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IMPACT OF NON-LOCAL DRIVERS ON THE CAPACITY AT SIGNALIZED INTERSECTIONS

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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 predicable 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:

$$c_i = s_i \times \frac{g_i}{C} = s_i \times \frac{(G + Y + AR - t_{sl} - t_{cl})}{C}$$

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_{sl}= start-up lost time, sec.;

t_{cl}= clearance lost time, sec.; and

g_i = effective green time, $g_i = G + Y + AR - t_{sl} - t_{cl}$, sec.

The above equation indicates that the capacity at a signalized intersection depends on saturation flow rate s_i, start-up lost time t_{sl} and clearance lost time t_{cl} 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].

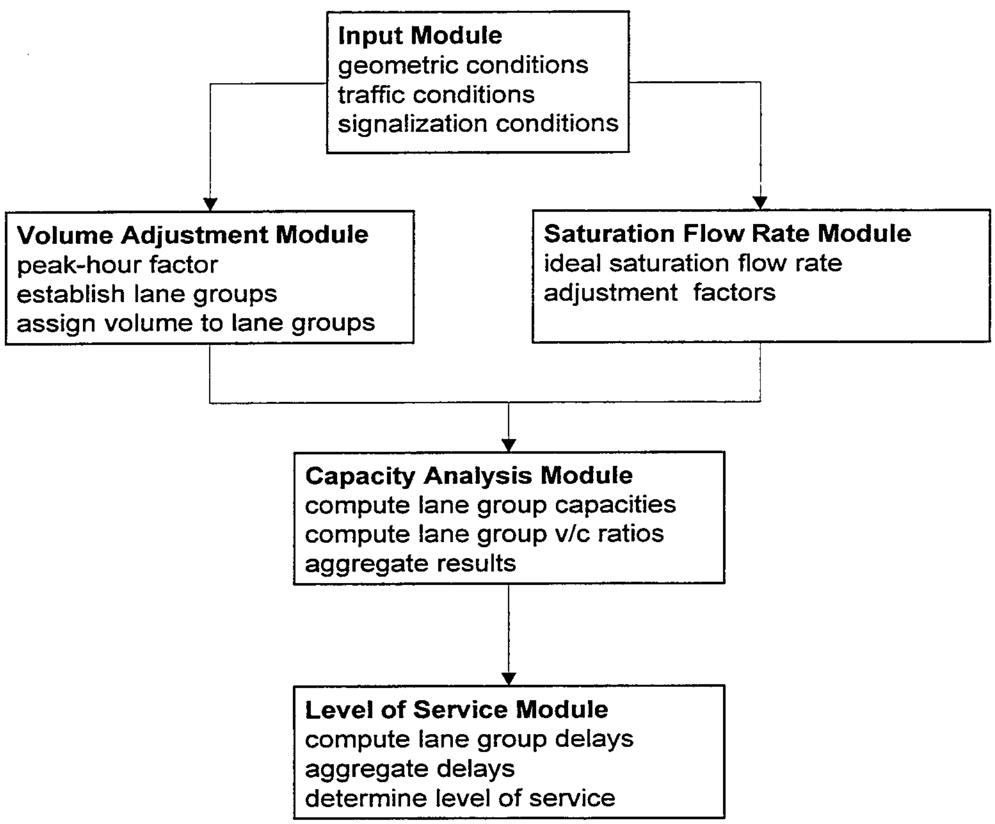

Figure 2-1 Operational Analysis Modules for Signalized Intersections

(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:

$$c_i = s_i\,(g_i/C)$$

where:

c_i = capacity of lane group i, vph;

s_i = saturation flow rate for lane group i, vphg; and

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 = 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 = ideal saturation flow rate per lane, usually 1900 pcphgpl;

N = number of lanes in the lane group;

f_w = adjustment factor for lane width;

f_{HV} = adjustment factor for heavy vehicles in the traffic stream;

f_g = adjustment factor for approach grade;

f_p = adjustment factor for the existence of a parking lane adjacent to the lane group and the parking activity in that lane;

f_{bb} = adjustment factor for the blocking effect of local buses that stop within the intersection area;

f_a = adjustment factor for area type;

f_{LU} = adjustment factor for lane utilization;

f_{RT} = adjustment factor for right-turns in the lane group; and

f_{LT} = 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:

$$s = 3600 / h_s$$

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.

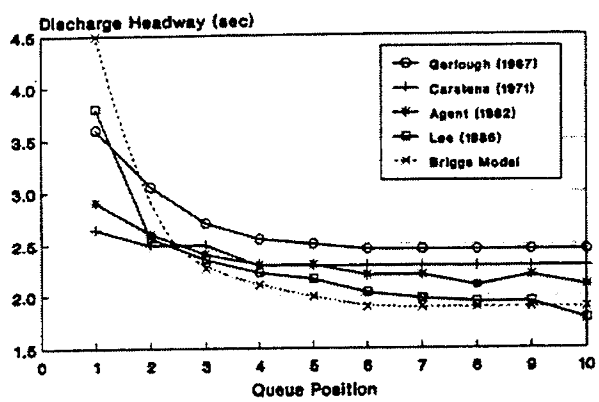

Figure 2-2 Comparison of Past Studies of Queue Discharge Headway

(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

Queue	Range of Discharge Headway (Sec.)	Average Discharge Headway (Sec.)
1	2.04-3.74	2.90
2	1.87-2.44	2.04
3	2.00-2.33	2.10
4	1.96-2.36	2.04
5	1.64-2.10	1.87
6	1.87-1.95	1.91
7	1.69-1.82	1.75

(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} = start-up lost time (sec/phase);

h_n = discharge headway of n^{th} queued vehicle (sec);

h_s = saturation headway (sec); and

N = number of queue positions having headways larger than h_s.

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

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

(Source: Niittymaeki, 1997)

The above equation indicats 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

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

(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} = clearance lost time, sec.;

AR= all-red time, sec.;

Y = yellow change interval, sec.; and

EU = 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:

$$f_w = 1 + (w - 12)/30$$

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].

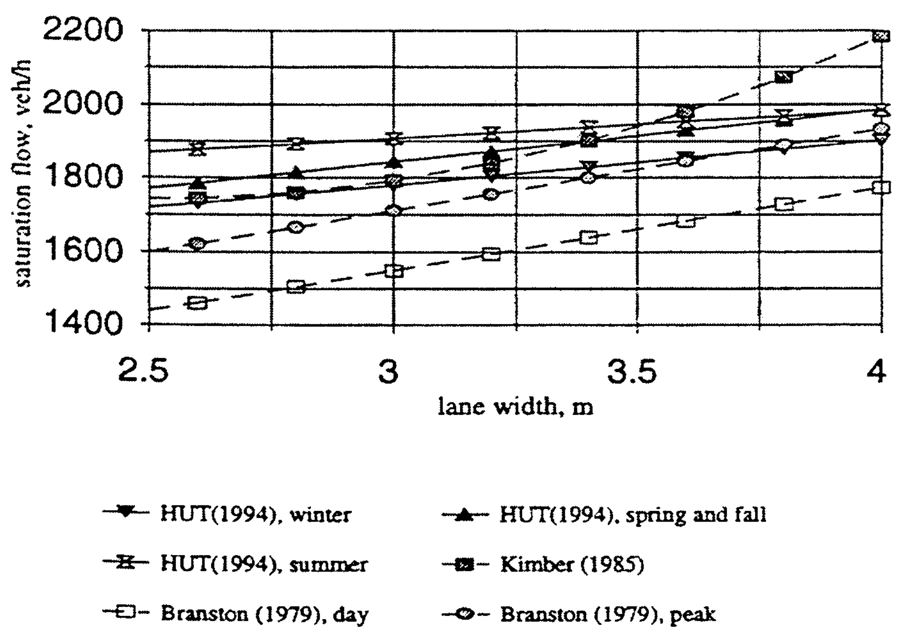

Figure 2-3 Effect of Lane Width on Saturation Flow Values

(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} = adjustment factor for heavy vehicles in the traffic stream;

%HV = percentage of heavy vehicles; and

E_T = 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:

$$\text{Equivalency Factor} = 1 + 5729.7 / (r^{2.9})$$

where:

r = 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 2-4 Effect of Speed Limit on Saturation Flows

Speed Limit	Total Headways	Average Headway (sec.)	Saturation Flow (vphg)
35	4614	2.19	1644
40	1042	2.15	1674
45	10726	2.17	1659
50	489	2.13	1690
55	391	2.11	1706

Table 2-5 Effect of Speed Limit on Start-Up Lost Time

Speed Limit (mph)	Average Start-Up Lost Time (sec.)
35	0.60
45	1.16
55	1.55

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

Speed Limit (mph)	Average Clearance Lost Time (second)
35	1.81
45	1.64
55	0.88

(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].

(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

Cycle Length (sec.)	Average Start-Up Lost Time (sec.)
60 to 90	1.42
90 to 120	1.28
120 or more	0.85

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

Cycle Length (sec.)	Average Clearance Lost Time (sec.)
less than 90	2.09
90 to 120	1.62
120 to 180	1.62
greater than 180	1.29

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

Green Time (sec.)	Average Start-Up Lost Time (sec.)
30 or less	1.72
30 to 60	1.20
60 or more	1.05

Table 2-10 Effect of Green Time on Clearance Lost Time

Green Time (sec.)	Average Clearance Lost Time (sec.)
less than 30	2.09
30 to 60	1.62
greater than 60	1.35

(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

		Straight-through Saturation Flow (pcphgpl)	
Site	Lane Width (ft)	Daylight	Darkness
1	11.8	1771	-
2	9.8	1757	1658
3	10.8	1767	1661
4	14.1	2007	1917
5	14.1	2092	1897

(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 10^{th} 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 2-12 Driver Population Factor f_p Recommended by Sharma

Traffic Stream Type	Driver Population Factor f_p
Urban Commuter	1.0
Regional Commuter	0.95
Regional Recreational & Commuter	0.90
Inter-Regional	0.85
Long Distance	0.85
Long Distance and Recreational	0.80
Highly Recreational	0.75

(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:

$$c_i = s_i \times \frac{g_i}{C} = s_i \times \frac{(G + Y + AR - t_{sl} - t_{cl})}{C}$$

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_{sl}= start-up lost time, sec.;

t_{cl}= clearance lost time, sec.; and

g_i = effective green time, $g_i = G + Y + AR - t_{sl} - t_{cl}$, 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^{th} queued vehicle to the last queued vehicle. It is expressed as:

$$h_s = \frac{\sum_{i=1}^{m}\sum_{j=5}^{n_i} h_{ij}}{\sum_{i=1}^{m}(n_i - 4)}$$

where:

h_s = saturation headway, sec.;

h_{ij} = discharge headway of j^{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^{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:

$$t_{sl} = \sum_{i=1}^{4} h_i - 4 \times h_s = \text{HFF} - 4 \times h_s$$

where:

t_{sl} = start-up lost time, sec.;

h_i = discharge headway of the i^{th} vehicle in the queue, sec.; and

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:

$$t_{cl} = \frac{1}{m}\sum_{i=1}^{m}(Y - EU_i) + AR$$

where:

t_{cl} = average clearance 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.

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.

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% (α=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_{local} and the total number of surveyed drivers n_{total} were computed. Then, the non-local driver population level NPL was calculated by the following equation:

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

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.

Area Type	Observation Period	Saturation Headway	HFF	Start-Up Lost Time	PYU	NPL
Residential	1	$h_{s(1)}$	$HFF_{(1)}$	$t_{sl(1)}$	$PYU_{(1)}$	$NPL_{(1)}$
	2	$h_{s(2)}$	$HFF_{(2)}$	$t_{sl(2)}$	$PYU_{(2)}$	$NPL_{(2)}$
Business	....					
Shopping	...					
Recreational	...					

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:

$$f_{pop(NPL)} = \frac{CP_{NPL}}{CP_0}$$

where:

$f_{pop(NPL)}$ = 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.

Non-Local Driver Population Level	Driver Population Factor
0	
10%	
20%	
...	
100%	

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

Area Type	Driver Population Factor
Residential Area	
Business Area	
Shopping Area	
Recreational Area	

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_{adjust} can be calculated as:

$$s_{adjust} = s_{field} / F$$

where:

s_{field} =saturation flow rate estimated by field data, vphgpl;

s_{adjust}=saturation flow rate under ideal conditions except driver population conditions, pcphgpl;

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 on-going 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.

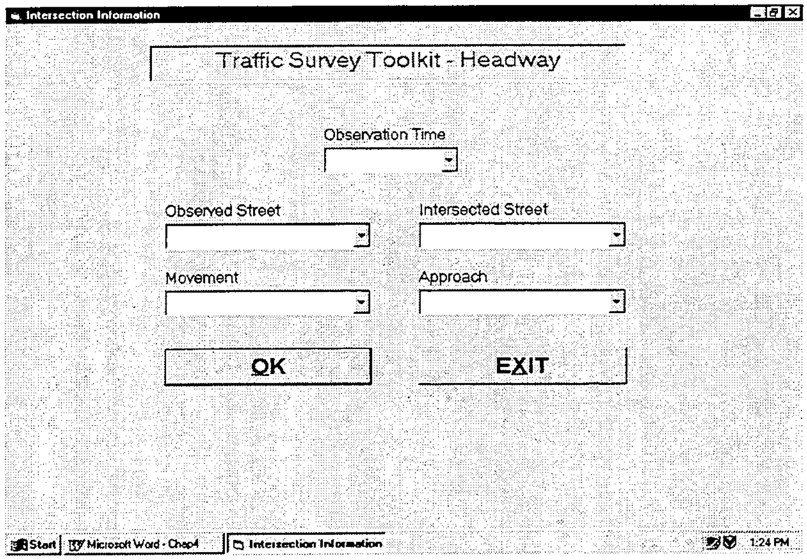

Figure 4-1 Physical Information of Data Collection

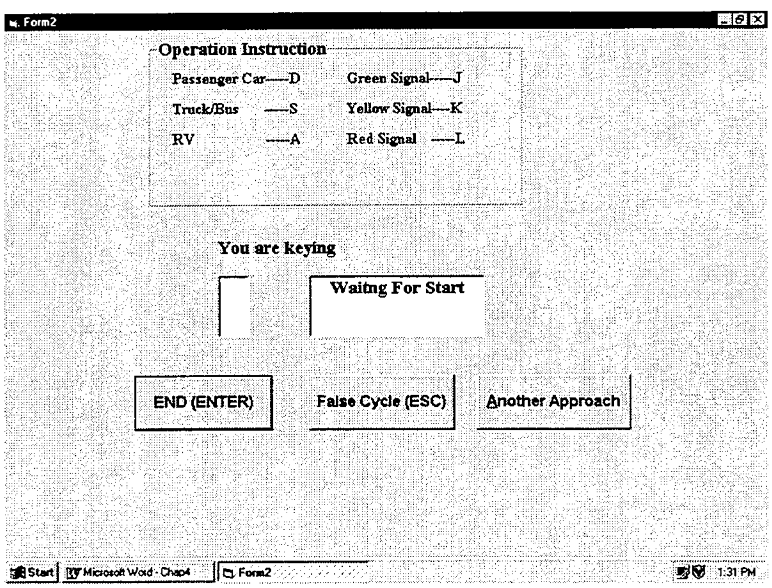

Figure 4-2 Interface of Traffic Data Collection Window

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 record;

- 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 4^{th} 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

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

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_{local} and the total surveyed people n_{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

	Left-Turn Movement		Through Movement	
Capacity Parameters	Range	Average	Range	Average
Saturation Headway (sec.)	1.67 – 2.10	1.85	1.75 – 2.19	1.92
HFF (sec.)	8.41 – 10.60	9.36	8.79 – 11.13	9.64
Start-Up Lost Time (sec.)	1.05 – 2.80	1.90	1.14 – 2.99	1.97
PYU (%)	19 - 100	71	39 – 96	76

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% (α=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

	t statistics	t_{crit} ($\alpha = 0.05$)	H_0
Saturation Headway	3.50	1.98	rejected
HFF	2.61	1.98	rejected
Start-Up Lost Time	1.09	1.98	not rejected
PYU	2.00	1.98	rejected

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

Area Type	Saturation Headway (sec.)	HFF (sec.)	Start-Up Lost Time (sec.)	PYU (%)
Residential Area	1.79	9.18	2.01	73
Business Area	1.83	9.10	1.80	70
Shopping Area	1.83	9.02	1.70	80
Recreational Area	2.03	10.24	2.11	73

(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% (α=0.05) was chosen for the tests. The principal of one-way ANOVA tests is to compare the value of F ratio and F_{crit} at a given level of significance α. 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_{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 $F>F_{crit}$, 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

Capacity Parameters	Left-Turn Movement			Through Movement		
	F	F_{crit} ($\alpha = 0.05$)	H_0	F	F_{crit} ($\alpha = 0.05$)	H_0
Saturation Headway	22.03	2.18	reject	39.02	2.11	reject
HFF	3.70	2.18	reject	43.92	2.11	reject
Start-Up Lost Time	1.77	2.18	Not reject	5.55	2.11	reject
PYU	0.75	2.18	Not reject	1.83	2.11	not reject

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.

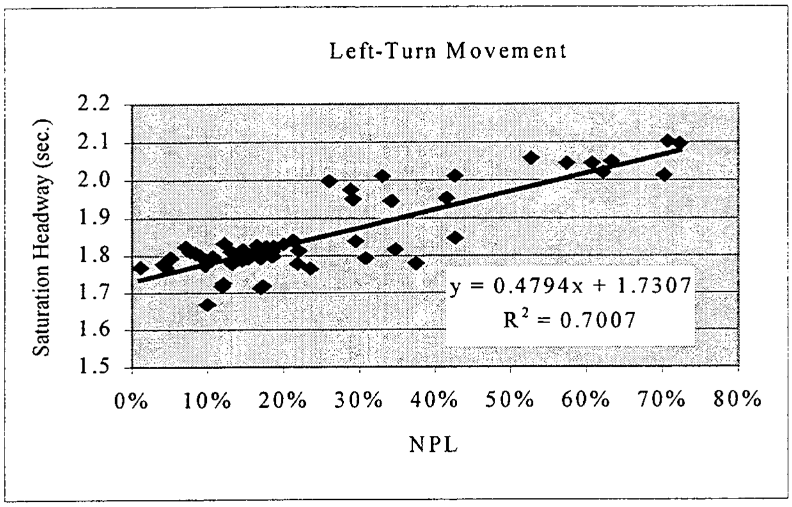

Figure 5-1: Left-Turn Saturation Headway vs. Non-Local Driver Population Level (NPL)

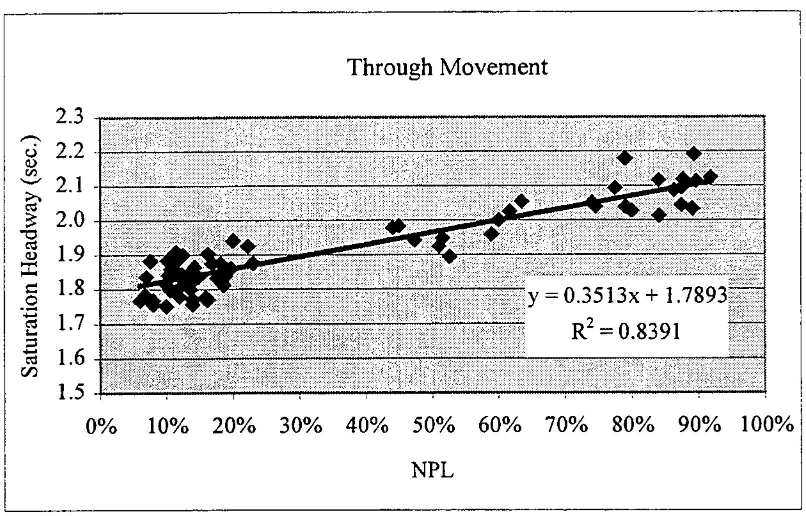

Figure 5-2: Through Saturation Headway vs. Non-Local Driver Population Level (NPL)

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

	LEFT-TURN MOVEMENT	THROUGH MOVEMENT
A	1.73	1.79
B	0.48	0.35
R^2	0.70	0.84
t-value	11.85	18.27

Table 5-6 Saturation Flow Rate under a Given Population Level

Non-Local Driver Population Level	Left-Turn Movement		Through Movement	
	Saturation headway (sec.)	Saturation flow rate (pcphgpl)	Saturation headway (sec.)	Saturation flow rate (pcphgpl)
0%	1.73	2080	1.79	2010
10%	1.78	2020	1.83	1970
20%	1.83	1970	1.86	1935
30%	1.87	1925	1.90	1895
40%	1.92	1875	1.93	1865
50%	1.97	1830	1.97	1830
60%	2.02	1780	2.00	1800
70%	2.07	1740	2.04	1765
80%	2.11	1710	2.07	1740
90%	2.16	1670	2.11	1710
100%	2.21	1630	2.14	1680

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:

$$f_{pop(NPL)} = \frac{S_{NPL}}{S_{0\%}}$$

where, $f_{pop(NPL)}$ = driver population adjustment factor for saturation flow rate;

$s_{0\%}$ = saturation flow rate with 0% non-local driver population level; and

s_{NPL} = 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

Non-Local Driver Population Level	f_{pop} for Saturation Flow Rate		
	Left-Turn	Through	Average
0%	1.0	1.0	1.0
10%	0.97	0.98	0.98
20%	0.95	0.96	0.95
30%	0.92	0.94	0.93
40%	0.90	0.93	0.91
50%	0.88	0.91	0.89
60%	0.86	0.90	0.88
70%	0.84	0.88	0.86
80%	0.82	0.86	0.84
90%	0.80	0.85	0.83
100%	0.78	0.84	0.81

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

Area Type	Non-Local Driver Population Level
Residential Area	1% - 19%
Business Area	5% - 43%
Shopping Area	14% - 23%
Recreational Area	29% - 92%

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.

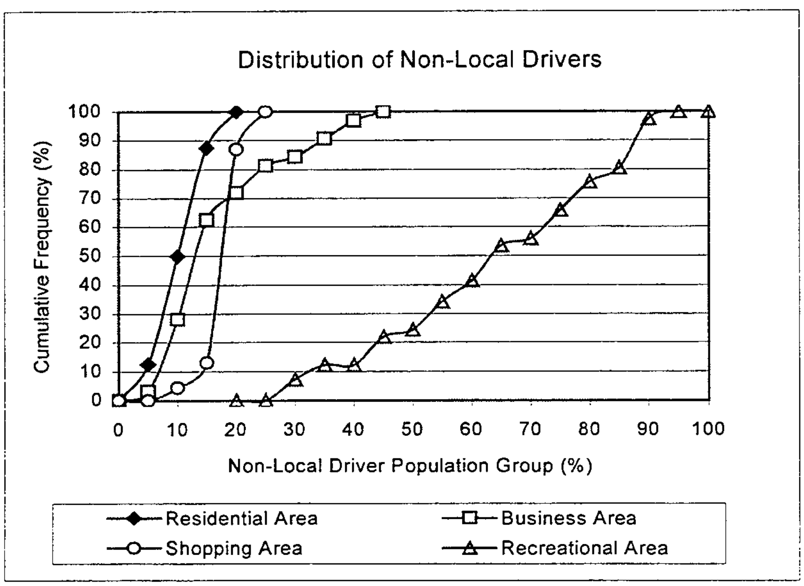

Figure 5-3 Distribution of Non-Local Drivers of Each Area Type

Table 5-9 Distribution of Non-Local Driver Population Level

	Non-Local Driver Population Level (%)			
Area Type	Minimum	Maximum	50th percentile	85th percentile
Residential Area	0	20	10	15
Business Area	5	45	15	30
Shopping Area	10	25	15	20
Recreational Area	30	95	65	85

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_{crit} was 2.68 at 5% level of significance. Since $F>F_{crit}$, 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_{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_{pop} value for the business areas. For the recreational areas with some residential communities nearby, the higher f_{pop} value should be selected, and for the recreational area close to the major attractions, such as Kissimmee, the lower f_{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

	f_{pop} for saturation flow rate		
Area Type	Range	50^{th} percentile	85^{th} percentile
Residential Area	1.0 – 0.95	0.98	0.96
Business Area	0.99 – 0.90	0.96	0.93
Shopping Area	0.97 – 0.94	0.96	0.95
Recreational Area	0.93 – 0.82	0.87	0.83

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

	East-bound			West-bound			South-bound		
	Through	Left	Right	Through	Left	Right	Through	Left	Right
10:00am	311	85		283		116		116	31
10:15am	311	85		292		151		126	17
10:30am	322	109		306		148		142	17
10:45am	316	76		304		104		122	33
Total	1260	355		1185		519		506	98

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

	Phase I	Phase II	Phase III
Movement	EB Left-Turn	WB & EB Through	SB Left Turn
Green Time (sec.)	15	55	40
Yellow Interval (sec.)	2	3	3
All Red Interval (sec.)	0	1	1

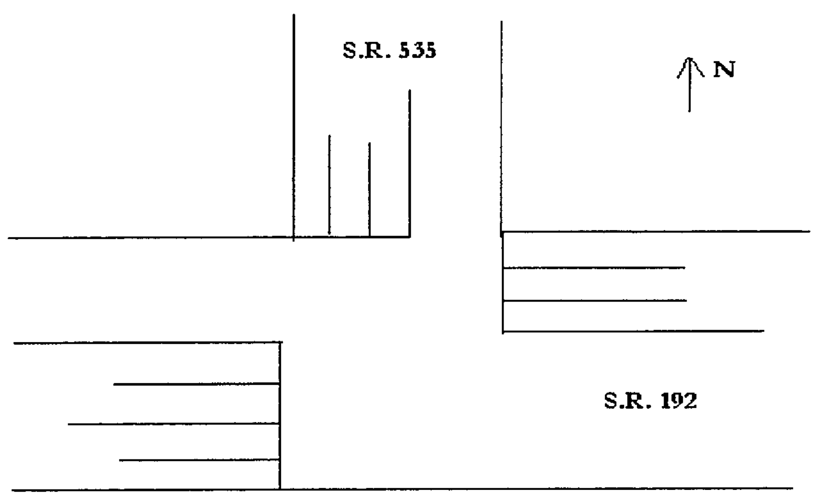

Figure 5-4: Geometric Condition of the Intersection S.R. 192 & S.R. 535

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

Approach	Lane Group	Before		After	
		Capacity	LOS	Capacity	LOS
EB	Left-Turn	396	F	325	F
	Through	1619	C	1328	D
	Overall EB	E		F	
WB	Through	1619	C	1328	D
	Right-Turn	725	D	594	E
	Overall WB	C		D	
SB	Left-Turn	1134	C	930	D
	Right-Turn	523	C	429	C
	Overall SB	C		D	
Overall Intersections		D		E	

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_{pop} in Terms of Non-Local Driver Population Level

Non-Local Driver Population Level	f_{pop} for Saturation Flow Rate
0%	1.0
10%	0.98
20%	0.95
30%	0.93
40%	0.91
50%	0.89
60%	0.88
70%	0.86
80%	0.84
90%	0.83
100%	0.81

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 extend, 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_{pop} in Terms of Area Type

	f_{pop} for saturation flow rate		
Area Type	Range	50^{th} percentile	85^{th} percentile
Residential Area	1.0 – 0.95	0.98	0.96
Business Area	0.99 – 0.90	0.96	0.93
Shopping Area	0.97 – 0.94	0.96	0.95
Recreational Area	0.93 – 0.82	0.87	0.83

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.

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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(cboMov
e, 1) & "_" & Format$(Now, "m_d") & "A" & "_origin.txt"
        DataFile = "c:\pop\data\" & Left(cboName, 1) & Left(cboIntersect, 1) & "_" & Left(cboMov
e, 1) & "_" & Format$(Now, "m_d") & "A" & "_headway.txt"
        AfterYellow = "c:\pop\data\" & Left(cboName, 1) & Left(cboIntersect, 1) & "_" & Left(cbo
Move, 1) & "_" & Format$(Now, "m_d") & "A" & "_sneaker.txt"
        ElseIf UCase(cboPeriod) = "PM" Then
        FileName = "c:\pop\data\" & Left(cboName, 1) & Left(cboIntersect, 1) & "_" & Left(cboMov
e, 1) & "_" & Format$(Now, "m_d") & "P" & "_origin.txt"
        DataFile = "c:\pop\data\" & Left(cboName, 1) & Left(cboIntersect, 1) & "_" & Left(cboMov
e, 1) & "_" & Format$(Now, "m_d") & "P" & "_headway.txt"
        AfterYellow = "c:\pop\data\" & Left(cboName, 1) & Left(cboIntersect, 1) & "_" & Left(cbo
Move, 1) & "_" & Format$(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,
        Write #2,
        Write #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 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
      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
            Vtype(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(1) = 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
  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
        Vtype(i) = "U Turn"
Open FileName For Append As #1
Write #1, i, "Left", Vtype(i), Ptime, Head(i)
Close #1
    Else
        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

'signal turns yellow'
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 truns 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(1) = 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
```

```
End Sub
```

APPENDIX B

SURVEY FORM FOR THE ZIPCODE INFORMATION

USF University of South Florida

ZIPCODE SURVEY

LOCATION: TIME:

CYCLE	SURVEY PERIOD							
	ZIP CODE							
	NO RESPONSE	FL						
		NON-FL						
	FOREIGNER							
	ZIP CODE							
	NO RESPONSE	FL						
		NON-FL						
	FOREIGNER							
	ZIP CODE							
	NO RESPONSE	FL						
		NON-FL						
	FOREIGNER							
	ZIP CODE							
	NO RESPONSE	FL						
		NON-FL						
	FOREIGNER							

APPENDIX C

REDUCED DATASET

REDUCED DATASET OF LEFT-TURN MOVEMENT

FILENAME	AREATYPE	POP LEVEL	ORIGINAL SATURATION HEADWAY	LANE WIDTH	LANE ADJUST	ADJUSTED SATURATION HEADWAY	START-UP LOST TIME	PYU
BTL0518P	Res	0.07	1.824	12	1.000	1.824	1.192	0.81
BTL0521P	Res	0.12	1.833	12	1.000	1.833	1.840	0.69
BTL0617P	Res	0.13	1.812	12	1.000	1.812	2.344	0.52
BTL0723P	Res	0.08	1.808	12	1.000	1.808	2.023	0.50
BTL0805P	Res	0.08	1.806	12	1.000	1.806	2.367	0.48
BTL0917P	Res	0.08	1.811	12	1.000	1.811	2.327	0.64
BTL0922P	Res	0.05	1.795	12	1.000	1.795	2.279	0.55
BTL1006P	Res	0.09	1.801	12	1.000	1.801	2.370	0.59
SWL0513A	Res	0.15	1.855	11	0.967	1.793	1.971	0.86
SWL0520P	Res	0.14	1.865	11	0.967	1.803	1.976	0.62
SWL0603A	Res	0.18	1.887	11	0.967	1.824	2.277	0.81
SWL0611A	Res	0.05	1.856	11	0.967	1.794	1.960	0.85
SWL0618A	Res	0.11	1.860	11	0.967	1.798	1.768	0.91
SWL0630A	Res	0.10	1.836	11	0.967	1.775	2.163	0.84
SWL0723A	Res	0.04	1.839	11	0.967	1.778	2.262	0.87
SWL0730A	Res	0.01	1.830	11	0.967	1.769	2.378	0.88
DCL0512A	Bus	0.05	1.770	12	1.000	1.770	1.939	0.46
DCL0514P	Bus	0.14	1.806	12	1.000	1.806	1.964	0.32
CDL0602P	Bus	0.17	1.713	12	1.000	1.713	2.476	0.19
CDL0616P	Bus	0.10	1.669	12	1.000	1.669	2.192	0.24
CDL0730P	Bus	0.17	1.719	12	1.000	1.719	2.221	0.23
CDL0827P	Bus	0.12	1.726	12	1.000	1.726	1.916	0.50
CDL0910P	Bus	0.13	1.779	12	1.000	1.779	1.758	0.57
CDL0929P	Bus	0.12	1.718	12	1.000	1.718	1.781	0.44
WBL0526P	Bus	0.29	1.839	12	1.000	1.839	1.612	0.77
WBL0603P	Bus	0.35	1.817	12	1.000	1.817	1.957	0.74
WBL0611P	Bus	0.38	1.779	12	1.000	1.779	1.995	0.81
WBL0629P	Bus	0.38	1.780	12	1.000	1.780	1.993	0.83
WBL0727P	Bus	0.43	1.847	12	1.000	1.847	1.988	0.74
WBL0804P	Bus	0.22	1.779	12	1.000	1.779	1.966	0.83

REDUCED DATASET OF LEFT-TURN MOVEMENT

FILENAME	AREATYPE	POP LEVEL	ORIGINAL SATURATION HEADWAY	LANE WIDTH	LANE ADJUST	ADJUSTED SATURATION HEADWAY	START-UP LOST TIME	PYU
WBL0817P	Bus	0.31	1.794	12	1.000	1.794	2.110	0.77
WBL0903P	Bus	0.24	1.766	12	1.000	1.766	1.773	0.79
SUL0522A	Sho	0.16	1.828	12	1.000	1.828	1.776	0.90
SUL1107A	Sho	0.21	1.841	12	1.000	1.841	1.049	0.90
SUL1122A	Sho	0.15	1.816	12	1.000	1.816	1.210	0.88
SUL1205A	Sho	0.19	1.823	12	1.000	1.823	1.349	0.91
SUL1206A	Sho	0.16	1.810	12	1.000	1.810	1.505	1.00
SUL0116A	Sho	0.20	1.831	12	1.000	1.831	1.149	0.88
SUL0123A	Sho	0.18	1.814	12	1.000	1.814	1.541	0.84
SBL0531P	Sho	0.19	1.861	11	0.967	1.799	2.095	0.76
SBL0628A	Sho	0.17	1.855	11	0.967	1.793	2.002	0.79
SBL0719A	Sho	0.15	1.879	11	0.967	1.787	1.591	0.80
SBL0802A	Sho	0.19	1.888	11	0.967	1.825	2.008	0.62
SBL0822A	Sho	0.16	1.871	11	0.967	1.809	1.776	0.59
SBL0829A	Sho	0.18	1.875	11	0.967	1.813	2.018	0.48
SBL0906A	Sho	0.18	1.860	11	0.967	1.798	1.818	0.76
SBL1002A	Sho	0.22	1.876	11	0.967	1.813	1.378	0.25
CIL0516P	Rec	0.42	1.952	12	1.000	1.952	2.142	0.88
CIL0530P	Rec	0.26	1.998	12	1.000	1.998	1.066	0.72
CIL0613P	Rec	0.43	2.011	12	1.000	2.011	2.000	0.75
CIL0912P	Rec	0.33	2.011	12	1.000	2.011	2.302	0.74
CIL0913P	Rec	0.29	1.975	12	1.000	1.975	2.390	0.74
CIL1011P	Rec	0.29	1.950	12	1.000	1.950	2.799	0.80
CIL1108P	Rec	0.34	1.945	12	1.000	1.945	1.363	0.86
SSL0919A	Rec	0.57	2.045	12	1.000	2.045	1.641	0.68
SSL1011A	Rec	0.62	2.021	12	1.000	2.021	2.386	0.84
SSL1025P	Rec	0.53	2.058	12	1.000	2.058	1.348	0.83
SSL1108A	Rec	0.61	2.045	12	1.000	2.045	1.694	0.86
SSL1114A	Rec	0.70	2.013	12	1.000	2.013	1.829	0.82
SSL1115A	Rec	0.71	2.102	12	1.000	2.102	1.774	0.77

REDUCED DATASET OF LEFT-TURN MOVEMENT								
FILENAME	AREATYPE	POP LEVEL	ORIGINAL SATURATION HEADWAY	LANE WIDTH	LANE ADJUST	ADJUSTED SATURATION HEADWAY	START-UP LOST TIME	PYU
SSL1213A	Rec	0.72	2.096	12	1.000	2.096	1.108	0.66
SSL0117A	Rec	0.63	2.050	12	1.000	2.050	2.281	0.75

REDUCED DATASET OF THROUGH MOVEMENT

FILENAME	AREATYPE	POP LEVEL	ORIGINAL SATURATION HEADWAY	LANE WIDTH	LANE ADJUST	ADJUSTED SATURATION HEADWAY	START-UP	PYU
BTT0311A	Res	0.16	1.777	12	1.000	1.777	1.723	74%
BTT0430A	Res	0.08	1.768	12	1.000	1.768	1.808	70%
BTT0514A	Res	0.06	1.767	12	1.000	1.767	1.831	81%
BTT0528A	Res	0.14	1.770	12	1.000	1.770	1.732	75%
BTT0610A	Res	0.10	1.750	12	1.000	1.750	1.919	75%
BTT0617A	Res	0.12	1.779	12	1.000	1.779	1.805	51%
BTT0714A	Res	0.08	1.757	12	1.000	1.757	1.780	69%
BTT0826A	Res	0.14	1.756	12	1.000	1.756	1.953	58%
SWT0511P	Res	0.16	1.769	12	1.000	1.769	1.714	75%
SWT0515A	Res	0.14	1.816	12	1.000	1.816	2.015	87%
SWT0601P	Res	0.14	1.800	12	1.000	1.800	2.076	82%
SWT0608P	Res	0.19	1.837	12	1.000	1.837	2.107	85%
SWT0615P	Res	0.12	1.812	12	1.000	1.812	2.009	84%
SWT0624P	Res	0.07	1.784	12	1.000	1.784	2.089	84%
SWT0803P	Res	0.11	1.797	12	1.000	1.797	2.135	75%
SWT0825P	Res	0.10	1.812	12	1.000	1.812	2.031	79%
DKT0609A	Bus	0.22	1.926	12	1.000	1.926	1.144	94%
DKT0616A	Bus	0.11	1.859	12	1.000	1.859	2.102	88%
DKT0625A	Bus	0.14	1.857	12	1.000	1.857	1.555	94%
DKT0701A	Bus	0.18	1.832	12	1.000	1.832	1.801	96%
DKT0716A	Bus	0.07	1.835	12	1.000	1.835	1.715	93%
DKT0722A	Bus	0.20	1.942	12	1.000	1.942	1.488	78%
DKT0729A	Bus	0.11	1.880	12	1.000	1.880	1.699	94%
DKT0818A	Bus	0.12	1.844	12	1.000	1.844	1.862	89%
CWT0604P	Bus	0.12	1.851	12	1.000	1.851	1.737	81%
CWT0610P	Bus	0.11	1.910	12	1.000	1.910	1.890	71%
CWT0618P	Bus	0.13	1.900	12	1.000	1.900	1.432	72%
CWT0623P	Bus	0.16	1.903	12	1.000	1.903	1.371	68%
CWT0728P	Bus	0.08	1.882	12	1.000	1.882	1.678	63%
CWT0813P	Bus	0.10	1.884	12	1.000	1.884	1.447	78%

REDUCED DATASET OF THROUGH MOVEMENT

FILENAME	AREATYPE	POP LEVEL	ORIGINAL SATURATION HEADWAY	LANE WIDTH	LANE ADJUST	ADJUSTED SATURATION HEADWAY	START-UP	PYU
CWT0908P	Bus	0.12	1.896	12	1.000	1.896	1.699	66%
CWT0924P	Bus	0.10	1.890	12	1.000	1.890	1.451	79%
SUT0523P	Sho	0.17	1.875	12	1.000	1.875	1.719	92%
SUT0614A	Sho	0.20	1.860	12	1.000	1.860	1.849	85%
SUT0726A	Sho	0.10	1.841	12	1.000	1.841	2.035	88%
SUT0801A	Sho	0.23	1.875	12	1.000	1.875	1.571	83%
SUT0823A	Sho	0.14	1.865	12	1.000	1.865	2.127	88%
SUT0905A	Sho	0.15	1.836	12	1.000	1.836	1.856	88%
SUT1010A	Sho	0.18	1.875	12	1.000	1.875	1.944	89%
SUT1024A	Sho	0.19	1.812	12	1.000	1.812	1.738	84%
SST0516A	Rec	0.89	2.034	12	1.000	2.034	2.839	84%
SST0530A	Rec	0.90	2.114	12	1.000	2.114	2.678	79%
SST0613A	Rec	0.92	2.125	12	1.000	2.125	1.929	80%
SST0620A	Rec	0.89	2.192	12	1.000	2.192	2.009	81%
SST0718A	Rec	0.88	2.119	12	1.000	2.119	1.536	78%
SST0830A	Rec	0.84	2.117	12	1.000	2.117	2.464	76%
SST0919A	Rec	0.80	2.027	12	1.000	2.027	2.991	76%
SST1011P	Rec	0.86	2.088	12	1.000	2.088	1.852	79%
IKT0913A	Rec	0.88	2.094	12	1.000	2.094	2.364	61%
IKT1011A	Rec	0.88	2.044	12	1.000	2.044	2.359	55%
IKT1025A	Rec	0.84	2.013	12	1.000	2.013	2.252	39%
IKT1108A	Rec	0.79	2.039	12	1.000	2.039	2.062	75%
IKT1114A	Rec	0.74	2.052	12	1.000	2.052	2.398	86%
IKT1115A	Rec	0.79	2.180	12	1.000	2.180	1.538	77%
IKT0117P	Rec	0.75	2.040	12	1.000	2.040	2.918	53%
IKT0124A	Rec	0.78	2.093	12	1.000	2.093	2.103	61%
IKT0503A	Rec	0.60	2.000	12	1.000	2.000	2.578	79%
IKT0504P	Rec	0.44	1.978	12	1.000	1.978	2.169	90%
SJT0503P	Rec	0.51	1.925	12	1.000	1.925	1.597	70%
SJT0504A	Rec	0.52	1.948	12	1.000	1.948	2.092	53%

REDUCED DATASET OF THROUGH MOVEMENT

FILENAME	AREATYPE	POP LEVEL	ORIGINAL SATURATION HEADWAY	LANE WIDTH	LANE ADJUST	ADJUSTED SATURATION HEADWAY	START-UP	PYU
IKT0510P	Rec	0.59	1.959	12	1.000	1.959	1.970	76%
IKT0511A	Rec	0.63	2.055	12	1.000	2.055	2.342	86%
IKT0512A	Rec	0.62	2.026	12	1.000	2.026	2.684	71%
SJT0510A	Rec	0.47	1.942	12	1.000	1.942	2.043	54%
SJT0511P	Rec	0.53	1.894	12	1.000	1.894	2.236	51%
SJT0512P	Rec	0.45	1.983	12	1.000	1.983	2.346	56%