{\text{mean}(x_{\text{left}}) - \text{mean}(x_{\text{through}})}{\text{variance}(x_{\text{left}} - x_{\text{through}})}$$

"x" is the variable of the capacity parameters. The value of the $t_{crit}$ is associated with the significance level and degree of the freedom, and can be obtained from the statistical table. If $t > t_{crit}$, the null hypothesis $H_0$ will be rejected, otherwise cannot be rejected. If a hypothesis is rejected, it can be concluded that the difference of the mean among the test groups is statistically significant under the given confidence level. Results of the t-test were presented in Table 5-2.

**Table 5-2 t-test of Capacity Parameters Between Left-Turn and Through Movements**

<table>
<thead>
<tr>
<th>Capacity Parameter</th>
<th>t statistics</th>
<th>$t_{crit}$ ($\alpha = 0.05$)</th>
<th>$H_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Saturation Headway</td>
<td>3.50</td>
<td>1.98</td>
<td>rejected</td>
</tr>
<tr>
<td>HFF</td>
<td>2.61</td>
<td>1.98</td>
<td>rejected</td>
</tr>
<tr>
<td>Start-Up Lost Time</td>
<td>1.09</td>
<td>1.98</td>
<td>not rejected</td>
</tr>
<tr>
<td>PYU</td>
<td>2.00</td>
<td>1.98</td>
<td>rejected</td>
</tr>
</tbody>
</table>
As indicated in Table 5-2, except for the hypothesis about start-up lost time, all other hypotheses were rejected at 5% level of significance. The following conclusions were drawn from statistical tests:

- The average saturation headway of left-turn movement was significantly different from that of through movement. The average saturation headway of left-turn movements was 1.85 second, lower than that of through movements (1.92 second). One possible reason was the difference in non-local driver population levels between the movements. The non-local driver population levels at all the studied left-turn movements was in the range of 1%-72%, however, that of through movements was in the range of 7%-92%. The other reason was that there was always a very long queue at one of the left-turn sites. Shorter discharge headway would be expected at this site because drivers in the longer queue was likely to be in a hurry to avoid a perceived lengthy delay.

- The results of HFF indicated that left-turn drivers responded to the indication of green signal more quickly (9.29sec. vs. 9.64sec.). One reason might be that the green phase for the left-turn movement was not as long as that for the through movement, which might resulted in a great likelihood for left-turn drivers to be in a hurry.

- Start-up lost time was not significantly different between the movements, which indicated that the response of the first four vehicles to the signal change was not different between the movements. The start-up lost time for left-turn movement from the field data was approximately 1.90 seconds, and for through movement was approximately 1.97 seconds. The average start-up lost time for both movement was 1.93 seconds.

- The average usage of the yellow interval for left-turn vehicles was 29%, in contrast to that for through movement 24%. The results indicated that left-turn drivers had more tendency to utilize the yellow interval than through drivers. This might contribute to the reason that left-turn drivers were more aggressive to avoid the perceived lengthy delay.
Since most capacity parameters of left-turn movement were significantly different from through movement, statistical models, which were used to quantify the relationships between the capacity parameters and NPL, were developed for left-turn movement and through movement separately.

(2) Capacity Parameters by Land Use Characteristics

The capacity parameters by land use characteristics were summarized in Table 5-3. As shown in Table 5-3, saturation headway was the highest at recreational areas, and the lowest at residential areas. That is, saturation flow rate at recreational areas was the lowest among all the area types. Since start-up lost time reflected the driver's response time to the signal change, it could be concluded from Table 5-3 that on average, drivers at recreational areas responded to the indication of green signal slower than at other areas. There was no significant difference of the usage of yellow interval among the area types.

Table 5-3 Mean of Capacity Parameters by Land Use Characteristics

<table>
<thead>
<tr>
<th>Area Type</th>
<th>Saturation Headway (sec.)</th>
<th>HFF (sec.)</th>
<th>Start-Up Lost Time (sec.)</th>
<th>PYU (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residential Area</td>
<td>1.79</td>
<td>9.18</td>
<td>2.01</td>
<td>73</td>
</tr>
<tr>
<td>Business Area</td>
<td>1.83</td>
<td>9.10</td>
<td>1.80</td>
<td>70</td>
</tr>
<tr>
<td>Shopping Area</td>
<td>1.83</td>
<td>9.02</td>
<td>1.70</td>
<td>80</td>
</tr>
<tr>
<td>Recreational Area</td>
<td>2.03</td>
<td>10.24</td>
<td>2.11</td>
<td>73</td>
</tr>
</tbody>
</table>

(3) Capacity Parameters by NPL

One-way ANOVA tests were performed to examine whether or not the impacts of non-local driver population levels (NPL) on the capacity parameters were significant. In order to perform one-way ANOVA tests, non-local driver population levels obtained from field interviews were divided into ten groups, with 10% difference in each group. The null hypothesis $H_0$ of the ANOVA tests was that the mean of all the test groups were statistically equal. More specifically, the following hypotheses were tested in this research:
\( H_05: \) The average saturation headway was the same for each population group of left-turn movement;

\( H_06: \) The average HFF was the same for each population group of left-turn movement;

\( H_07: \) The average start-up lost time was the same for each population group of left-turn movement;

\( H_08: \) The average PYU was the same for each population group of left-turn movement;

\( H_09: \) The average saturation headway was the same for each population group of through movement;

\( H_010: \) The average HFF was the same for each population group of through movement;

\( H_011: \) The average start-up lost time was the same for each population group of through movement;

\( H_012: \) The average PYU was the same for each population group of through movement.

A confidence level of 95% \((\alpha=0.05)\) was chosen for the tests. The principal of one-way ANOVA tests is to compare the value of F ratio and \( F_{\text{crit}} \) at a given level of significance \( \alpha \). F ratio is calculated by taking the “average” sum of squares due to group differences (MEAN SQUARE between) divided by the “average” sum of squares due to subject differences (MEAN SQUARE error). \( F_{\text{crit}} \) is given by a table indicating the probability of obtaining a value of F this large or larger when the null hypothesis is true. If \( F>F_{\text{crit}} \), the null hypothesis will be rejected. Results of one-way ANOVA tests are presented in Table 5-4. As indicated in Table 5-4, the impacts of non-local drivers on the saturation headway and HFF were statistically significant at 5% level of significance for both left-turn movement and through movement. The impacts of non-local drivers on the start-up lost time was significant for through movement, but not significant for left-turn movement. Start-up lost time reflected the driver’s response to the green signal. According to the current HCM procedure, start-up lost time was determined by the HFF and saturation headway. As indicated by the field data, HFF for the left-turn movement did not vary significantly with the change of population level because most left-turn drivers, either
familiar or unfamiliar with the facility, responded to the signal change quickly, which resulted in the impacts of non-local drivers on start-up lost time of the left-turn movement inconclusive. The impacts of non-local drivers on the PYU were not significant for both movements. That may be due to the fact that besides driver population factors, many other factors, such as traffic demand, arrival pattern, signal coordination, also have impacts on the usage of yellow interval.

Table 5-4 One-way ANOVA tests of Capacity Parameters among Driver Population Groups

<table>
<thead>
<tr>
<th>Capacity Parameters</th>
<th>Left-Turn Movement</th>
<th>Through Movement</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>F</td>
<td>$F_{crit}$ (α = 0.05)</td>
</tr>
<tr>
<td>Saturation Headway</td>
<td>22.03</td>
<td>2.18 reject</td>
</tr>
<tr>
<td>HFF</td>
<td>3.70</td>
<td>2.18 reject</td>
</tr>
<tr>
<td>Start-Up Lost Time</td>
<td>1.77</td>
<td>2.18 Not reject</td>
</tr>
<tr>
<td>PYU</td>
<td>0.75</td>
<td>2.18 Not reject</td>
</tr>
</tbody>
</table>

In the HCM capacity analysis procedure, the capacity at a signalized intersection is determined by the saturation headway, start-up lost time and clearance lost time. As analyzed in the above, non-local drivers only had significant impacts on the saturation headway. The impacts of non-local drives on the start-up lost time and PYU were either inconclusive or insignificant. Furthermore, in traditional capacity analysis, a single value is applied to the start-up lost time and clearance lost time to simplify the capacity analysis at signalized intersections. Adjustment factors in HCM are only applied to adjust for the saturation flow rate. Thus, in this research, driver population factors were developed only to adjust the saturation flow rate. The start-up lost time and clearance lost time were kept constant under all of the population levels. Driver population factors were derived from the statistical models.
Development of Statistical Models

Statistical tests indicated that saturation headways were significantly different between the movements and among different non-local driver population levels. Statistical models were developed to address the relationship between the saturation headway and the corresponding NPL for both left-turn movement and through movement. The saturation headway and the corresponding NPL were plotted and shown in Figures 5-1 and 5-2 for left-turn movement and through movement, respectively.

**Figure 5-1: Left-Turn Saturation Headway vs. Non-Local Driver Population Level (NPL)**

![Graph showing the relationship between saturation headway and NPL for left-turn movement. The equation is given as $y = 0.4794x + 1.7307$ with $R^2 = 0.7007$.]

**Figure 5-2: Through Saturation Headway vs. Non-Local Driver Population Level (NPL)**

![Graph showing the relationship between saturation headway and NPL for through movement. The equation is given as $y = 0.3513x + 1.7893$ with $R^2 = 0.8391$.]

53
By examining the data patterns presented in the figures, linear regression models were used to represent the relationships between saturation headway $h_s$ and the non-local driver population level NPL. The general format of linear models was as follows:

$$h_s = a + b \times NPL$$

The coefficients “a” and “b” were determined by the Least Squares Estimation. The results of linear models were presented in Table 5-5. Based on $R^2$ value and t-value of both models, it was indicated that saturation headways had a good relationship with the non-local driver population levels. Since all other factors that would affect saturation headway had been controlled or adjusted, the difference among the saturation headway was considered due to the different non-local driver population levels. Base on the linear regression models, saturation headway and saturation flow rate were calculate for each population level and presented in Table 5-6.

According to Table 5-6 when the non-local driver population level increased from 0% to 100%, the saturation flow rate decreased 22% for left-turn movement from 2080 pcphgpl to 1630 pcphgpl, and decreased 16% for through movement from 2010 pcphgpl to 1680 pcphgpl. It was also indicated in Table 5-6 that at low non-local driver population levels, the saturation flow rate of left-turn movement was higher than the that of through movement. This may contribute to the fact that left-turn drivers are more aggressive and alert at the intersections in order to avoid the perceived lengthy delay. However, at higher non-local driver population levels, the saturation flow rate was reduced more significantly for left-turn movement, meaning that non-local driver population levels had more impacts on the left-turn movement.

Table 5-5 Results of Linear Models

<table>
<thead>
<tr>
<th></th>
<th>LEFT-TURN MOVEMENT</th>
<th>THROUGH MOVEMENT</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>A</strong></td>
<td>1.73</td>
<td>1.79</td>
</tr>
<tr>
<td><strong>B</strong></td>
<td>0.48</td>
<td>0.35</td>
</tr>
<tr>
<td><strong>$R^2$</strong></td>
<td>0.70</td>
<td>0.84</td>
</tr>
<tr>
<td><strong>t-value</strong></td>
<td>11.85</td>
<td>18.27</td>
</tr>
</tbody>
</table>
Table 5-6 Saturation Flow Rate under a Given Population Level

<table>
<thead>
<tr>
<th>Non-Local Driver Population Level</th>
<th>Left-Turn Movement</th>
<th>Through Movement</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Saturation headway (sec.)</td>
<td>Saturation flow rate (pcphgpl)</td>
</tr>
<tr>
<td>0%</td>
<td>1.73</td>
<td>2080</td>
</tr>
<tr>
<td>10%</td>
<td>1.78</td>
<td>2020</td>
</tr>
<tr>
<td>20%</td>
<td>1.83</td>
<td>1970</td>
</tr>
<tr>
<td>30%</td>
<td>1.87</td>
<td>1925</td>
</tr>
<tr>
<td>40%</td>
<td>1.92</td>
<td>1875</td>
</tr>
<tr>
<td>50%</td>
<td>1.97</td>
<td>1830</td>
</tr>
<tr>
<td>60%</td>
<td>2.02</td>
<td>1780</td>
</tr>
<tr>
<td>70%</td>
<td>2.07</td>
<td>1740</td>
</tr>
<tr>
<td>80%</td>
<td>2.11</td>
<td>1710</td>
</tr>
<tr>
<td>90%</td>
<td>2.16</td>
<td>1670</td>
</tr>
<tr>
<td>100%</td>
<td>2.21</td>
<td>1630</td>
</tr>
</tbody>
</table>

Development of Driver Population Factors

In HCM, adjustment factors for saturation flow rate were developed by dividing the prevailing saturation flow rate against the “ideal” saturation flow rate. Thus, driver population adjustment factors $f_{pop}$ were estimated by the following equation:

$$f_{pop(NPL)} = \frac{SNPL}{S0\%}$$

where, $f_{pop(NPL)}$ = driver population adjustment factor for saturation flow rate; $S0\% = \text{saturation flow rate with 0\% non-local driver population level}$; and $SNPL = \text{saturation flow rate with a given non-local driver population level}.$

Based on Table 5-6, driver population adjustment factors were calculated and presented in Table 5-7. It was indicated in Table 5-7 that when non-local driver population level increased from 0% to 100%, saturation flow rate was reduced to 81% of the ideal value calculated by the current HCM procedure. Since saturation flow rate was proportional to the capacity at signalized intersections, the capacity also decreased up to 81% of the ideal value when the population level increased from 0% to 100%.
Table 5-7 Driver Population Factors Based on Non-Local Driver Population Level

<table>
<thead>
<tr>
<th>Non-Local Driver Population Level</th>
<th>$f_{pop}$ for Saturation Flow Rate</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Left-Turn</td>
</tr>
<tr>
<td>0%</td>
<td>1.0</td>
</tr>
<tr>
<td>10%</td>
<td>0.97</td>
</tr>
<tr>
<td>20%</td>
<td>0.95</td>
</tr>
<tr>
<td>30%</td>
<td>0.92</td>
</tr>
<tr>
<td>40%</td>
<td>0.90</td>
</tr>
<tr>
<td>50%</td>
<td>0.88</td>
</tr>
<tr>
<td>60%</td>
<td>0.86</td>
</tr>
<tr>
<td>70%</td>
<td>0.84</td>
</tr>
<tr>
<td>80%</td>
<td>0.82</td>
</tr>
<tr>
<td>90%</td>
<td>0.80</td>
</tr>
<tr>
<td>100%</td>
<td>0.78</td>
</tr>
</tbody>
</table>

However, the factors in Table 5-7 may not be easily used by transportation practitioners due to the fact that the exact non-local driver population level for a study site is not always available in practice. Therefore, an alternative format of driver population factors in terms of area type was developed in this study and presented in the following section.

**Development of Population Factors Based on Area Types**

As indicated earlier, in order to cover a wide range of non-local driver population levels, signalized intersections within different area types were selected in the study. Four different area types, including residential area, business area, shopping area and recreational area, were studied. In general, the area type surrounding an intersection could reflect its non-local driver population level. For example, the intersections located at residential areas would have less non-local drivers, but the intersections in a major recreational area would have much more non-local drivers. With such considerations, it would be reasonable to develop driver population adjustment factors in terms of area type. The advantage of such format is that there is no need for the detailed information of non-local driver population levels, which are not always available to the transportation...
practitioners. Based on survey results, the range of non-local driver population levels of each area type was summarized in Table 5-8. As shown in Table 5-8, the non-local driver population level ranges were different among different area types.

Table 5-8 Range of Non-Local Driver Population Levels of Each Area Type

<table>
<thead>
<tr>
<th>Area Type</th>
<th>Non-Local Driver Population Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residential Area</td>
<td>1% - 19%</td>
</tr>
<tr>
<td>Business Area</td>
<td>5% - 43%</td>
</tr>
<tr>
<td>Shopping Area</td>
<td>14% - 23%</td>
</tr>
<tr>
<td>Recreational Area</td>
<td>29% - 92%</td>
</tr>
</tbody>
</table>

The cumulative frequency distribution curves of non-local driver population levels for each area type were plotted in Figure 5-3. The 50th percentile level and the 85th percentile level of population group can be identified from Figure 5-3 and results were presented in Table 5-9. The 50th percentile level is defined as the population level that divides the distribution into equal halves, i.e., there are as many field observation that have population level higher than this value as that lower than this value. The 85th percentile level means that 85 percentage of field observation with population level less than this value.

Figure 5-3 Distribution of Non-Local Drivers of Each Area Type
Table 5-9 Distribution of Non-Local Driver Population Level

<table>
<thead>
<tr>
<th>Area Type</th>
<th>Minimum</th>
<th>Maximum</th>
<th>50th percentile</th>
<th>85th percentile</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residential Area</td>
<td>0</td>
<td>20</td>
<td>10</td>
<td>15</td>
</tr>
<tr>
<td>Business Area</td>
<td>5</td>
<td>45</td>
<td>15</td>
<td>30</td>
</tr>
<tr>
<td>Shopping Area</td>
<td>10</td>
<td>25</td>
<td>15</td>
<td>20</td>
</tr>
<tr>
<td>Recreational Area</td>
<td>30</td>
<td>95</td>
<td>65</td>
<td>85</td>
</tr>
</tbody>
</table>

As shown in Figure 5-3, non-local driver population levels were different among different area types. One-way ANOVA test was performed to examine whether or not the difference among the area types was statistically significant. The null hypothesis $H_0$ was that the non-local driver population level was the same among the different land use. The $F$ ratio of the one-way ANOVA test was 146.28, and the $F_{\text{crit}}$ was 2.68 at 5% level of significance. Since $F>F_{\text{crit}}$, it was concluded that non-local driver population levels were significantly different among the different land use.

Given the range of non-local driver population levels of each area type shown in Table 5-9, the corresponding range of driver population adjustment factors could be assigned to each area type based on the data shown in the average column of Table 5-7. The driver population adjustment factors in terms of area type were presented in Table 5-10. In general, for residential areas, the analyst should select 1.0 of $f_{\text{pop}}$, unless there are shopping malls nearby. In that case, a smaller value, but within the range, would be applied. Central business district (CBD) areas should use the lower $f_{\text{pop}}$ value for the business areas. For the recreational areas with some residential communities nearby, the higher $f_{\text{pop}}$ value should be selected, and for the recreational area close to the major attractions, such as Kissimmee, the lower $f_{\text{pop}}$ value should be used. It was indicated in Table 5-10 that due to the behavior of non-local drivers, the saturation flow rate was reduced as much as 18% for the recreational area and 10% for other area types.
Table 5-10 Driver Population Factors $f_{pop}$ Based on Area Types

<table>
<thead>
<tr>
<th>Area Type</th>
<th>$f_{pop}$ for saturation flow rate</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Range</td>
</tr>
<tr>
<td>Residential Area</td>
<td>1.0 - 0.95</td>
</tr>
<tr>
<td>Business Area</td>
<td>0.99 - 0.90</td>
</tr>
<tr>
<td>Shopping Area</td>
<td>0.97 - 0.94</td>
</tr>
<tr>
<td>Recreational Area</td>
<td>0.93 - 0.82</td>
</tr>
</tbody>
</table>

The $f_{pop}$ values included in either Table 5-7 or Table 5-10 could be used with other adjustment factors in HCM to perform capacity analysis at signalized intersections. Due to the fact that $f_{pop}$ developed in this research reflects the impacts of area type, the adjustment factor of area type $f_a$ used in the current HCM procedure would be excluded from capacity analysis when the $f_{pop}$ is applied.

Case Study

A case study was performed in this research to examine the change of capacity and level of service (LOS) at a signalized intersection due to the introduction of driver population adjustment factors into the capacity analysis procedure. LOS is a measurement of quality of flow. At signalized intersections, LOS is based on the average control delay per vehicle for various movements within the intersection. In order to perform capacity analysis at signalized intersections, one of the study sites, the intersection of S.R.192 & S.R.535, was selected for the case study. Intersection S.R.192 & S.R.535 is a T-intersection. It is located in Osceola County, Florida, and close to the Orlando Disney World. Many tourists drive through this intersection each day. The geometric conditions of this intersection are presented in Figure 5-4. All the lanes are 12 feet wide with 0% grade. During the survey peak hour, most of traffic are passenger cars. There are no parking lanes within the 250-ft of the intersection. The traffic volume counts at this intersection are summarized in Table 5-11. The traffic signal phase plan is shown in Table 5-12.
Table 5-11 Peak-Hour Traffic Volume (vph) at Intersection S.R.192 & S.R.535

<table>
<thead>
<tr>
<th></th>
<th>East-bound</th>
<th></th>
<th>West-bound</th>
<th></th>
<th>South-bound</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Through</td>
<td>Left</td>
<td>Right</td>
<td>Through</td>
<td>Left</td>
</tr>
<tr>
<td>10:00am</td>
<td>311</td>
<td>85</td>
<td>283</td>
<td>116</td>
<td>116</td>
</tr>
<tr>
<td>10:15am</td>
<td>311</td>
<td>85</td>
<td>292</td>
<td>151</td>
<td>126</td>
</tr>
<tr>
<td>10:30am</td>
<td>322</td>
<td>109</td>
<td>306</td>
<td>148</td>
<td>142</td>
</tr>
<tr>
<td>10:45am</td>
<td>316</td>
<td>76</td>
<td>304</td>
<td>104</td>
<td>122</td>
</tr>
<tr>
<td>Total</td>
<td>1260</td>
<td>355</td>
<td>1185</td>
<td>519</td>
<td>506</td>
</tr>
</tbody>
</table>

Table 5-12 Traffic Signal Phase Plan at Intersection S.R.192 & S.R.535

<table>
<thead>
<tr>
<th>Movement</th>
<th>Phase I</th>
<th>Phase II</th>
<th>Phase III</th>
</tr>
</thead>
<tbody>
<tr>
<td>EB Left-Turn</td>
<td>15</td>
<td>55</td>
<td>40</td>
</tr>
<tr>
<td>Green Time (sec.)</td>
<td>2</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Yellow Interval (sec.)</td>
<td>0</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>All Red Interval (sec.)</td>
<td>0</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

Figure 5-4: Geometric Condition of the Intersection S.R. 192 & S.R. 535

HCM capacity analysis procedure was applied to determine the capacity and LOS. Default values were applied to all the adjustment factors in the HCM capacity analysis
procedure except for the adjustment factors of area type and driver population. The area type adjustment factors were excluded from the capacity estimation in this simulation due to the introduction of driver population factor. Driver population factor was selected from Table 5-10. Driver population factors were only applied to adjust the saturation flow rate. Based on the survey data, the start-up lost time was 1.93 sec/phase, and the yellow lost time was the 75% of the yellow interval. All-red interval was always considered a part of lost time. Since the intersection S.R.192 & S.R.535 is located in a recreational area close to the Orlando Disney World, the population adjustment factor of 0.82 was selected from Table 5-10 as driver population adjustment factor. The capacity and LOS before and after the driver population factor was applied were calculated for each lane group and each movement and results were presented in Table 5-13.

Table 5-13 Capacity and LOS Before and After Driver Population Adjustment

<table>
<thead>
<tr>
<th>Approach</th>
<th>Lane Group</th>
<th>Before</th>
<th>After</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Capacity</td>
<td>LOS</td>
<td>Capacity</td>
</tr>
<tr>
<td></td>
<td>E</td>
<td></td>
<td>F</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td></td>
<td>D</td>
</tr>
<tr>
<td></td>
<td>D</td>
<td></td>
<td>E</td>
</tr>
<tr>
<td></td>
<td>Overall</td>
<td></td>
<td>F</td>
</tr>
<tr>
<td></td>
<td>Overall</td>
<td></td>
<td>D</td>
</tr>
<tr>
<td></td>
<td>Overall</td>
<td></td>
<td>D</td>
</tr>
<tr>
<td></td>
<td>Overall</td>
<td></td>
<td>E</td>
</tr>
</tbody>
</table>

From Table 5-13, it can be seen that when the driver population factor was applied, capacity at this particular intersection was reduced to 82% of the value estimated by the current HCM procedure, and level of service was changed from D to E. Therefore, the current HCM capacity analysis procedure overestimated the capacity at locations with high level of non-local drivers, which would mislead the decisions by the transportation practitioners.
CHAPTER 6
SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

Summary

This report summarizes the second phase study of the project “Driver Population Factors In Highway Capacity.” The first phase of this project was to examine the impacts of non-local drivers on the capacity of basic freeway sections, which was done in 1996. Results are included in the final report “Driver Population Factors In Freeway Capacity.” The second phase of this project was to analyze the relationship between the capacity and non-local driver population level at signalized intersections, and develop driver population adjustment factors for the capacity analysis procedure of signalized intersections. The procedures and results of the second phase study are included in this report.

Non-local drivers refer to those drivers who are not familiar with the facilities. Previous studies have indicated that any driver population groups other than weekday commuters would utilize freeways less efficiently, and that when there were a large number of non-local driver in the freeway traffic, the capacity could be significantly reduced. Non-local drivers or non-commuters in the traffic stream may cause capacity reductions in several ways, including perception-reaction time, car-following behavior, lane-change and gap acceptance behavior, and driving speed. As the percentage of non-local drivers or non-commuters increase, these factors may contribute to the expected reduction in freeway capacity. Due to the different driving behavior of non-local drivers, when there are a large number of non-local drivers in the traffic of other facilities, such as signalized intersection, the capacity reduction may also be expected.

However, the Highway Capacity Manual (HCM), which is the most prevalent highway capacity analysis reference in the United States, pays very little attention to the driver population adjustment factors. Driver population adjustment factors are only included to adjust for the capacity reduction of basic freeway sections. No such factors are applied to the capacity analysis procedure of signalized intersections. One main reason driver
population factors are not accurately described in HCM is that driver population factors vary greatly among the states. In order to overcome such problem, it is recommended to perform driver population factor studies at those states with large number of visitors.

Tourism is the biggest industry in Florida. Each year millions of domestic and overseas tourists visit Florida. Most of tourists drive to the attraction places. Non-local driver population levels are always very significant at intersections near major tourist attractions. These non-local drivers may have significant impacts on the capacity at signalized intersections. Therefore, it is necessary to develop a procedure to evaluate and quantify the impacts of non-local drivers on the capacity at signalized intersections.

The research project was conducted in the cities of Tampa, Clearwater and Kissimmee, Florida. A total of 260 peak hours were spent in the field for data collection, among which 174 hours were for the collection of discharge headways, and 86 hours for roadside interviews to obtain the driver population information.

Discharge headway of each individual vehicle and signal phase plan were recorded in the field by a laptop computer. More than 36,660 individual vehicles were recorded. Capacity parameters, including saturation flow rate, start-up lost time and yellow lost time, were calculated from discharge headway data by using the HCM procedures. Driver population information, including zipcode and age information, was obtained by roadside interviews. A total of 12,219 drivers were interviewed in this study. In this research, non-local drivers were defined as those who were living outside the county boundary of the study site. Driver zipcode information was processed by Maptitude, a GIS software. Non-local driver population level of each observation was also determined by Maptitude.

Statistical analyses were performed to examine the relationships between capacity parameters and non-local driver population levels. Linear regression models were developed to quantify the impacts of non-local driver population levels on the capacity parameters. Based on the results of the linear regression models, driver population adjustment factors were calculated.
Conclusions

One-way ANOVA tests and t-tests were performed in this study to evaluate the differences of capacity parameters among the movements, land use groups and driver population groups at a confidence level of 95%. The following is the conclusions:

1. Saturation flow rate of left-turn movement was significantly higher than that of through movement; start-up lost time of left-turn movement was not significantly different from through movement; left-turn drivers had more tendency to utilize the yellow change interval.

2. Saturation flow rate at recreational areas was the lowest among all the areas; drivers at recreational areas responded to the indication of green signal slower than that at other areas.

3. Saturation flow rate decreased when the non-local driver population levels increased; the impacts of non-local driver population levels on the start-up lost time and clearance lost time were not statistically significant.

Linear regression models were established to quantify the relationships between the saturation flow rate and the non-local driver population level. The model indicated that when the non-local driver population level was increased from 0% to 100%, the saturation flow rate was decreased 22% for left-turn movement from 2080 pcphgpl to 1630 pcphgpl, and decreased 16% for through movement from 2010 pcphgpl to 1680 pcphgpl. On average, when a signalized intersection was exposed with extremely high non-local driver population levels, saturation flow rate, as well as the capacity, was reduced to 81% of the value under normal conditions. Driver population factors were calculated based on the regression models and presented in Table 6-1.
Table 6-1 Driver Population Factors \( f_{\text{pop}} \) in Terms of Non-Local Driver Population Level

<table>
<thead>
<tr>
<th>Non-Local Driver Population Level</th>
<th>( f_{\text{pop}} ) for Saturation Flow Rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>0%</td>
<td>1.0</td>
</tr>
<tr>
<td>10%</td>
<td>0.98</td>
</tr>
<tr>
<td>20%</td>
<td>0.95</td>
</tr>
<tr>
<td>30%</td>
<td>0.93</td>
</tr>
<tr>
<td>40%</td>
<td>0.91</td>
</tr>
<tr>
<td>50%</td>
<td>0.89</td>
</tr>
<tr>
<td>60%</td>
<td>0.88</td>
</tr>
<tr>
<td>70%</td>
<td>0.86</td>
</tr>
<tr>
<td>80%</td>
<td>0.84</td>
</tr>
<tr>
<td>90%</td>
<td>0.83</td>
</tr>
<tr>
<td>100%</td>
<td>0.81</td>
</tr>
</tbody>
</table>

It was also found in this research that non-local driver population level had a significant correlation with the area type. Each area type had its distinctive distribution of non-local driver population levels. To some extent, the variation of saturation flow rate among the area types was attributed to the different non-local driver population levels at these places. Thus, an alternative format of driver population adjustment factor table was recommended in terms of area type and presented in Table 6-2. It was indicated in Table 6-2 that saturation flow rate was reduced as much as 18% at recreational areas and 10% at the other areas due to the non-local drivers' behavior.

Table 6-2 Driver Population Factors \( f_{\text{pop}} \) in Terms of Area Type

<table>
<thead>
<tr>
<th>Area Type</th>
<th>Range</th>
<th>50th percentile</th>
<th>85th percentile</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residential Area</td>
<td>1.0 – 0.95</td>
<td>0.98</td>
<td>0.96</td>
</tr>
<tr>
<td>Business Area</td>
<td>0.99 – 0.90</td>
<td>0.96</td>
<td>0.93</td>
</tr>
<tr>
<td>Shopping Area</td>
<td>0.97 – 0.94</td>
<td>0.96</td>
<td>0.95</td>
</tr>
<tr>
<td>Recreational Area</td>
<td>0.93 – 0.82</td>
<td>0.87</td>
<td>0.83</td>
</tr>
</tbody>
</table>
Recommendations

Based on the literature review performed in the project, it was found that no research had been done in the past to quantify the impacts of unfamiliar drivers on the capacity at signalized intersections. The procedures and results included in this report may provide a guidance to transportation practitioners on how to accurately and efficiently perform capacity analysis at signalized intersections with high percentage of non-local drivers in the traffic. However, due to the limited resource and time, the effort of the data collection was only conducted in Florida. Thus, the results obtained through this study and presented in this report may have some limitations. For the other states, since the driving behavior may change, field validation should be conducted before applying the adjustment factors from this research. If possible, it is strongly recommended to perform a nationwide study on driver population adjustment factors at signalized intersections, and the development of a general adjustment factor table that can be used by nationwide transportation practitioners.

The other limitation of this project is the definition of non-local drivers. In this research, definition of non-local drivers was based on the county boundary. Although such definition can be easily applied in practice, it is in fact an arbitrary definition. In the future research, a more reasonable definition of non-local drivers should be established, and new survey methodology should be applied based on such definition.
REFERENCES

Turn Lanes,” Department of Civil and Environmental Engineering, University of
Nevada, Las Vegas, May, 1996

Intersections,” Report No. UKTRP-82-8, Kentucky Transportation Research
Program, University of Kentucky, Lexington, Kentucky, 1982

Intersections,” Report No. UKTRP-83-3, Kentucky Transportation Research
Program, University of Kentucky, Lexington, Kentucky, 1983

State Departments of Transportation,” ITE Journal, July, 1995, pp. 28-33

Interchange, Dissertation,” Civil Engineering Department, Texas A&M University,
1990

Interchanges,” Transportation Research Record, No. 1365, 1992, pp. 30-39

1992, pp. 631-650

Transportation Science, Vol. 10, No. 2, May 1976

Autobahns,” Transportation Research Record, No. 1555, 1996, pp. 91-98


Johnsen, R. R., and Matthias, J. S., “Investigation to Determine the Capacity of Protected Left-Turn Movements,” Highway Research Record, No. 453, pp. 49-55


Tampa Tribune, “Florida Hot Spots,” Travel, Sunday, May 16, 1999


Williams, W. L. (1977), “Driver Behavior During the Yellow Interval,” Transportation Research Record 644, 1977, pp. 75-78

BIBLIOGRAPHY


Rouphail, N. M. (1989), "Progression Adjustment Factors at Signalized Intersections," Transportation Research Record, No. 1225, 1989, pp. 8-17


APPENDIX A

PROGRAM FOR TRAFFIC DATA COLLECTION

AT SIGNALIZED INTERSECTIONS
Option Explicit

Public FileName, DataFile, AfterYellow As String
Public Start, Green, Yellow, Red
Option Explicit

Private Sub cmdExit_Click()
    End
End Sub

Private Sub cmdOK_Click()
    If UCase(cboPeriod) = "AM" Then
        FileName = "c:\pop\data\" & Left(cboName, 1) & Left(cboIntersect, 1) & "_" & Left(cboMove, 1) & " & Format$(Now, "m_d") & "A" & "_origin.txt"
        DataFile = "c:\pop\data\" & Left(cboName, 1) & Left(cboIntersect, 1) & "_" & Left(cboMove, 1) & " & Format$(Now, "m_d") & "A" & "_headway.txt"
        AfterYellow = "c:\pop\data\" & Left(cboName, 1) & Left(cboIntersect, 1) & "_" & Left(cboMove, 1) & " & Format$(Now, "m_d") & "A" & "_sneaker.txt"
    ElseIf UCase(cboPeriod) = "PM" Then
        FileName = "c:\pop\data\" & Left(cboName, 1) & Left(cboIntersect, 1) & "_" & Left(cboMove, 1) & " & Format$(Now, "m_d") & "P" & "_origin.txt"
        DataFile = "c:\pop\data\" & Left(cboName, 1) & Left(cboIntersect, 1) & "_" & Left(cboMove, 1) & " & Format$(Now, "m_d") & "P" & "_headway.txt"
        AfterYellow = "c:\pop\data\" & Left(cboName, 1) & Left(cboIntersect, 1) & "_" & Left(cboMove, 1) & " & Format$(Now, "m_d") & "P" & "_sneaker.txt"
    End If
    Open FileName For Append As #1
    Open DataFile For Append As #2
    Open AfterYellow For Append As #5
    Write #1, "Observed Street Is ", cboName
    Write #1, "Intersected Street Is ", cboIntersect
    Write #1, "Observed Movement Is ", cboMove
    Write #1, "Observed Approach Is ", cboApproach
    Close #1
    Close #2
    Close #5

    If cboMove = "Through" Then
        Unload Me
        Form2.Show
    ElseIf cboMove = "Left Turn" Then
        Unload Me
        Form3.Show
    End If

End Sub

Private Sub Form_Load()
    cboPeriod.AddItem "AM"
    cboPeriod.AddItem "PM"
    cboName.AddItem "Boy Scout"
    cboName.AddItem "Kennedy Blvd. W."
    cboIntersect.AddItem "West Shore"
    cboIntersect.AddItem "Kennedy Blvd. W."
    cboName.AddItem "West Shore"
    cboIntersect.AddItem "Boy Scout"
    cboName.AddItem "Cypress Street"
    cboName.AddItem "Dale Mabry Hwy N."
    cboName.AddItem "Sheldon Road"
    cboIntersect.AddItem "Waters Ave."
    cboName.AddItem "Sun City Blvd."
    cboIntersect.AddItem "Pebble Beach"
    cboName.AddItem "Busch Blvd."
    cboIntersect.AddItem "Columbus"
    cboIntersect.AddItem "Mckinley Dr.N."
    cboName.AddItem "SR 580"
cboIntersect.AddItem "US 19"
cboName.AddItem "Fowler Ave."
cboIntersect.AddItem "University Mall"
cboName.AddItem "SR 60"
cboIntersect.AddItem "Brandon Mall"
cboName.AddItem "Bruce B. Downs Blvd."
cboIntersect.AddItem "Tampa Palms"
cboName.AddItem "Waters Ave."
cboIntersect.AddItem "Sheldon Road"
cboName.AddItem "S.R. 192"
cboIntersect.AddItem "S.R. 535"
cboName.AddItem "Central FL Pkwy"
cboIntersect.AddItem "International Dr."
cboName.AddItem "Columbus"
cboIntersect.AddItem "Dale Mabry Hwy."

cboApproach.AddItem "Northbound"
cboApproach.AddItem "Southbound"
cboApproach.AddItem "Eastbound"
cboApproach.AddItem "Westbound"
cboMove.AddItem "Through"
cboMove.AddItem "Left Turn"

End Sub
Option Explicit

Dim Totred As Integer
Dim i, j, Cycle As Integer
Dim Head(1 To 40) As Single
Dim Sneak(51 To 60) As Single
Dim Vtype(1 To 40) As String
Dim Ytype(51 To 60) As String
Dim Ptime, Pass
Dim Ginterval, Yinterval As Single

Private Sub cmdAnother_Click()
    'Another approach command is doing the other movement observation'
    Unload Me
    Form1.Show
End Sub

Private Sub cmdEnd_Click()
    Open FileName For Append As #1
    Write #1,
    Write #1, "Total Red Interval Being Used: ", Totred
    Close #1
End End

Private Sub Command1_Click()
    'If cycle is unsaturated or mistake happened, write information'
    Open FileName For Append As #1
    Write #1, "THIS IS A FALSE CYCLE"
    Close #1
    lblShow.ForeColor = vbBlack
    lblShow.BackColor = vbRed
    lblShow = "Waiting For Start"
End Sub

Private Sub Form_Load()
    i = 0
    Totred = 0
End Sub

Private Sub txtKey_Change()
    'unused yellow time'
    Dim Response
    Dim x, m As Integer
    Dim My, Last
    'passing stop line'
    Dim Passline
    'number of vehicles in the queue'
    Dim Nqueue As Integer
    Dim Usered As Integer
    'duration of green and yellow interval'
    'pass is the time the key pressed, headway defined each vehicle headway,'
    'ptime is time each vehicle passing stop line, gtime is time signal turns green'
    Dim Headway, Gtime
    'Signal turns green'
    If UCase(txtKey) = "J" Then
lblShow.Caption = ""
lblShow.BackColor = vbGreen
i = 0
j = 0
Ptime = 0
Pass = 0
Ginterval = 0
Yinterval = 0
For x = 1 To 40
    Head(x) = 0
    Vtype(x) = ""
Next x
For m = 51 To 60
    Sneak(m) = 0
    Ytype(m) = ""
Next m
'Green time, computer time'
Gtime = Format(Now, "h:mm:ss a/p")
Start = Timer
Green = Start
Cycle = Cycle + 1
Open FileName For Append As #1
Write #1,
Write #1, "Number of Cycle", Cycle
Write #1, "Green Time Began", Gtime
Close #1
'passenger car is passing'
ElseIf UCase(txtKey) = "D" Then
    Pass = Timer
    Ptime = Format(Now, "h:mm:ss a/p")
    i = i + 1
    lblShow = "PASSENGER CAR"
    If i < 41 Then
        Vtype(i) = "Passenger"
        Call HeadData(Ptime)
    Else
        Ytype(i) = "Passenger"
        Call LostData(Ptime)
    End If
    picPC.Picture = LoadPicture("c:\pop\pc.bmp")
    picPC.Visible = True
    Start = Pass
    Do
        My = Timer
        Last = My - Start
        Loop While Last < 0.75
    picPC.Visible = False
    'truck is passing stop line'
ElseIf UCase(txtKey) = "S" Then
    Pass = Timer
    Ptime = Format(Now, "h:mm:ss a/p")
    i = i + 1
    lblShow = "TRUCK/BUS"
    If i < 41 Then
        Vtype(i) = "Truck"
        Call HeadData(Ptime)
    Else
        Ytype(i) = "Truck"
        Call LostData(Ptime)
    End If
    picPC.Picture = LoadPicture("c:\pop\truck.bmp")
    picPC.Visible = True
    Start = Pass
    Do
My = Timer
Last = My - Start
Loop While Last < 0.75
picPC.Visible = False

'RV passing stop line'
ElseIf UCase(txtKey) = "A" Then
  Pass = Timer
  Ptime = Format(Now, "h:mm:ss a/p")
  i = i + 1
  lblShow = "RV"
  If i < 41 Then
    Ytype(i) = "RV"
    Call HeadData(Ptime)
  Else
    Ytype(i) = "RV"
    Call LostData(Ptime)
  End If
  picPC.Picture = LoadPicture("c:\pop\rv.bmp")
  picPC.Visible = True
  Start = Pass
  Do
    My = Timer
    Last = My - Start
    Loop While Last < 0.75
    picPC.Visible = False
    ElseIf UCase(txtKey) = "U" Then
      lblShow = "U Turn"
      If i < 41 Then
        Ytype(i) = "U Turn"
        Open FileName For Append As #1
        Write #1, i, "Through", Vtype(i), Ptime, Head(i)
        Close #1
      Else
        Ytype(i) = "U Turn"
        Open FileName For Append As #1
        Write #1, i, "Through", Ytype(i), Ptime, Sneak(i)
        Close #1
      End If
      Do
        My = Timer
        Last = My - Start
        Loop While Last < 0.75
      'yellow light turns on'
      ElseIf UCase(txtKey) = "K" Then
        If i >= 5 Then
          lblShow.BackColor = vbYellow
          lblShow = ""
          i = 50
          Pass = Timer
          Ptime = Format(Now, "h:mm:ss a/p")
          Yellow = Pass
          Ginterval = Yellow - Green
          Open FileName For Append As #1
          Write #1, "YELLOW INTERVAL BEGAN"
          Write #1, "Green Interval ", Format(Ginterval, "0.00"), ", Yellow Time ", Ptime
          Close #1
          Start = Pass
          Else
            Command1_Click
        End If
      'red light turns on'
      ElseIf UCase(txtKey) = "L" Then
lblShow = "Waiting For Start"
lblShow.BackColor = vbRed
Pass = Timer
Ptime = Format(Now, "h:mm:ss a/p")
Red = Pass
Yinterval = Red - Yellow
Nqueue = InputBox("Number of Vehicles in Queue")

Passline = MsgBox("Does The First Vehicle Passing The Stop Line", vbYesNo)
Response = MsgBox("Does Left Turn Movement affect Through Movement?", vbYesNo)
Usered = MsgBox("Are There Any Vehicles Using the Red Interval", vbYesNo)

Open FileName For Append As #1
Write #1, "Yellow Interval ", Format(Yinterval, "0.00"), "Red Time ", Ptime
Write #1, "Number of Vehicles in Queue is ", Nqueue
If Response = vbYes Then
    Write #1, "The intersection is not clear"
End If
If Usered = vbYes Then
    Totred = Totred + 1
    Write #1, "There are vehicles using all-red interval"
End If
Close #1
Open DataFile For Append As #2
Write #2, Cycle,
For x = 1 To Nqueue
    If Passline = vbYes Then
        Head(l) = 0
    End If
    If Vtype(x) = "Passenger" Then
        Write #2, Head(x), Vtype(x),
    Else
        Write #2, , Vtype(x),
    End If
Next x
Write #2,
Close #2

If Usered = vbNo Then
Open AfterYellow For Append As #5
Write #5, Cycle, Format(Ginterval, "0.00"), Format(Yinterval, "0.00"),
For m = 51 To 60
Write #5, Sneak(m), Ytype(m),
Next m
Write #5,
Close #5
End If
End If
txtKey = ""
End Sub

Public Sub HeadData(yanhu)
    Head(i) = Format(Pass - Start, "0.00")
Open FileName For Append As #1
Write #1, i, "Through", Vtype(i), yanhu, Head(i)
Close #1
End Sub

Public Sub LostData(yanhu)
    Sneak(i) = Format(Pass - Start, "0.00")
Open FileName For Append As #1
Write #1, i, "Through", Ytype(i), yanhu, Sneak(i)
Close #1
End Sub
Option Explicit

Dim Totred As Integer
Dim i, j, Cycle As Integer
Dim Head(1 To 40) As Single
Dim Sneak(51 To 60) As Single
Dim Vtype(1 To 40) As String
Dim Ytype(51 To 60) As String

Dim Ptime, Pass
Dim Ginterval, Yinterval As Single

Private Sub cmdAnother_Click()
    'Another approach command is doing the other movement observation'
    Unload Me
    Form1.Show
End Sub

Private Sub cmdEnd_Click()
    Open FileName For Append As #1
    Write #1,
    Write #1, "Total Red Interval Being Used: ", Totred
    Close #1
End End Sub

Private Sub Command1_Click()
    'If cycle is unsaturated or mistake happened, write information'
    Open FileName For Append As #1
    Write #1, " THIS IS A FALSE CYCLE"
    Close #1
    lblShow.ForeColor = vbBlack
    lblShow.BackColor = vbRed
    lblShow = "Waiting For Start"
End Sub

Private Sub Form_Load()
    i = 0
    Totred = 0
End Sub

Private Sub txtKey_Change()
    'unused yellow time'
    Dim Response
    Dim x, m As Integer
    Dim My, Last

    'passing stop line'
    Dim Passline

    'number of vehicles in the queue'
    Dim Nqueue As Integer
    Dim Usered As Integer

    'duration of green and yellow interval'

    'pass is the time the key pressed, headway defined each vehicle headway,'
    'ptime is time each vehicle passing stop line,gtime is time signal turns green'
    Dim Headway, Gtime

    'Signal turns green'
If UCase(txtKey) = "J" Then
    lblShow.Caption = ""
    lblShow.BackColor = vbGreen
    i = 0
    j = 0
    Ptime = 0
    Pass = 0
    Ginterval = 0
    Yinterval = 0
    For x = 1 To 40
        Head(x) = 0
        Vtype(x) = ""
    Next x
    For m = 51 To 60
        Sneak(m) = 0
        Ytype(m) = ""
    Next m

'Green time, computer time'
    Gtime = Format(Now, "h:mm:ss a/p")
    Start = Timer
    Green = Start
    Cycle = Cycle + 1
    Open FileName For Append As #1
    Write #1,
    Write #1, "Number of Cycle", Cycle
    Write #1, "Green Time Began", Gtime
    Close #1

'passenger car is passing'
ElseIf UCase(txtKey) = "D" Then
    Pass = Timer
    Ptime = Format(Now, "h:mm:ss a/p")
    i = i + 1
    lblShow = "PASSenger CAR"
    If i < 41 Then
        Vtype(i) = "Passenger"
        Call HeadData(Ptime)
    Else
        Ytype(i) = "Passenger"
        Call LostData(Ptime)
    End If
    picPC.Picture = LoadPicture("c:\pop\pc.bmp")
    picPC.Visible = True
    Start = Pass
Do
    My = Timer
    Last = My - Start
    Loop While Last < 0.75
    picPC.Visible = False

'truck is passing stop line'
ElseIf UCase(txtKey) = "S" Then
    Pass = Timer
    Ptime = Format(Now, "h:mm:ss a/p")
    i = i + 1
    lblShow = "TRUCK/BUS"
    If i < 41 Then
        Vtype(i) = "Truck"
        Call HeadData(Ptime)
    Else
        Ytype(i) = "Truck"
        Call LostData(Ptime)
    End If
    picPC.Picture = LoadPicture("c:\pop\truck.bmp")
    picPC.Visible = True
    Start = Pass
Do
  My = Timer
  Last = My - Start
Loop While Last < 0.75
picPC.Visible = False

'RV passing stop line'
ElseIf UCase(txtKey) = "A" Then
  Pass = Timer
  Ptime = Format(Now, "h:mm:ss a/p")
  i = i + 1
  lblShow = "RV"
  If i < 41 Then
    Vtype(i) = "RV"
    Call HeadData(Ptime)
  Else
    Ytype(i) = "RV"
    Call LostData(Ptime)
  End If
  picPC.Picture = LoadPicture("c:\pop\rv.bmp")
  picPC.Visible = True
  Start = Pass
  My = Timer
  Last = My - Start
Loop While Last < 0.75
picPC.Visible = False
ElseIf UCase(txtKey) = "U" Then
  lblShow = "U Turn"
  If i < 41 Then
    Vtype(i) = "U Turn"
    Open FileName For Append As #1
    Write #1, i, "Left", Vtype(i), Ptime, Head(i)
    Close #1
    Ytype(i) = "U Turn"
    Open FileName For Append As #1
    Write #1, i, "Left", Ytype(i), Ptime, Sneak(i)
    Close #1
  End If
  Do
    My = Timer
    Last = My - Start
  Loop While Last < 0.75
ElseIf UCase(txtKey) = "K" Then
  If i >= 5 Then
    lblShow.BackColor = vbYellow
    lblShow = ""
  i = 50
  Pass = Timer
  Ptime = Format(Now, "h:mm:ss a/p")
  Yellow = Pass
  Ginterval = Yellow - Green
  Open FileName For Append As #1
  Write #1, "YELLOW INTERVAL BEGAN"
  Write #1, "Green Interval ", Format(Ginterval, "0.00"), "Yellow Time ", Ptime
  Close #1
  Start = Pass
  Else
    Command1_Click
  End If

'signal turns red'
ElseIf UCase(txtKey) = "L" Then
    lblShow = "Waiting For Start"
    lblShow.BackColor = vbRed
    Pass = Timer
    Ptime = Format(Now, "h:mm:ss a/p")
    Red = Pass
    Yinterval = Red - Yellow
    Nqueue = InputBox("Number of Vehicles in Queue")
    Passline = MsgBox("Does The First Vehicle Passing The Stop Line", vbYesNo)
    Usered = MsgBox("Are There Any Vehicles Using the Red Interval?", vbYesNo)
    Open FileName For Append As #1
    Write #1, "Yellow Interval ", Format(Yinterval, "0.00"), "Red Time ", Ptime
    Write #1, "Number of Vehicles in Queue is ", Nqueue
    If Usered = vbYes Then
        Write #1, "There are vehicles using the red interval."
    End If
    Close #1
    Open DataFile For Append As #2
    Write #2, Cycle,
    For x = 1 To Nqueue
        If Passline = vbYes Then
            Head(x) = 0
        End If
        If Vtype(x) = "Passenger" Then
            Write #2, Head(x), Vtype(x),
        Else
            Write #2, , Vtype(x),
        End If
    Next x
    Write #2,
    Close #2
    If Usered = vbNo Then
    Open AfterYellow For Append As #5
    Write #5, Cycle, Format(Ginterval, "0.00"), Format(Yinterval, "0.00"),
    For m = 51 To 60
    Write #5, Sneak(m), Ytype(m),
    Next m
    Write #5,
    Close #5
    Else
    Totred = Totred + 1
    End If
End If
    txtKey = ""
End Sub

Public Sub HeadData(yanhu)
    Head(i) = Format(Pass - Start, "0.00")
    Open FileName For Append As #1
    Write #1, i, "Left", Vtype(i), yanhu, Head(i)
    Close #1
End Sub

Public Sub LostData(yanhu)
    Sneak(i) = Format(Pass - Start, "0.00")
    Open FileName For Append As #1
    Write #1, i, "Left", Ytype(i), yanhu, Sneak(i)
    Close #1
APPENDIX B

SURVEY FORM FOR THE ZIPCODE INFORMATION
<table>
<thead>
<tr>
<th>LOCATION:</th>
<th>TIME:</th>
</tr>
</thead>
<tbody>
<tr>
<td>CYCLE</td>
<td>SURVEY PERIOD</td>
</tr>
<tr>
<td>ZIP CODE</td>
<td></td>
</tr>
<tr>
<td>NO RESPONSE</td>
<td>FL</td>
</tr>
<tr>
<td></td>
<td>NON-FL</td>
</tr>
<tr>
<td>FOREIGNER</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>LOCATION:</th>
<th>TIME:</th>
</tr>
</thead>
<tbody>
<tr>
<td>CYCLE</td>
<td>SURVEY PERIOD</td>
</tr>
<tr>
<td>ZIP CODE</td>
<td></td>
</tr>
<tr>
<td>NO RESPONSE</td>
<td>FL</td>
</tr>
<tr>
<td></td>
<td>NON-FL</td>
</tr>
<tr>
<td>FOREIGNER</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>LOCATION:</th>
<th>TIME:</th>
</tr>
</thead>
<tbody>
<tr>
<td>CYCLE</td>
<td>SURVEY PERIOD</td>
</tr>
<tr>
<td>ZIP CODE</td>
<td></td>
</tr>
<tr>
<td>NO RESPONSE</td>
<td>FL</td>
</tr>
<tr>
<td></td>
<td>NON-FL</td>
</tr>
<tr>
<td>FOREIGNER</td>
<td></td>
</tr>
</tbody>
</table>

@CUTR June 1998
APPENDIX C

REDUCED DATASET
<table>
<thead>
<tr>
<th>FILENAME</th>
<th>AREATYPE</th>
<th>POP LEVEL</th>
<th>ORIGINAL SATURATION HEADWAY</th>
<th>LANE WIDTH</th>
<th>LANE ADJUST</th>
<th>ADJUSTED SATURATION HEADWAY</th>
<th>START-UP LOST TIME</th>
<th>PYU</th>
</tr>
</thead>
<tbody>
<tr>
<td>BTL0518P</td>
<td>Res</td>
<td>0.07</td>
<td>1.824</td>
<td>12</td>
<td>1.000</td>
<td>1.824</td>
<td>1.192</td>
<td>0.81</td>
</tr>
<tr>
<td>BTL0521P</td>
<td>Res</td>
<td>0.12</td>
<td>1.833</td>
<td>12</td>
<td>1.000</td>
<td>1.833</td>
<td>1.840</td>
<td>0.69</td>
</tr>
<tr>
<td>BTL0617P</td>
<td>Res</td>
<td>0.13</td>
<td>1.812</td>
<td>12</td>
<td>1.000</td>
<td>1.812</td>
<td>2.344</td>
<td>0.52</td>
</tr>
<tr>
<td>BTL0723P</td>
<td>Res</td>
<td>0.08</td>
<td>1.808</td>
<td>12</td>
<td>1.000</td>
<td>1.808</td>
<td>2.023</td>
<td>0.50</td>
</tr>
<tr>
<td>BTL0805P</td>
<td>Res</td>
<td>0.08</td>
<td>1.806</td>
<td>12</td>
<td>1.000</td>
<td>1.806</td>
<td>2.367</td>
<td>0.48</td>
</tr>
<tr>
<td>BTL0917P</td>
<td>Res</td>
<td>0.08</td>
<td>1.811</td>
<td>12</td>
<td>1.000</td>
<td>1.811</td>
<td>2.327</td>
<td>0.64</td>
</tr>
<tr>
<td>BTL0922P</td>
<td>Res</td>
<td>0.05</td>
<td>1.795</td>
<td>12</td>
<td>1.000</td>
<td>1.795</td>
<td>2.279</td>
<td>0.55</td>
</tr>
<tr>
<td>BTL1006P</td>
<td>Res</td>
<td>0.09</td>
<td>1.801</td>
<td>12</td>
<td>1.000</td>
<td>1.801</td>
<td>2.370</td>
<td>0.59</td>
</tr>
<tr>
<td>SWL0513A</td>
<td>Res</td>
<td>0.15</td>
<td>1.855</td>
<td>11</td>
<td>0.967</td>
<td>1.793</td>
<td>1.971</td>
<td>0.86</td>
</tr>
<tr>
<td>SWL0520P</td>
<td>Res</td>
<td>0.14</td>
<td>1.865</td>
<td>11</td>
<td>0.967</td>
<td>1.803</td>
<td>1.976</td>
<td>0.62</td>
</tr>
<tr>
<td>SWL0603A</td>
<td>Res</td>
<td>0.18</td>
<td>1.887</td>
<td>11</td>
<td>0.967</td>
<td>1.824</td>
<td>2.277</td>
<td>0.81</td>
</tr>
<tr>
<td>SWL0611A</td>
<td>Res</td>
<td>0.05</td>
<td>1.856</td>
<td>11</td>
<td>0.967</td>
<td>1.794</td>
<td>1.960</td>
<td>0.85</td>
</tr>
<tr>
<td>SWL0618A</td>
<td>Res</td>
<td>0.11</td>
<td>1.860</td>
<td>11</td>
<td>0.967</td>
<td>1.798</td>
<td>1.768</td>
<td>0.91</td>
</tr>
<tr>
<td>SWL0630A</td>
<td>Res</td>
<td>0.10</td>
<td>1.836</td>
<td>11</td>
<td>0.967</td>
<td>1.775</td>
<td>2.163</td>
<td>0.84</td>
</tr>
<tr>
<td>SWL0723A</td>
<td>Res</td>
<td>0.04</td>
<td>1.839</td>
<td>11</td>
<td>0.967</td>
<td>1.778</td>
<td>2.262</td>
<td>0.87</td>
</tr>
<tr>
<td>SWL0730A</td>
<td>Res</td>
<td>0.01</td>
<td>1.830</td>
<td>11</td>
<td>0.967</td>
<td>1.769</td>
<td>2.378</td>
<td>0.88</td>
</tr>
<tr>
<td>DCL0512A</td>
<td>Bus</td>
<td>0.05</td>
<td>1.770</td>
<td>12</td>
<td>1.000</td>
<td>1.770</td>
<td>1.939</td>
<td>0.46</td>
</tr>
<tr>
<td>DCL0514P</td>
<td>Bus</td>
<td>0.14</td>
<td>1.806</td>
<td>12</td>
<td>1.000</td>
<td>1.806</td>
<td>1.964</td>
<td>0.32</td>
</tr>
<tr>
<td>CDL0602P</td>
<td>Bus</td>
<td>0.17</td>
<td>1.713</td>
<td>12</td>
<td>1.000</td>
<td>1.713</td>
<td>2.476</td>
<td>0.19</td>
</tr>
<tr>
<td>CDL0918P</td>
<td>Bus</td>
<td>0.10</td>
<td>1.669</td>
<td>12</td>
<td>1.000</td>
<td>1.669</td>
<td>2.192</td>
<td>0.24</td>
</tr>
<tr>
<td>CDL0730P</td>
<td>Bus</td>
<td>0.17</td>
<td>1.719</td>
<td>12</td>
<td>1.000</td>
<td>1.719</td>
<td>2.221</td>
<td>0.23</td>
</tr>
<tr>
<td>CDL0827P</td>
<td>Bus</td>
<td>0.12</td>
<td>1.726</td>
<td>12</td>
<td>1.000</td>
<td>1.726</td>
<td>1.916</td>
<td>0.50</td>
</tr>
<tr>
<td>CDL0910P</td>
<td>Bus</td>
<td>0.13</td>
<td>1.779</td>
<td>12</td>
<td>1.000</td>
<td>1.779</td>
<td>1.758</td>
<td>0.57</td>
</tr>
<tr>
<td>CDL0929P</td>
<td>Bus</td>
<td>0.12</td>
<td>1.718</td>
<td>12</td>
<td>1.000</td>
<td>1.718</td>
<td>1.781</td>
<td>0.44</td>
</tr>
<tr>
<td>WBL0526P</td>
<td>Bus</td>
<td>0.29</td>
<td>1.839</td>
<td>12</td>
<td>1.000</td>
<td>1.839</td>
<td>1.612</td>
<td>0.77</td>
</tr>
<tr>
<td>WBL0603P</td>
<td>Bus</td>
<td>0.35</td>
<td>1.817</td>
<td>12</td>
<td>1.000</td>
<td>1.817</td>
<td>1.957</td>
<td>0.74</td>
</tr>
<tr>
<td>WBL0611P</td>
<td>Bus</td>
<td>0.38</td>
<td>1.779</td>
<td>12</td>
<td>1.000</td>
<td>1.779</td>
<td>1.995</td>
<td>0.81</td>
</tr>
<tr>
<td>WBL0629P</td>
<td>Bus</td>
<td>0.38</td>
<td>1.780</td>
<td>12</td>
<td>1.000</td>
<td>1.780</td>
<td>1.993</td>
<td>0.83</td>
</tr>
<tr>
<td>WBL0727P</td>
<td>Bus</td>
<td>0.43</td>
<td>1.847</td>
<td>12</td>
<td>1.000</td>
<td>1.847</td>
<td>1.986</td>
<td>0.74</td>
</tr>
<tr>
<td>WBL0804P</td>
<td>Bus</td>
<td>0.22</td>
<td>1.779</td>
<td>12</td>
<td>1.000</td>
<td>1.779</td>
<td>1.986</td>
<td>0.83</td>
</tr>
</tbody>
</table>
## REDUCED DATASET OF LEFT-TURN MOVEMENT

<table>
<thead>
<tr>
<th>FILENAME</th>
<th>AREATYPE</th>
<th>POP LEVEL</th>
<th>ORIGINAL SATURATION HEADWAY</th>
<th>LANE WIDTH</th>
<th>LANE ADJUST</th>
<th>ADJUSTED SATURATION HEADWAY</th>
<th>START-UP LOST TIME</th>
<th>PYU</th>
</tr>
</thead>
<tbody>
<tr>
<td>WBL0817P</td>
<td>Bus</td>
<td>0.31</td>
<td>1.794</td>
<td>12</td>
<td>1.000</td>
<td>1.794</td>
<td>2.110</td>
<td>0.77</td>
</tr>
<tr>
<td>WBL0903P</td>
<td>Bus</td>
<td>0.24</td>
<td>1.766</td>
<td>12</td>
<td>1.000</td>
<td>1.766</td>
<td>1.773</td>
<td>0.79</td>
</tr>
<tr>
<td>SUL0522A</td>
<td>Sho</td>
<td>0.16</td>
<td>1.828</td>
<td>12</td>
<td>1.000</td>
<td>1.828</td>
<td>1.776</td>
<td>0.90</td>
</tr>
<tr>
<td>SUL1107A</td>
<td>Sho</td>
<td>0.21</td>
<td>1.841</td>
<td>12</td>
<td>1.000</td>
<td>1.841</td>
<td>1.049</td>
<td>0.90</td>
</tr>
<tr>
<td>SUL1122A</td>
<td>Sho</td>
<td>0.15</td>
<td>1.816</td>
<td>12</td>
<td>1.000</td>
<td>1.816</td>
<td>1.210</td>
<td>0.88</td>
</tr>
<tr>
<td>SUL1205A</td>
<td>Sho</td>
<td>0.19</td>
<td>1.823</td>
<td>12</td>
<td>1.000</td>
<td>1.823</td>
<td>1.349</td>
<td>0.91</td>
</tr>
<tr>
<td>SUL1206A</td>
<td>Sho</td>
<td>0.16</td>
<td>1.810</td>
<td>12</td>
<td>1.000</td>
<td>1.810</td>
<td>1.505</td>
<td>1.00</td>
</tr>
<tr>
<td>SUL0116A</td>
<td>Sho</td>
<td>0.20</td>
<td>1.831</td>
<td>12</td>
<td>1.000</td>
<td>1.831</td>
<td>1.149</td>
<td>0.88</td>
</tr>
<tr>
<td>SUL0123A</td>
<td>Sho</td>
<td>0.18</td>
<td>1.814</td>
<td>12</td>
<td>1.000</td>
<td>1.814</td>
<td>1.541</td>
<td>0.84</td>
</tr>
<tr>
<td>SBL0531P</td>
<td>Sho</td>
<td>0.19</td>
<td>1.861</td>
<td>11</td>
<td>0.967</td>
<td>1.799</td>
<td>2.095</td>
<td>0.76</td>
</tr>
<tr>
<td>SBL0528A</td>
<td>Sho</td>
<td>0.17</td>
<td>1.855</td>
<td>11</td>
<td>0.967</td>
<td>1.793</td>
<td>2.002</td>
<td>0.79</td>
</tr>
<tr>
<td>SBL0719A</td>
<td>Sho</td>
<td>0.15</td>
<td>1.879</td>
<td>11</td>
<td>0.967</td>
<td>1.787</td>
<td>1.591</td>
<td>0.80</td>
</tr>
<tr>
<td>SBL0802A</td>
<td>Sho</td>
<td>0.19</td>
<td>1.888</td>
<td>11</td>
<td>0.967</td>
<td>1.825</td>
<td>2.008</td>
<td>0.62</td>
</tr>
<tr>
<td>SBL0822A</td>
<td>Sho</td>
<td>0.16</td>
<td>1.871</td>
<td>11</td>
<td>0.967</td>
<td>1.809</td>
<td>1.776</td>
<td>0.59</td>
</tr>
<tr>
<td>SBL0829A</td>
<td>Sho</td>
<td>0.18</td>
<td>1.875</td>
<td>11</td>
<td>0.967</td>
<td>1.813</td>
<td>2.018</td>
<td>0.48</td>
</tr>
<tr>
<td>SBL0905A</td>
<td>Sho</td>
<td>0.18</td>
<td>1.860</td>
<td>11</td>
<td>0.967</td>
<td>1.798</td>
<td>1.818</td>
<td>0.76</td>
</tr>
<tr>
<td>SSL0902A</td>
<td>Sho</td>
<td>0.22</td>
<td>1.876</td>
<td>11</td>
<td>0.967</td>
<td>1.813</td>
<td>1.378</td>
<td>0.25</td>
</tr>
<tr>
<td>CIL0516P</td>
<td>Rec</td>
<td>0.42</td>
<td>1.952</td>
<td>12</td>
<td>1.000</td>
<td>1.952</td>
<td>2.142</td>
<td>0.88</td>
</tr>
<tr>
<td>CIL0530P</td>
<td>Rec</td>
<td>0.26</td>
<td>1.998</td>
<td>12</td>
<td>1.000</td>
<td>1.998</td>
<td>1.066</td>
<td>0.72</td>
</tr>
<tr>
<td>CIL0613P</td>
<td>Rec</td>
<td>0.43</td>
<td>2.011</td>
<td>12</td>
<td>1.000</td>
<td>2.011</td>
<td>2.000</td>
<td>0.75</td>
</tr>
<tr>
<td>CIL0912P</td>
<td>Rec</td>
<td>0.33</td>
<td>2.011</td>
<td>12</td>
<td>1.000</td>
<td>2.011</td>
<td>2.302</td>
<td>0.74</td>
</tr>
<tr>
<td>CIL0913P</td>
<td>Rec</td>
<td>0.29</td>
<td>1.975</td>
<td>12</td>
<td>1.000</td>
<td>1.975</td>
<td>2.390</td>
<td>0.74</td>
</tr>
<tr>
<td>CIL1011P</td>
<td>Rec</td>
<td>0.29</td>
<td>1.950</td>
<td>12</td>
<td>1.000</td>
<td>1.950</td>
<td>2.799</td>
<td>0.80</td>
</tr>
<tr>
<td>CIL1108P</td>
<td>Rec</td>
<td>0.34</td>
<td>1.945</td>
<td>12</td>
<td>1.000</td>
<td>1.945</td>
<td>1.363</td>
<td>0.86</td>
</tr>
<tr>
<td>SSL0919A</td>
<td>Rec</td>
<td>0.57</td>
<td>2.045</td>
<td>12</td>
<td>1.000</td>
<td>2.045</td>
<td>1.641</td>
<td>0.68</td>
</tr>
<tr>
<td>SSL1011A</td>
<td>Rec</td>
<td>0.62</td>
<td>2.021</td>
<td>12</td>
<td>1.000</td>
<td>2.021</td>
<td>2.386</td>
<td>0.84</td>
</tr>
<tr>
<td>SSL1025P</td>
<td>Rec</td>
<td>0.53</td>
<td>2.058</td>
<td>12</td>
<td>1.000</td>
<td>2.058</td>
<td>1.348</td>
<td>0.83</td>
</tr>
<tr>
<td>SSL1108A</td>
<td>Rec</td>
<td>0.61</td>
<td>2.045</td>
<td>12</td>
<td>1.000</td>
<td>2.045</td>
<td>1.694</td>
<td>0.86</td>
</tr>
<tr>
<td>SSL1114A</td>
<td>Rec</td>
<td>0.70</td>
<td>2.013</td>
<td>12</td>
<td>1.000</td>
<td>2.013</td>
<td>1.829</td>
<td>0.82</td>
</tr>
<tr>
<td>SSL1115A</td>
<td>Rec</td>
<td>0.71</td>
<td>2.102</td>
<td>12</td>
<td>1.000</td>
<td>2.102</td>
<td>1.774</td>
<td>0.77</td>
</tr>
<tr>
<td>FILENAME</td>
<td>AREATYPE</td>
<td>POP LEVEL</td>
<td>ORIGINAL SATURATION HEADWAY</td>
<td>LANE WIDTH</td>
<td>LANE ADJUST</td>
<td>ADJUSTED SATURATION HEADWAY</td>
<td>START-UP LOST TIME</td>
<td>PYU</td>
</tr>
<tr>
<td>----------</td>
<td>----------</td>
<td>-----------</td>
<td>-----------------------------</td>
<td>------------</td>
<td>-------------</td>
<td>-----------------------------</td>
<td>-------------------</td>
<td>-----</td>
</tr>
<tr>
<td>SSL1213A</td>
<td>Rec</td>
<td>0.72</td>
<td>2.096</td>
<td>12</td>
<td>1.000</td>
<td>2.096</td>
<td>1.108</td>
<td>0.66</td>
</tr>
<tr>
<td>SSL0117A</td>
<td>Rec</td>
<td>0.63</td>
<td>2.050</td>
<td>12</td>
<td>1.000</td>
<td>2.050</td>
<td>2.281</td>
<td>0.75</td>
</tr>
<tr>
<td>FILENAME</td>
<td>AREATYPE</td>
<td>POP LEVEL</td>
<td>ORIGINAL SATURATION HEADWAY</td>
<td>LANE WIDTH</td>
<td>LANE ADJUST</td>
<td>ADJUSTED SATURATION HEADWAY</td>
<td>START-UP</td>
<td>PYU</td>
</tr>
<tr>
<td>-----------</td>
<td>----------</td>
<td>-----------</td>
<td>----------------------------</td>
<td>------------</td>
<td>-------------</td>
<td>----------------------------</td>
<td>----------</td>
<td>------</td>
</tr>
<tr>
<td>BTT0311A</td>
<td>Res</td>
<td>0.16</td>
<td>1.777</td>
<td>12</td>
<td>1.000</td>
<td>1.777</td>
<td>1.723</td>
<td>74%</td>
</tr>
<tr>
<td>BTT0430A</td>
<td>Res</td>
<td>0.08</td>
<td>1.768</td>
<td>12</td>
<td>1.000</td>
<td>1.768</td>
<td>1.808</td>
<td>70%</td>
</tr>
<tr>
<td>BTT0514A</td>
<td>Res</td>
<td>0.06</td>
<td>1.767</td>
<td>12</td>
<td>1.000</td>
<td>1.767</td>
<td>1.831</td>
<td>81%</td>
</tr>
<tr>
<td>BTT0528A</td>
<td>Res</td>
<td>0.14</td>
<td>1.770</td>
<td>12</td>
<td>1.000</td>
<td>1.770</td>
<td>1.732</td>
<td>75%</td>
</tr>
<tr>
<td>BTT0610A</td>
<td>Res</td>
<td>0.10</td>
<td>1.750</td>
<td>12</td>
<td>1.000</td>
<td>1.750</td>
<td>1.919</td>
<td>75%</td>
</tr>
<tr>
<td>BTT0617A</td>
<td>Res</td>
<td>0.12</td>
<td>1.779</td>
<td>12</td>
<td>1.000</td>
<td>1.779</td>
<td>1.805</td>
<td>51%</td>
</tr>
<tr>
<td>BTT0714A</td>
<td>Res</td>
<td>0.08</td>
<td>1.757</td>
<td>12</td>
<td>1.000</td>
<td>1.757</td>
<td>1.780</td>
<td>69%</td>
</tr>
<tr>
<td>BTT0826A</td>
<td>Res</td>
<td>0.14</td>
<td>1.756</td>
<td>12</td>
<td>1.000</td>
<td>1.756</td>
<td>1.953</td>
<td>58%</td>
</tr>
<tr>
<td>SWT0511P</td>
<td>Res</td>
<td>0.16</td>
<td>1.769</td>
<td>12</td>
<td>1.000</td>
<td>1.769</td>
<td>1.714</td>
<td>75%</td>
</tr>
<tr>
<td>SWT0515A</td>
<td>Res</td>
<td>0.14</td>
<td>1.816</td>
<td>12</td>
<td>1.000</td>
<td>1.816</td>
<td>2.015</td>
<td>87%</td>
</tr>
<tr>
<td>SWT0601P</td>
<td>Res</td>
<td>0.14</td>
<td>1.800</td>
<td>12</td>
<td>1.000</td>
<td>1.800</td>
<td>2.076</td>
<td>82%</td>
</tr>
<tr>
<td>SWT0608P</td>
<td>Res</td>
<td>0.19</td>
<td>1.837</td>
<td>12</td>
<td>1.000</td>
<td>1.837</td>
<td>2.107</td>
<td>85%</td>
</tr>
<tr>
<td>SWT0615P</td>
<td>Res</td>
<td>0.12</td>
<td>1.812</td>
<td>12</td>
<td>1.000</td>
<td>1.812</td>
<td>2.099</td>
<td>84%</td>
</tr>
<tr>
<td>SWT0624P</td>
<td>Res</td>
<td>0.07</td>
<td>1.784</td>
<td>12</td>
<td>1.000</td>
<td>1.784</td>
<td>2.089</td>
<td>84%</td>
</tr>
<tr>
<td>SWT0803P</td>
<td>Res</td>
<td>0.11</td>
<td>1.797</td>
<td>12</td>
<td>1.000</td>
<td>1.797</td>
<td>2.135</td>
<td>75%</td>
</tr>
<tr>
<td>SWT0825P</td>
<td>Res</td>
<td>0.10</td>
<td>1.812</td>
<td>12</td>
<td>1.000</td>
<td>1.812</td>
<td>2.031</td>
<td>79%</td>
</tr>
<tr>
<td>DKT0609A</td>
<td>Bus</td>
<td>0.22</td>
<td>1.926</td>
<td>12</td>
<td>1.000</td>
<td>1.926</td>
<td>1.144</td>
<td>94%</td>
</tr>
<tr>
<td>DKT0616A</td>
<td>Bus</td>
<td>0.11</td>
<td>1.859</td>
<td>12</td>
<td>1.000</td>
<td>1.859</td>
<td>2.102</td>
<td>88%</td>
</tr>
<tr>
<td>DKT0625A</td>
<td>Bus</td>
<td>0.14</td>
<td>1.857</td>
<td>12</td>
<td>1.000</td>
<td>1.857</td>
<td>1.555</td>
<td>94%</td>
</tr>
<tr>
<td>DKT0701A</td>
<td>Bus</td>
<td>0.18</td>
<td>1.832</td>
<td>12</td>
<td>1.000</td>
<td>1.832</td>
<td>1.801</td>
<td>96%</td>
</tr>
<tr>
<td>DKT0716A</td>
<td>Bus</td>
<td>0.07</td>
<td>1.835</td>
<td>12</td>
<td>1.000</td>
<td>1.835</td>
<td>1.715</td>
<td>93%</td>
</tr>
<tr>
<td>DKT0722A</td>
<td>Bus</td>
<td>0.20</td>
<td>1.942</td>
<td>12</td>
<td>1.000</td>
<td>1.942</td>
<td>1.488</td>
<td>78%</td>
</tr>
<tr>
<td>DKT0729A</td>
<td>Bus</td>
<td>0.11</td>
<td>1.880</td>
<td>12</td>
<td>1.000</td>
<td>1.880</td>
<td>1.699</td>
<td>94%</td>
</tr>
<tr>
<td>DKT0818A</td>
<td>Bus</td>
<td>0.12</td>
<td>1.844</td>
<td>12</td>
<td>1.000</td>
<td>1.844</td>
<td>1.862</td>
<td>89%</td>
</tr>
<tr>
<td>CWT0604P</td>
<td>Bus</td>
<td>0.12</td>
<td>1.851</td>
<td>12</td>
<td>1.000</td>
<td>1.851</td>
<td>1.737</td>
<td>81%</td>
</tr>
<tr>
<td>CWT0610P</td>
<td>Bus</td>
<td>0.11</td>
<td>1.910</td>
<td>12</td>
<td>1.000</td>
<td>1.910</td>
<td>1.890</td>
<td>71%</td>
</tr>
<tr>
<td>CWT0618P</td>
<td>Bus</td>
<td>0.13</td>
<td>1.900</td>
<td>12</td>
<td>1.000</td>
<td>1.900</td>
<td>1.432</td>
<td>72%</td>
</tr>
<tr>
<td>CWT0623P</td>
<td>Bus</td>
<td>0.16</td>
<td>1.903</td>
<td>12</td>
<td>1.000</td>
<td>1.903</td>
<td>1.371</td>
<td>68%</td>
</tr>
<tr>
<td>CWT0728P</td>
<td>Bus</td>
<td>0.08</td>
<td>1.882</td>
<td>12</td>
<td>1.000</td>
<td>1.882</td>
<td>1.678</td>
<td>63%</td>
</tr>
<tr>
<td>CWT0813P</td>
<td>Bus</td>
<td>0.10</td>
<td>1.884</td>
<td>12</td>
<td>1.000</td>
<td>1.884</td>
<td>1.447</td>
<td>78%</td>
</tr>
</tbody>
</table>
## REDUCED DATASET OF THROUGH MOVEMENT

<table>
<thead>
<tr>
<th>FILENAME</th>
<th>AREATYPE</th>
<th>POP LEVEL</th>
<th>ORIGINAL SATURATION</th>
<th>LANE WIDTH</th>
<th>LANE ADJUST</th>
<th>ADJUSTED SATURATION</th>
<th>START-UP</th>
<th>PYU</th>
</tr>
</thead>
<tbody>
<tr>
<td>CWT0908P</td>
<td>Bus</td>
<td>0.12</td>
<td>1.896</td>
<td>12</td>
<td>1.000</td>
<td>1.896</td>
<td>1.699</td>
<td>66%</td>
</tr>
<tr>
<td>CWT0924P</td>
<td>Bus</td>
<td>0.10</td>
<td>1.890</td>
<td>12</td>
<td>1.000</td>
<td>1.890</td>
<td>1.451</td>
<td>79%</td>
</tr>
<tr>
<td>SUT0523P</td>
<td>Sho</td>
<td>0.17</td>
<td>1.875</td>
<td>12</td>
<td>1.000</td>
<td>1.875</td>
<td>1.719</td>
<td>92%</td>
</tr>
<tr>
<td>SUT0614A</td>
<td>Sho</td>
<td>0.20</td>
<td>1.860</td>
<td>12</td>
<td>1.000</td>
<td>1.860</td>
<td>1.849</td>
<td>85%</td>
</tr>
<tr>
<td>SUT0726A</td>
<td>Sho</td>
<td>0.10</td>
<td>1.841</td>
<td>12</td>
<td>1.000</td>
<td>1.841</td>
<td>2.035</td>
<td>88%</td>
</tr>
<tr>
<td>SUT0801A</td>
<td>Sho</td>
<td>0.23</td>
<td>1.875</td>
<td>12</td>
<td>1.000</td>
<td>1.875</td>
<td>1.571</td>
<td>83%</td>
</tr>
<tr>
<td>SUT0823A</td>
<td>Sho</td>
<td>0.14</td>
<td>1.865</td>
<td>12</td>
<td>1.000</td>
<td>1.865</td>
<td>2.127</td>
<td>88%</td>
</tr>
<tr>
<td>SUT0905A</td>
<td>Sho</td>
<td>0.15</td>
<td>1.836</td>
<td>12</td>
<td>1.000</td>
<td>1.836</td>
<td>1.856</td>
<td>88%</td>
</tr>
<tr>
<td>SUT1010A</td>
<td>Sho</td>
<td>0.18</td>
<td>1.875</td>
<td>12</td>
<td>1.000</td>
<td>1.875</td>
<td>1.944</td>
<td>89%</td>
</tr>
<tr>
<td>SUT1024A</td>
<td>Sho</td>
<td>0.19</td>
<td>1.812</td>
<td>12</td>
<td>1.000</td>
<td>1.812</td>
<td>1.738</td>
<td>84%</td>
</tr>
<tr>
<td>SST0516A</td>
<td>Rec</td>
<td>0.89</td>
<td>2.034</td>
<td>12</td>
<td>1.000</td>
<td>2.034</td>
<td>2.839</td>
<td>84%</td>
</tr>
<tr>
<td>SST0530A</td>
<td>Rec</td>
<td>0.90</td>
<td>2.114</td>
<td>12</td>
<td>1.000</td>
<td>2.114</td>
<td>2.678</td>
<td>79%</td>
</tr>
<tr>
<td>SST0613A</td>
<td>Rec</td>
<td>0.92</td>
<td>2.125</td>
<td>12</td>
<td>1.000</td>
<td>2.125</td>
<td>1.929</td>
<td>80%</td>
</tr>
<tr>
<td>SST0620A</td>
<td>Rec</td>
<td>0.89</td>
<td>2.192</td>
<td>12</td>
<td>1.000</td>
<td>2.192</td>
<td>2.099</td>
<td>81%</td>
</tr>
<tr>
<td>SST0718A</td>
<td>Rec</td>
<td>0.88</td>
<td>2.119</td>
<td>12</td>
<td>1.000</td>
<td>2.119</td>
<td>1.538</td>
<td>78%</td>
</tr>
<tr>
<td>SST0830A</td>
<td>Rec</td>
<td>0.84</td>
<td>2.117</td>
<td>12</td>
<td>1.000</td>
<td>2.117</td>
<td>2.464</td>
<td>76%</td>
</tr>
<tr>
<td>SST0919A</td>
<td>Rec</td>
<td>0.80</td>
<td>2.027</td>
<td>12</td>
<td>1.000</td>
<td>2.027</td>
<td>2.991</td>
<td>76%</td>
</tr>
<tr>
<td>SST1011P</td>
<td>Rec</td>
<td>0.86</td>
<td>2.088</td>
<td>12</td>
<td>1.000</td>
<td>2.088</td>
<td>1.852</td>
<td>79%</td>
</tr>
<tr>
<td>IKT0913A</td>
<td>Rec</td>
<td>0.88</td>
<td>2.094</td>
<td>12</td>
<td>1.000</td>
<td>2.094</td>
<td>2.364</td>
<td>61%</td>
</tr>
<tr>
<td>IKT1011A</td>
<td>Rec</td>
<td>0.88</td>
<td>2.044</td>
<td>12</td>
<td>1.000</td>
<td>2.044</td>
<td>2.359</td>
<td>55%</td>
</tr>
<tr>
<td>IKT1025A</td>
<td>Rec</td>
<td>0.84</td>
<td>2.013</td>
<td>12</td>
<td>1.000</td>
<td>2.013</td>
<td>2.252</td>
<td>39%</td>
</tr>
<tr>
<td>IKT4108A</td>
<td>Rec</td>
<td>0.79</td>
<td>2.039</td>
<td>12</td>
<td>1.000</td>
<td>2.039</td>
<td>2.062</td>
<td>75%</td>
</tr>
<tr>
<td>IKT1114A</td>
<td>Rec</td>
<td>0.74</td>
<td>2.052</td>
<td>12</td>
<td>1.000</td>
<td>2.052</td>
<td>2.398</td>
<td>86%</td>
</tr>
<tr>
<td>IKT1115A</td>
<td>Rec</td>
<td>0.79</td>
<td>2.180</td>
<td>12</td>
<td>1.000</td>
<td>2.180</td>
<td>1.538</td>
<td>77%</td>
</tr>
<tr>
<td>IKT0117P</td>
<td>Rec</td>
<td>0.75</td>
<td>2.040</td>
<td>12</td>
<td>1.000</td>
<td>2.040</td>
<td>2.918</td>
<td>53%</td>
</tr>
<tr>
<td>IKT0124A</td>
<td>Rec</td>
<td>0.78</td>
<td>2.093</td>
<td>12</td>
<td>1.000</td>
<td>2.093</td>
<td>2.103</td>
<td>61%</td>
</tr>
<tr>
<td>IKT0503A</td>
<td>Rec</td>
<td>0.60</td>
<td>2.000</td>
<td>12</td>
<td>1.000</td>
<td>2.000</td>
<td>2.578</td>
<td>79%</td>
</tr>
<tr>
<td>IKT0504P</td>
<td>Rec</td>
<td>0.44</td>
<td>1.978</td>
<td>12</td>
<td>1.000</td>
<td>1.978</td>
<td>2.169</td>
<td>90%</td>
</tr>
<tr>
<td>SJT0503P</td>
<td>Rec</td>
<td>0.51</td>
<td>1.925</td>
<td>12</td>
<td>1.000</td>
<td>1.925</td>
<td>1.597</td>
<td>70%</td>
</tr>
<tr>
<td>SJT0504A</td>
<td>Rec</td>
<td>0.52</td>
<td>1.948</td>
<td>12</td>
<td>1.000</td>
<td>1.948</td>
<td>2.092</td>
<td>53%</td>
</tr>
<tr>
<td>FILENAME</td>
<td>AREATYPE</td>
<td>POP LEVEL</td>
<td>ORIGINAL SATURATION HEADWAY</td>
<td>LANE WIDTH</td>
<td>LANE ADJUST</td>
<td>ADJUSTED SATURATION HEADWAY</td>
<td>START-UP</td>
<td>PYU</td>
</tr>
<tr>
<td>-----------</td>
<td>-----------</td>
<td>-----------</td>
<td>-----------------------------</td>
<td>------------</td>
<td>-------------</td>
<td>-------------------------------</td>
<td>----------</td>
<td>------</td>
</tr>
<tr>
<td>IKT0510P</td>
<td>Rec</td>
<td>0.59</td>
<td>1.959</td>
<td>12</td>
<td>1.000</td>
<td>1.959</td>
<td>1.970</td>
<td>76%</td>
</tr>
<tr>
<td>IKT0511A</td>
<td>Rec</td>
<td>0.63</td>
<td>2.055</td>
<td>12</td>
<td>1.000</td>
<td>2.055</td>
<td>2.342</td>
<td>86%</td>
</tr>
<tr>
<td>IKT0512A</td>
<td>Rec</td>
<td>0.62</td>
<td>2.026</td>
<td>12</td>
<td>1.000</td>
<td>2.026</td>
<td>2.684</td>
<td>71%</td>
</tr>
<tr>
<td>SJT0510A</td>
<td>Rec</td>
<td>0.47</td>
<td>1.942</td>
<td>12</td>
<td>1.000</td>
<td>1.942</td>
<td>2.043</td>
<td>54%</td>
</tr>
<tr>
<td>SJT0511P</td>
<td>Rec</td>
<td>0.53</td>
<td>1.894</td>
<td>12</td>
<td>1.000</td>
<td>1.894</td>
<td>2.236</td>
<td>51%</td>
</tr>
<tr>
<td>SJT0512P</td>
<td>Rec</td>
<td>0.45</td>
<td>1.983</td>
<td>12</td>
<td>1.000</td>
<td>1.983</td>
<td>2.346</td>
<td>56%</td>
</tr>
</tbody>
</table>