Guidelines for Quantifying the Influence of Area Type and Other Factors on Saturation Flow Rate

FINAL REPORT

by

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16. Abstract

Chapter 16 of the 2000 *Highway Capacity Manual* (*HCM)* describes a procedure for estimating the saturation flow rate of a signalized intersection lane group. This procedure includes numerous saturation flow rate adjustment factors that are intended to adjust the base saturation flow rate such that the resulting saturation flow estimate accurately reflects conditions on the intersection approach. Unfortunately, the *HCM* procedure is not complete in terms of the adjustment factors offered. It does not offer factors that reflect the effect of traffic pressure, number of lanes in the lane group, or approach speed. Moreover, guidance for using its area-type adjustment factor is too subjective to yield confidence in the accuracy of the saturation flow rate estimate.

Saturation flow adjustment factors reflecting the effect of population, traffic pressure, number of lanes, and speed limit were developed for this research. A procedure for using these adjustment factors was also developed. This procedure is intended to be used with the *HCM* or FDOT's *Quality/Level of Service Handbook* to estimate the saturation flow rate for an intersection. Field measurements indicate that a base saturation flow rate of 1950 pc/h/ln is appropriate for Florida intersections.

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Prepared in cooperation with the State of Florida Department of Transportation.

EXECUTIVE SUMMARY

PROBLEM STATEMENT

The procedures in Chapter 16 of the 2000 *Highway Capacity Manual* (*HCM*) recognize that area demographics influence the capacity of a signalized intersection. Factors such as area population, average age, average income, and average trip distance are likely correlated with intersection capacity. The procedures in the *HCM* use a common base saturation flow rate that is unaffected by the number of lanes on the intersection approach. However, measurements of saturation flow rate in Florida indicate that flow rate tends to increase with the number of approach lanes. The *HCM* procedures indicate that right-turning vehicles have a lower saturation flow rate than that of a through movement. However, the *HCM* adjustment factor for right turns does not reflect the effect of turn radius.

The effect of area demographics, number of lanes, and right-turn radius have been reported in the literature but have not been sufficiently researched to the extent that they are suitable for practical application. The *HCM* authors recommend that analysts capture these effects by conducting field studies to calibrate the *HCM* procedures for use in their jurisdictions. While this recommendation is rational, it is not often implemented in practice because it is time consuming and expensive. Research is needed to develop saturation flow rate adjustment factors that can be used to estimate the effect of area demographics, number of lanes, and right-turn radius.

OBJECTIVES

The objectives of this research project are to:

1. quantify the base saturation flow rate for through movements, and
2. develop adjustment factors that account for the effects of area type, number of lanes, and the presence of right-turn vehicles.

FINDINGS AND CONCLUSIONS

The findings from a review of the literature indicate that some areawide influences and local conditions have been found to be correlated with saturation flow rate but are not included in the *HCM*. Areawide influences reflect differences between drivers in small towns and those in large cities. Local conditions reflect differences between drivers during off-peak and peak traffic hours. Local conditions also reflect the effect of constraints on the driving environment, as may be characterized by the number of approach traffic lanes or speed limit.

Saturation flow adjustment factors reflecting the effect of population, traffic pressure, number of lanes, and speed limit were developed for this research. An examination of factor trends indicates that saturation flow rate is lower in lightly populated (e.g., rural) areas.

Traffic pressure reflects the presence of aggressive drivers who are anxious to minimize their travel time in otherwise high-volume conditions (e.g., rush hour). They achieve their goal by accepting relatively short headways during queue discharge. The traffic pressure adjustment factor indicates that saturation flow rates increase as traffic queues increase.

The curb lane of a through lane group typically has a lower saturation flow rate than that of the adjacent through lanes. This lower rate is likely due to various sources of "friction" associated with the edge of the roadway (e.g., driveways, pedestrians, signs, poles, and other street furniture) and its proximity to moving traffic. When this effect is averaged across all lanes in the through lane group, saturation flow rate is found to increase with increasing number of lanes.

The physical elements of the urban street environment tend to be correlated with the posted speed limit. These elements include driveway density, roadside development, sidewalks, on-street parking, and street segment length. When these elements are present in the vicinity of the intersection, they are likely to influence saturation flow rate. This influence was observed in field measurements of saturation flow rate at intersections on streets having a range of speed limits. Specifically, lower saturation flow rates were found on lower-speed streets.

Saturation flow rate adjustment factors for lanes with right-turn movements and for those with heavy vehicles were also developed. Saturation flow rate was found to decrease with an increase in right-turn vehicle percentage and with an increase in heavy-vehicle percentage. Right-turn radius was not found to be correlated with saturation flow rate. A comparison with similar factors reported in the *HCM* indicates that right-turn vehicles and heavy vehicles have less of an effect on saturation flow rate than suggested by the *HCM* factors.

A base saturation flow rate of 1950 pc/h/ln was estimated from the data collected at numerous Florida intersections. This rate is slightly larger than the 1900 pc/h/ln value currently used in FDOT's *Quality/Level of Service Handbook* (*Handbook*). The base rate of 1950 pc/h/ln and the aforementioned adjustment factors are recommended for inclusion in the next edition of the *Handbook*.

BENEFITS

The products of this research will be implemented through their incorporation in the *Handbook*. The *Handbook* is used by engineers, planners, and decision makers on a statewide basis in the development and review of roadway users' quality/level of service at planning and preliminary engineering levels. The implementation of the research products will improve the accuracy of the level of service estimates, which should translate into a more cost-effective distribution of construction project resources and a more equitable cost-share distribution among developers.

This research project was directed by James A. Bonneson, P.E., of the Texas Transportation Institute. For more information, contact: Gina Bonyani, project manager, at (850) 414-4707 (994-4707 SunCom) or gina.bonyani@dot.state.fl.us.

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CHAPTER 1. INTRODUCTION

OVERVIEW

The procedures in Chapter 16 of the 2000 *Highway Capacity Manual* (*1*) (*HCM*) recognize that area demographics influence the capacity of a signalized intersection. Factors such as area population, average age, average income, and average trip distance are likely correlated with intersection capacity. In general, areas with higher densities, lower average age, higher income, and longer trip distance are likely to have higher capacities. These factors are logically tied to the value that drivers place on their time and the anxiety they feel when it is not used efficiently.

The procedures in the *HCM* use a common base saturation flow rate that is unaffected by the number of lanes on the intersection approach. However, measurements of saturation flow rate in Florida indicate that flow rate tends to increase with the number of approach lanes. It is possible that this increase reflects a variation in saturation flow rate among the inside, outside, and intermediate lanes on the approach. Regardless of the reason, significant implications exist in the analysis of two-, four-, or six-lane arterial streets should their capacity not be a direct multiple of their number of lanes.

The *HCM* procedures indicate that right-turning vehicles have a lower saturation flow rate than that of a through movement. This lower rate is likely due to the relatively slow speed required to negotiate the right-turn maneuver. The *HCM* procedure does not reflect a sensitivity to right-turn radius. Also, the *HCM* procedure indicates that there is no difference between the saturation flow rate of an exclusive right-turn lane and that of a shared through plus right-turn lane with 100 percent right-turn vehicles in it. It is possible that driver uncertainty about the distribution of vehicles in the shared lane may result in it having a lower flow rate than would be expected by simple extrapolation between the flow rates of a through lane and an exclusive right-turn lane.

The effect of area demographics, number of lanes, and right-turn presence have been reported in the literature but have not been sufficiently researched to the extent that they are suitable for practical application. The *HCM* authors recommend that analysts capture these effects by conducting field studies to calibrate the *HCM* procedures for use in their jurisdictions. While this recommendation is rational, it is not often implemented in practice because it is time consuming and expensive. Instead, default values are routinely used that reflect averages on a nationwide basis. Analyses based on default values (as opposed to calibrated values) may lack accuracy because they do not adequately reflect the behavior of drivers in the area of interest.

OBJECTIVE AND SCOPE

The objectives of this research project are to: (1) quantify the base saturation flow rate for through movements and (2) develop adjustment factors that account for the effects of area type, number of lanes, and the presence of right-turn vehicles. The focus of the research is on the through traffic lanes at signalized intersections in Florida. To the extent possible, the research also examined flow rates in exclusive right-turn lanes. The approach lane combinations considered include:

through lanes (on one-, two-, and three-lane approaches), shared through and right-turn lanes, single-lane exclusive right-turn lanes, and single-lane continuous right-turn lanes. The guidelines and factors are applicable to intersections in urban and rural areas.

RESEARCH APPROACH

The research approach is based on a 20-month program of research development and evaluation that was directed at producing information engineers could use to estimate the saturation flow rate at signalized intersections. Seven tasks were specified in the proposed work plan as needed to achieve the project objectives. These tasks are:

1. Review State of the Art and Practice.
2. Examine Factors That Influence Saturation Flow Rate.
3. Develop Data Collection Plan.
4. Conduct Field Studies.
5. Calibrate and Validate the Saturation Flow Rate Adjustment Factors.
6. Develop Techniques for Using the Adjustment Factors in an HCM Analysis.
7. Prepare Final Report.

Tasks 1 through 4 were executed during the first year of the project. During these tasks, the project team identified a series of intersection approach study sites that collectively reflected a range of area types, lanes, and right-turn volumes. Researchers then collected saturation flow rate data at each of these intersections using videotape recorders. Finally, the data were extracted from the videotape during replay in a laboratory setting and entered into a saturation flow rate database.

Tasks 5 through 7 were executed during the last eight months of the project. The saturation flow rate database was used to calibrate a set of equations. Each equation was developed to model the effect of one factor on saturation flow rate. Equations were developed for the following factors:

- right turns,
- number of lanes,
- area population,
- traffic pressure,
- heavy vehicles, and
- speed limit.

This report documents the research conducted and the research findings, and presents guidelines for using the calibrated saturation flow rate adjustment factors.

CHAPTER 2. LITERATURE REVIEW

OVERVIEW

This chapter summarizes a review of the literature on topics related to factors that affect saturation flow rate. Initially, the saturation flow rate prediction equation provided in the *HCM* is described. In this part of the chapter, the effect of several factors on saturation flow rate is discussed. These factors include: right-turn radius, area type, area population, number of lanes, and traffic pressure. In the next section, methodological issues related to saturation flow rate estimation from field data are discussed. Finally, this chapter is concluded with a reexamination of the saturation flow rate data collected by FDOT District 4 in a previous research project. The objective of this examination is to confirm the reported effect of various factors on saturation flow rate.

SATURATION FLOW RATE PREDICTION EQUATION

Chapter 16 of the *HCM* provides the following equation for computing the saturation flow rate for a lane group:

$$s = s_o \, N \, f_w \, f_{HV} \, f_g \, f_p \, f_{bb} \, f_a \, f_{LU} \, f_{LT} \, f_{RT} \, f_{Lpb} \, f_{Rpb} \quad \textbf{(1)}$$

where:

s = saturation flow rate for the subject lane group, expressed as a total for all lanes in the lane group, veh/h;
s_o = base saturation flow rate per lane, pc/h/ln (i.e., passenger cars/hour/lane);
N = number of lanes in lane group;
f_w = adjustment factor for lane width;
f_{HV} = adjustment factor for heavy vehicles in traffic stream;
f_g = adjustment factor for approach grade;
f_p = adjustment factor for existence of a parking lane and parking activity adjacent to lane group;
f_{bb} = adjustment factor for blocking effect of local buses that stop within intersection area;
f_a = adjustment factor for area type;
f_{LU} = adjustment factor for lane utilization;
f_{LT} = adjustment factor for left turns in lane group;
f_{RT} = adjustment factor for right turns in lane group;
f_{Lpb} = pedestrian adjustment factor for left-turn movements; and
f_{Rpb} = pedestrian adjustment factor for right-turn movements.

The default base saturation flow rate offered in the *HCM* is 1900 pc/h/ln. For each adjustment factor, the *HCM* provides a table of values or an equation (or equations) that can be used to determine the value of the adjustment factor. These equations are not repeated in this chapter.

Equation 1 indicates that the lane group saturation flow rate is equal to the base saturation flow rate for a lane, multiplied by the number of lanes in the lane group, and any of the applicable adjustment factors. The multiplicative form of the adjustments implies that a change in any one factor has an effect on the magnitude of all other applicable adjustment factors. It is used extensively in the research literature and in the highway capacity manuals of other countries. An additive form of adjustment has also been offered in the research literature (see, for example, Kimber et al. [*2*]). With this form, the saturation flow rate is estimated as a base flow rate with adjustments for grade, lane width, etc. made by adding or subtracting a small value.

The advantage of the multiplicative form of adjustment is that it indirectly accounts for the interaction among adjustment factors by: (1) moderating the impact of multiple factors that reduce flow rate and (2) amplifying those factors that increase flow rate. The moderation of factor interactions ensures that the predicted saturation flow rate is always a positive quantity, a trait not shared by the additive form. For example, with the multiplicative form, the product of the "grade" and "heavy-vehicle" factors captures the effect of grade on heavy vehicles in the resulting saturation flow estimate. In contrast, the additive form assumes that the magnitude of the reduction for grade is independent of that for heavy vehicles, which is illogical.

Effect of Right-Turn Maneuver on Saturation Flow Rate

The effect of right-turn radius on saturation flow rate is reported in several research and engineering reference documents (*2, 3, 4*). The general trend is one of increasing saturation flow rate with right-turn radius. This relationship is shown in Figure 1 in terms of the adjustment factor for right turns (f_{RT}).

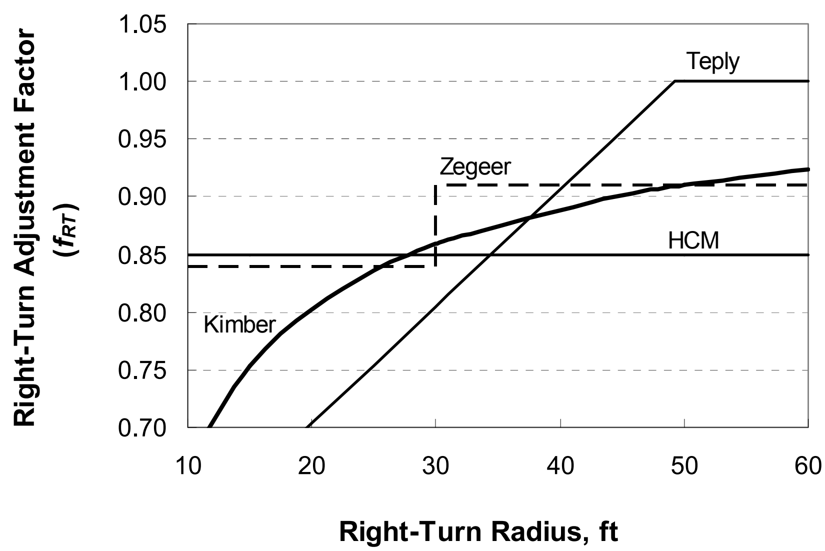


Figure 1. Reported Effect of Right-Turn Radius on Saturation Flow Rate.

As shown in Figure 1, there is little agreement on the exact effect of right-turn radius. For a 20-ft radius, values for the adjustment factor vary from 0.70 to 0.85. For a 60-ft radius, the adjustment factors vary from 0.85 to 1.00. The factor of 0.85 offered in the *HCM* does not reflect an influence of right-turn radius. It appears to underestimate the effect of radius for sharp radii and overestimate the effect for flat radii.

When the outside through lane is shared with right-turning vehicles, the resulting saturation flow rate is typically estimated as a weighted average of the discharge rates of the two traffic movements. Kimber et al. (*2*) recommend computing an adjustment factor using the weighted average of the saturation headways of the through and right-turn movements. The form of this equation is:

$$f_{RT} = \frac{1}{1 + P_{RT}(E_{RT} - 1)} \tag{2}$$

with

$$E_{RT} = 1 + \frac{4.92}{r} \tag{3}$$

where:

P_{RT} = proportion of right-turn vehicles in the lane group;

E_{RT} = right-turn equivalency factor expressed in terms of an equivalent number of through passenger cars; and

r = right-turn radius, ft.

The *HCM* uses a similar weighted average headway technique to define an adjustment factor for heavy vehicles and for shared through plus left-turn lanes.

For the right-turn adjustment factor, the *HCM* and Zegeer (*3*) both recommend the use of a weighted average of the saturation flow rates of the two traffic movements. The form of this equation is:

$$\begin{aligned} f_{RT} &= 1.0\,(1 - P_{RT}) + \frac{1}{E_{RT}} P_{RT} \\ &= 1.0 - \left(1 - \frac{1}{E_{RT}}\right) P_{RT} \end{aligned} \tag{4}$$

For the *HCM*, the recommended right-turn equivalency factor E_{RT} is 1.18 (= 1/0.85). For Zegeer (*3*), the equivalency factor is 1.19 for radii less than 30 ft and 1.10 otherwise.

The trends yielded by Equations 2 and 4 are shown in Figure 2 for a 25-ft turn radius. When there are no right turns in the shared lane, a right-turn adjustment factor of 1.0 is obtained. In contrast, when there are 100 percent right turns in the shared lane, it acts as a defacto right-turn lane and an adjustment factor of 0.84 to 0.85 is obtained.

The differences between the various tend lines in Figure 2 are primarily a reflection of their different sensitivity to radius. The differences in averaging method are more subtle. The weighted-average-headway approach introduces a slight curvature in the trend line whereas the weighted-average-flow rate approach yields a linear trend line. The amount of curvature is typically slight, as can be seen in the trend line labeled “Kimber” in Figure 2.

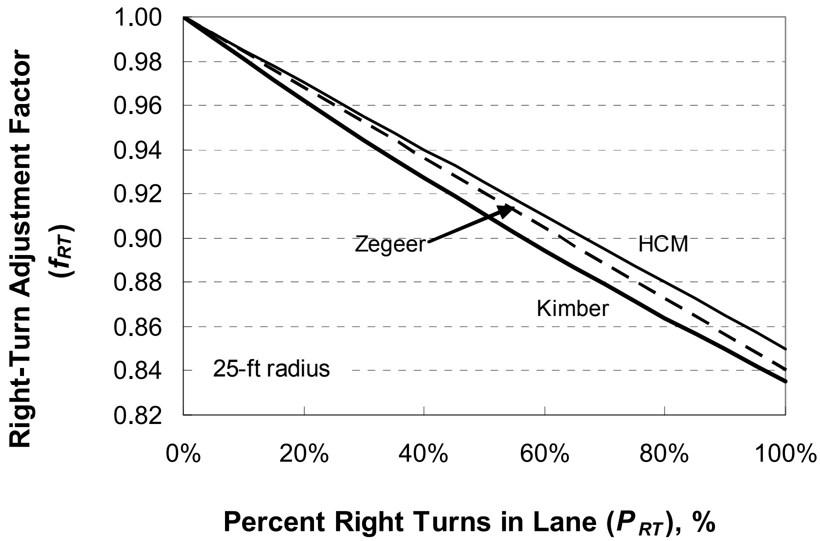


Figure 2. Reported Effect of Right-Turn Percentage on Saturation Flow Rate.

From a theoretic standpoint, the correct approach for determining saturation flow adjustment factors for mixed traffic streams is to use the weighted-average-headway approach (i.e., Equation 2). However, as suggested by the very slight curvature in the "Kimber" equation in Figure 2, the differences between the two approaches are minimal for most practical applications.

Relationship between Area Type and Saturation Flow Rate

The *HCM* includes an adjustment factor for "area type" to account for the relative inefficiency of intersections in business districts, relative to those in other areas. It is appropriate for use at intersections in a central business district (CBD) where intersection geometry, pedestrian flow, and roadside friction are more restrictive than otherwise reflected in the saturation flow rate obtained from Equation 1. The adjustment factor recommended in the *HCM* for these conditions is 0.90.

Researchers have evaluated the relationship between saturation flow rate and area type (*3, 5, 6*). The findings from these efforts are summarized in Table 1. The factors listed in this table indicate that saturation flow rates in the CBD are lower than those in other areas; however, the amount of the reported decrease is somewhat uncertain and varies from 0.90 to 0.99. Le et al. (*5*) and Zegeer (*3*) found that saturation flow rates were larger in residential areas than in outlying commercial districts or in the CBD. In contrast, Le et al. (*5*) found that flow rates in recreational areas were much lower than those in other areas and similar to those found in the CBD. This decrease was attributed to the high proportion of drivers unfamiliar to the local street system in recreational areas.

Table 1. Reported Relationship between Area Type and Saturation Flow Rate.

Reference	Adjustment Factors Categorized by Area Type			
	Central Business District	Outlying Commercial District [1, 2]	Residential Area	Recreational Area
HCM (*1*)	0.90	1.00	1.00	not available
Le et al. (*5*)	not available	1.00	1.03	0.92
Zegeer (*3*)	0.99	1.00	1.01	not available
Agent & Crabtree (*6*)	0.97	1.00	1.00	not available

Notes:
1 - Outlying commercial district is determined to be equivalent to "fringe area," as defined by Agent & Crabtree (*6*).
2 - Commercial lane use is defined to include business, shopping, or both.

Relationship between Area Population and Saturation Flow Rate

Two researchers have identified a relationship between the population of the area surrounding the subject intersection and the saturation flow rate of its traffic movements (*3, 6*). Both researchers found that saturation flow rate is lower in areas with small populations. Presumably, drivers in smaller cities and towns are less aggressive than those in large urbanized areas and prefer to drive at a less hurried pace. The relationship between area population and saturation flow rate, as recommended by these researchers, is shown in Figure 3.

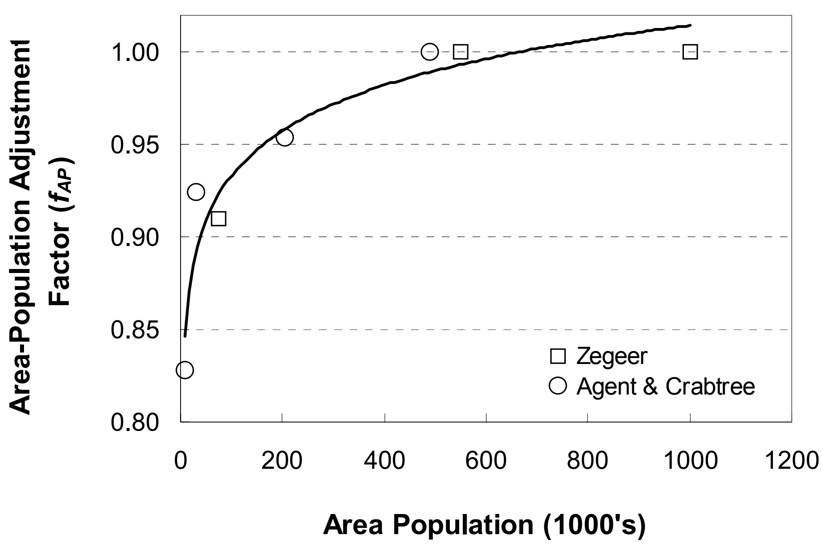


Figure 3. Reported Relationship between Area Population and Saturation Flow Rate.

The data points shown in Figure 3 represent the adjustment factors recommended by Zegeer (*3*) and by Agent and Crabtree (*6*). The trend line shown represents a "best fit" to these factors. The shape of the trend line suggests that saturation flow rate is highly sensitive to population in areas with population less than 100,000. For area populations above 500,000, the effect of population appears to be negligible.

Effect of Number of Lanes on Saturation Flow Rate

McMahon et al. (*7*) examined the effect of number of lanes on saturation flow rate. They studied the through movements on four approaches to each of 12 intersections in the Miami, Florida, area (i.e., FDOT District 4). Saturation flow data were measured for each of 2901 signal cycles. They found that the saturation flow rate of through movements on three-lane approaches was 1910 pc/h/ln. That for two-lane approaches was 1790 pc/h/ln and that for one-through-lane approaches was 1670 pc/h/ln. Based on these findings, they concluded that saturation flow rate increases with the number of through lanes on the approach.

Figure 4 illustrates the value of the adjustment factor for number of lanes based on the saturation flow rates reported by McMahon et al. (*7*). The adjustment factors shown were derived using a base saturation flow rate of 1800 pc/h/ln.

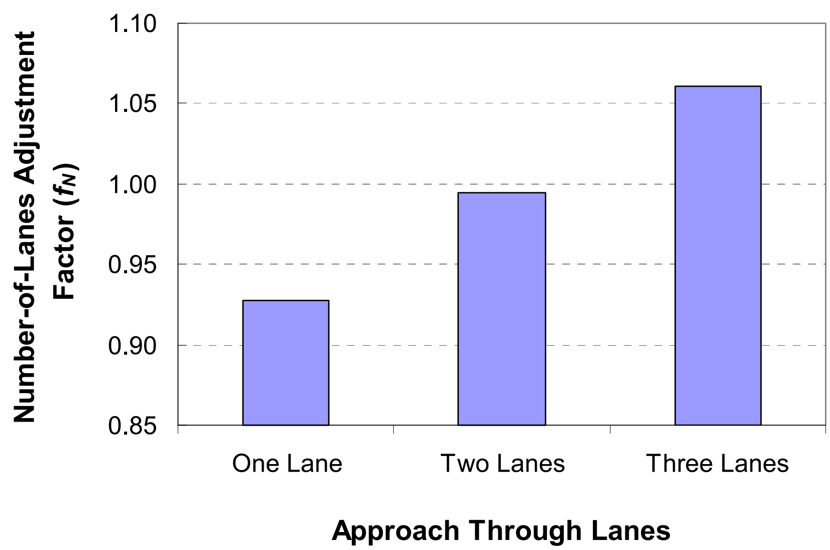


Figure 4. Reported Effect of Number of Lanes on Saturation Flow Rate.

Effect of Traffic Pressure on Saturation Flow Rate

In a study of saturation flow rate at high-type intersections and interchanges, Bonneson (*8*) found that the number of vehicles in queue had an effect on saturation flow rate. Specifically, he found that the saturation headway measured for the fifth car in queue decreased with increasing queue length. This same trend was found for the sixth, seventh, eighth, etc. queue positions. He called this effect that of "traffic pressure." It is believed to result from the presence of aggressive drivers (e.g., commuters) who are anxious to minimize their travel time in otherwise high-volume conditions. They achieve their goal by accepting relatively short headways during queue discharge.

More recently, Bonneson and Messer (*9*) examined the effect of traffic pressure at 12 interchanges and 12 intersections (one approach per location). Their analysis of 7704 saturation headways led to the development of the following traffic-pressure adjustment factor:

$$f_{TP} = \frac{1}{1.07 - 0.00486\, v_l} \tag{5}$$

where:

f_{TP} = adjustment factor for traffic pressure; and
v_l = average flow rate per lane (i.e., traffic pressure) for the lane group, veh/ln/cycle.

This equation was derived based on the premise that 15 veh/ln/cycle represented the "base" condition. The effect of traffic pressure on saturation flow rate is shown in Figure 5. The trend line indicates that intersection approaches with relatively low volume (and short queues) will have a low saturation flow rate and those with high volume will have a high saturation flow rate.

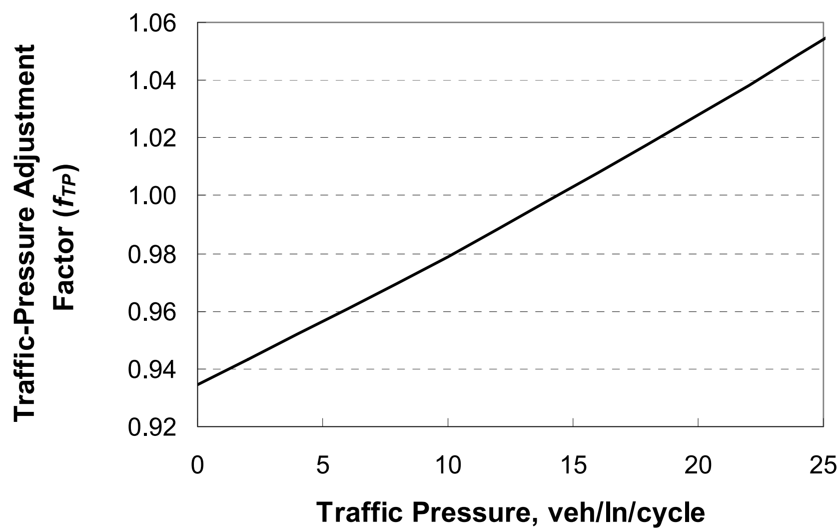


Figure 5. Reported Effect of Traffic Pressure on Saturation Flow Rate.

METHODOLOGICAL ISSUES IN SATURATION FLOW RATE ESTIMATION

This part of the chapter describes the appropriate method for estimating saturation flow rate using field data. Several field survey techniques are described in the literature for estimating the saturation flow rate of a specific lane or lane group. Some methods require counting the number of discharged vehicles in a set time interval; others require observers to record the discharge time of a specified pair of vehicles. The *HCM* follows the latter technique by specifying the recording of discharge times of the fourth and last queued vehicle. Each of these methods yields a slightly different saturation flow rate estimate for the same traffic lane. Teply (*4*) provides a detailed discussion of several methods and identifies their differences.

Regardless of the survey method used, the accuracy of the saturation flow rate estimate for a specific lane (or lane group) is highly dependent on the method used to aggregate the data that are recorded each signal cycle. The underlying issue is whether to base the computation of overall saturation flow rate on individual measurements of average headway per cycle or on individual observations of saturation flow rate per cycle. The two methods yield estimates of overall saturation flow rate that differ by about 50 veh/h/ln. The reason for the difference in the two methods is due to two factors: (1) the cycle-based statistics (i.e., average headway per cycle and average saturation flow rate per cycle) have a random component and (2) one statistic is the reciprocal of the other. From a mathematical standpoint, a randomly distributed variable that is converted by reciprocal and averaged will not equal the reciprocal of the average value of the randomly distributed variable.

The choice among the two averaging methods can be made clear by examining the equation of cycle capacity. The appropriate averaging method is the one that yields an unbiased estimate of cycle capacity. Consider the following equation:

$$Tn_i - T4_i = (n_i - 4) \times h_i \qquad \textbf{(6)}$$

where:

Tn_i = observed discharge time of the n^{th} queued vehicle during cycle i, s;
$T4_i$ = observed discharge time of the 4^{th} queued vehicle during cycle i, s;
n_i = number of queued vehicles observed during cycle i; and
h_i = average saturation headway for cycle i, s/veh.

Recognizing that $T4_i - 4\,h_i$ is equal to start-up lost time $l_{T,i}$ and that Tn_i - $l_{T,i}$ for a phase operating at its capacity equals the effective green time g, the following equation is obtained by substitution:

$$g = n_{c,i} \times h_i \qquad \textbf{(7)}$$

where:

$n_{c,i}$ = maximum number of queued vehicles that discharge during cycle i.

The variables with the subscript i are random variables when measured over a successive number of cycles. They can be converted to hourly averages as follows:

$$g = \frac{1}{M}\sum_{1}^{M} n_{c,i} \times \frac{1}{M}\sum_{1}^{M} h_i \qquad \textbf{(8)}$$

where:

M = number of signal cycles in the hour of observation (= 3600/C); and
C = signal cycle length, s.

Substituting c for the sum of $n_{c,i}$, 3600/C for M, and $\bar{h}$ for the second summation term (i.e., $1/M\sum h_i$), the following equation is obtained:

$$g = \frac{c\,C}{3600} \times \bar{h} \qquad \textbf{(9)}$$

where:

c = phase capacity, veh/h; and

$\bar{h}$ = overall average saturation headway, s/veh.

Rearranging terms, the following equation is obtained for estimating phase capacity:

$$\begin{aligned} c &= \frac{3600}{\bar{h}} \frac{g}{C} \\ &= \bar{s} \frac{g}{C} \end{aligned} \qquad \textbf{(10)}$$

with

$$\bar{h} = \frac{1}{M} \sum_{1}^{M} h_i \qquad \textbf{(11)}$$

and

$$\bar{s} = \frac{3600}{\bar{h}} \qquad \textbf{(12)}$$

Equation 10 indicates that the unbiased estimate of phase capacity is based on the calculation of an overall average saturation headway $\bar{h}$, as obtained from Equation 11. This overall average saturation headway can also be converted into the overall average saturation flow rate $\bar{s}$ using Equation 12. Either variable can then be used in Equation 10 to obtain an unbiased estimate of capacity.

Computing the saturation flow rate for each cycle observation s_i and then averaging these cycle-based saturation flow rates to obtain $\bar{s}$ will not yield an accurate estimate of phase capacity. The estimate of saturation flow rate obtained in this manner will be biased and exceed that obtained from Equation 12 by about 50 veh/h/ln (the exact amount of this bias is dependent on the variability in the data).

EXAMINATION OF McMAHON DATA

This section describes a brief reexamination of the saturation flow rate data collected by McMahon et al. (*7*). The objective of this examination is to investigate the influence of traffic pressure and area population on saturation flow rate using data previously collected at Florida intersections. The data used for this analysis are listed in Table 2. These data represent the passenger car saturation flow rates measured on the approaches to 12 intersections in FDOT District 4. The data reflect the saturation flow rates of the through-vehicle traffic stream at each intersection. Those approaches in the original database that have shared through plus turn lanes were excluded from this examination.

Table 2. McMahon Saturation Flow Rate Data.

County	County Pop. (1000's)	Site [1]	Approach Travel Direction	Saturation Flow Rate, pc/h/ln	Std. Deviation, pc/h/ln	Corrected Flow Rate, pc/h/ln	Cycle Length, s	Speed Limit, mph	Approach Through Lanes
Palm Beach	1131	1	WB	1890	200	**1869**	161	45	2
Palm Beach	1131	1	NB	1730	289	**1682**	152	40	3
Palm Beach	1131	1	EB	1830	215	**1805**	150	45	2
Palm Beach	1131	1	SB	1750	273	**1707**	148	40	3
Palm Beach	1131	2	WB	1900	432	**1802**	152	40	2
Palm Beach	1131	2	EB	1720	366	**1642**	152	40	2
Palm Beach	1131	3	WB	2030	281	**1991**	152	45	3
Palm Beach	1131	3	NB	1890	315	**1838**	154	45	3
Palm Beach	1131	3	EB	1890	366	**1819**	152	45	3
Broward	1623	4	WB	1890	301	**1842**	152	45	3
Broward	1623	4	NB	1940	250	**1908**	140	45	3
Broward	1623	4	EB	1800	236	**1769**	140	45	3
Broward	1623	4	SB	1880	392	**1798**	152	45	3
Broward	1623	5	WB	1960	203	**1939**	183	n.a.	3
Broward	1623	5	NB	1800	314	**1745**	166	n.a.	3
Broward	1623	5	EB	1860	397	**1775**	152	n.a.	3
Broward	1623	5	SB	1810	301	**1760**	152	n.a.	3
Broward	1623	6	WB	1990	261	**1956**	171	45	3
Broward	1623	6	NB	2000	220	**1976**	186	45	3
Broward	1623	6	EB	1930	329	**1874**	177	45	3
Broward	1623	6	SB	2060	213	**2038**	177	45	3
Martin	127	7	WB	1650	128	**1640**	186	40	1
Martin	127	7	NB	1800	387	**1717**	190	45	2
Martin	127	7	EB	1700	313	**1642**	190	40	1
Martin	127	7	SB	1910	361	**1842**	186	45	2
Martin	127	8	WB	1780	200	**1758**	212	45	2
Martin	127	8	NB	1990	248	**1959**	216	45	3
Martin	127	8	SB	1820	429	**1719**	216	45	3
St. Lucie	193	9	WB	1600	365	**1517**	93	30	1
St. Lucie	193	9	NB	1770	359	**1697**	86	30	1
St. Lucie	193	9	EB	1700	240	**1666**	91	35	1
St. Lucie	193	9	SB	1790	261	**1752**	87	45	1
St. Lucie	193	10	NB	1810	375	**1732**	204	45	3
St. Lucie	193	10	EB	1590	264	**1546**	204	30	1
St. Lucie	193	10	SB	1930	244	**1899**	200	45	3
Indian River	113	11	WB	1720	359	**1645**	216	45	2
Indian River	113	12	WB	1700	385	**1613**	113	30	1
Indian River	113	12	SB	1810	373	**1733**	119	45	1

Note:
1 - Each site number represents one intersection.

The saturation flow rates reported in column 5 of Table 2 were computed by McMahon et al. using the average of the saturation flow rates measured during each signal cycle. They computed an average saturation headway for each cycle. They then used the reciprocal of this average to estimate the saturation flow rate for each cycle. These cycle-based saturation flow rates were then averaged over all cycles to compute the overall saturation flow rate shown in column 5 of Table 2. However, as discussed in the previous section, this averaging method overestimates the true overall average saturation flow rate.

The saturation flow rate that should be used in Equation10 to estimate capacity can be obtained from the data reported by McMahon et al. by making an adjustment to the reported overall average saturation flow rate. The following equations were used to make this adjustment:

$$\bar{s} = \bar{s}_{McMahon} - 3600\ \sigma_{\bar{h}}^2 \left(\frac{\bar{s}_{McMahon}}{3600} \right)^3 \qquad \textbf{(13)}$$

with

$$\sigma_{\bar{h}} = \frac{\sigma_{s,\,McMahon}}{3600} \left(\frac{3600}{\bar{s}_{McMahon}} \right)^2 \qquad \textbf{(14)}$$

where:

$\bar{s}$ = overall average saturation flow rate, pc/h/ln;
$\bar{s}_{McMahon}$ = reported overall average saturation flow (from McMahon et al.), pc/h/ln;
$\sigma_{\bar{h}}$ = standard deviation of the average headway, s/veh; and
$\sigma_{s\ McMahon}$ = standard deviation of the average saturation flow rate reported by McMahon et al., pc/h/ln.

The corrected overall average saturation flow rate is shown in column 7 of Table 2. The adjustments made to each reported average saturation flow rate ranged from a reduction of 10 to 100 pc/h/ln for each site. The average reduction was 53 pc/h/ln. The corrected averages are what would have been reported by McMahon et al. if they had used Equation 11 for each site to estimate its overall average saturation headway, and then used Equation 12 to estimate the overall average saturation flow rate.

Linear regression analysis was used to evaluate the effect of population, number of lanes, and traffic pressure on saturation headway. The results of this analysis indicated that both population and traffic pressure were correlated with saturation headway. The nature of their relationship with headway is shown in Figure 6. The data for the three Indian River County sites are not shown in this figure because they are too few in number to detect a discernable trend.

The scatter in the data for any one trend line is relatively large; however, the “best-fit” trend lines are fairly consistent in their downward slope. This slope is consistent with the effect of traffic pressure, as described by Equation 5.

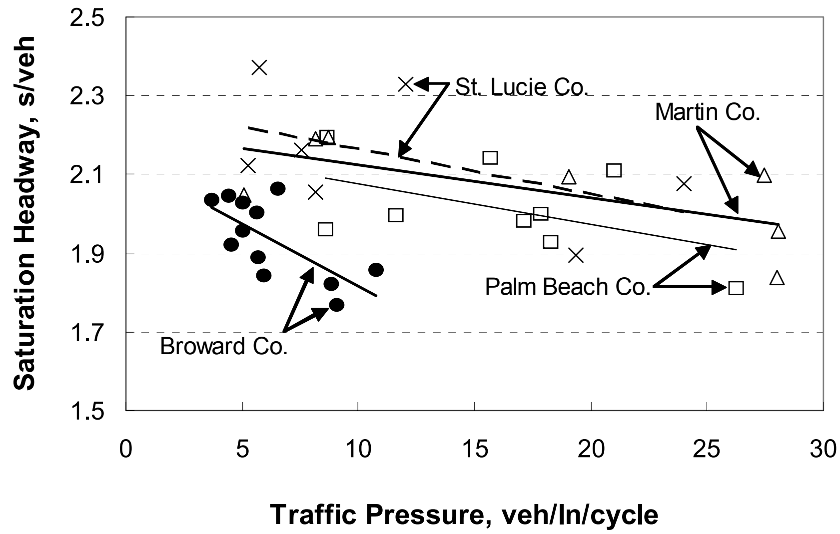


Figure 6. Effect of Traffic Pressure and Population on Saturation Headway.

The effect of population is also suggested by the relationship between the various trend lines in Figure 6. The data in Table 2 indicate that Broward County has the largest population. For a given traffic pressure, the trend lines in Figure 6 indicate that Broward County has the lowest saturation headway. The trend continues with Palm Beach County having a lower population and a larger saturation headway, relative to Broward County. St. Lucie and Martin Counties have even lower populations and corresponding higher saturation headways.

A linear regression model was developed using the data in Table 2. The dependent variable used in the model is the effective saturation flow rate adjustment factor, as computed using the ratio of the corrected saturation flow rate in Table 2 to 1800 pc/h/ln. The independent terms in the model were the computed traffic pressure (i.e., vehicles per lane per cycle) and the natural log of area population. The natural log of population was used because of the nonlinear relationship between population and saturation flow rate, as suggested by the trend in Figure 3. The results of this analysis are shown in Figures 7 and 8.

Figure 7 compares the regression relationship from the McMahon data with that obtained from Equation 5. The slopes of the two trend lines are very similar and suggest that traffic pressure in District 4 is similar in effect to that found in other states.

Figure 8 compares the regression relationship from the McMahon data with the individual adjustment factors recommended by Zegeer (*3*) and by Agent and Crabtree (*6*) for various population levels. The fit is quite good and tends to confirm the effect of population on saturation flow rate.

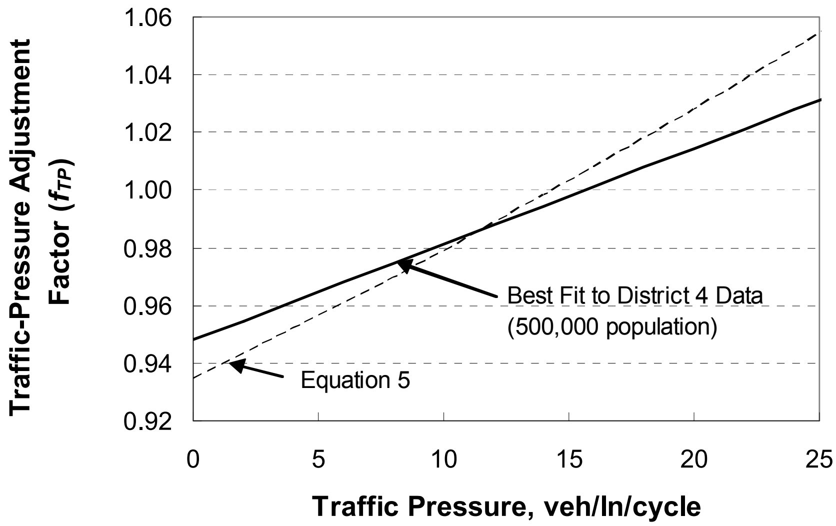


Figure 7. Predicted Effect of Traffic Pressure.

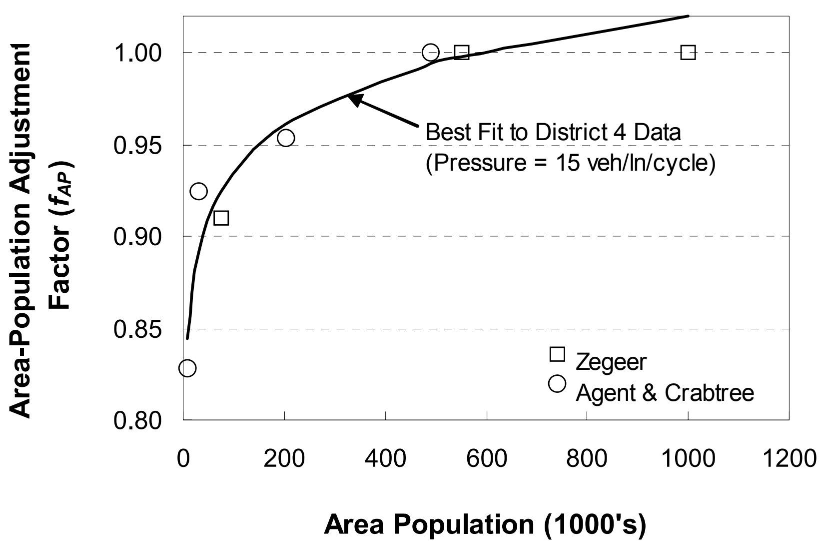


Figure 8. Predicted Effect of Area Population.

The findings from this reexamination serve to illustrate the challenges of computing the saturation flow rate and evaluating factors that influence saturation headways. Numerous factors can affect queue discharge characteristics, many of which are highly correlated with other factors. Statistical methods can be used to some degree to isolate the effect of one or two factors; however, to avoid being misled by multiple factor correlations, the database should include observations for the fullest range of factors (and factor levels). Thus, databases that tend to have a bias toward one-through-lane sites in low-population areas and three-through-lane sites in high-population areas should be avoided as they will mask the effect of number of lanes and population. Resources rarely allow for sufficient observations to be collected to control for a large number of extraneous influences (e.g., rain, driveway activity, spillback); hence, a good study design is needed to avoid introducing additional, unnecessary variability in the data. This variability is likely to cloud the search for cause and effect in the factors of interest and lower the level of confidence in the findings.

CHAPTER 3. STUDY SITE SELECTION AND DATA COLLECTION

OVERVIEW

This chapter describes the criteria used to select study sites and the plan established for collecting the data needed to quantify the effect of various factors on saturation flow rate. Intersections in the following four FDOT districts were considered for inclusion in the database assembled for this project:

- District 1 – Fort Myers,
- District 5 – Orlando,
- District 6 – Miami, and
- District 7 – Tampa.

These districts were selected because they offered a wide range of population densities.

For this research, a "study site" is defined as one intersection approach. At each site, data were collected for one or two lane groups. A "lane group" is defined as a set of adjacent approach traffic lanes serving a common travel direction (or turn movement) (*1*). Thus, one or more exclusive right-turn lanes form a "right-turn lane group," and one or more through lanes form a "through lane group." Turn movements that share a lane with the through movement are included in the through lane group.

The first part of this chapter describes the site selection criteria and the process used to select the study sites. The second part of this chapter describes the data collection plan. This plan describes the data to be collected, data collection methods, study duration, and sample size. The last part of this chapter describes the data reduction procedures.

SITE SELECTION

This part of the chapter consists of two sections. The first section describes the various criteria used to select the study sites. The second section describes the site selection process and includes a list of the selected study sites.

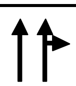
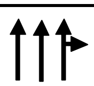
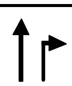
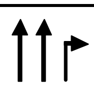
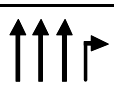
Site Selection Criteria

The selection of suitable study sites was based on consideration of a range of criteria. The criteria used include: approach configuration, area population, number of study sites, traffic volume, and intersection geometry. They are intended to: (1) ensure that the database contains intersection approaches that are typical to Florida conditions and (2) minimize extraneous factors that may influence saturation flow rate but which are beyond the scope of this research (e.g., pedestrian effects).

Approach Configuration

Table 3 identifies the various intersection approach configurations addressed in this research. As the table indicates, approaches with one, two, and three through lanes were sought during the site selection process. Each approach had either an exclusive right-turn lane or a curb lane that is shared by the through and right-turn movement. As described in a subsequent section, constraints in the field study procedure limited the field data collection to the simultaneous observation of up to three lanes during the study of any one approach. In recognition of this limitation, the study of approach configuration 6 focused on the three through lanes and did not include the exclusive right-turn lane. In contrast, both the through and the exclusive right-turn lanes were studied simultaneously during the study of approach configurations 4 and 5.

Table 3. Range of Approach Configurations Considered for Field Study.

No.	Approach Configuration	Area Population (1000's) [1]					Minimum AADT, [2] veh/d/ln
		< 5	5 to 50	50 to 200	200 to 500	> 500	
1		✔	✔	✔	✔	✔	4000
2		✔	✔	✔	✔	✔	3200
3		not likely	✔	✔	✔	✔	2900
4		✔	✔	✔	✔	✔	4000
5		not likely	✔	✔	✔	✔	3200
6 [3]		not likely	not likely	✔	✔	✔	2900

Notes:
1 - ✔ a combination of approach configuration and area population that is likely to be found based on typical traffic volumes found in these areas.
2 - The values listed represent the annual average daily traffic (AADT) needed to provide the minimum sample size for the through traffic lanes. Exclusive right-turn lanes require a minimum of 4000 veh/d/ln.
3 - Constraints in the data collection procedure limited the study of this configuration to only the three through lanes.

Area Population

Study sites were selected such that they collectively represent a range of area population. Table 3 indicates the desired range of area population. This range was needed to facilitate the assessment of area-type influence on saturation flow rate.

Traffic Volume

The following criteria were established to achieve statistically stable estimates of saturation flow rate at each site and to exclude the effect of extraneous factors:

- Each lane at an approach study site should experience an average queue of eight vehicles per lane or more for each cycle during peak periods.
- There should be negligible conflicting pedestrian and bicycle activity on approaches where right-turn saturation flows are to be recorded. For this study, "negligible" is defined as less than 10 conflicting pedestrians or bicyclists per hour.

The first criterion was evaluated by estimating an equivalent lane AADT that would need to be exceeded at a study site to have some assurance of obtaining the minimum number of observations. This AADT estimate was obtained from an analysis of the distribution of vehicle arrivals, signal timing, and approach configuration. The equivalent lane AADTs are listed in Table 3. Details of their derivation are provided in Appendix A.

Intersection Geometry

The following criteria were established to ensure that the data collected were not influenced by extraneous factors:

- Approaches should not have "active" driveways within 300 ft of the stop line.
- Approaches should not have local bus stops on the near or far side of the subject approach.
- For approaches where left-turn movements occur, a left-turn bay must be present.
- Approaches should have turn bay lengths that exceed the queue storage position of the eighth queued vehicle. Thus, a minimum turn bay length of 200 ft is desired. This requirement applies to both left-turn and right-turn bays.
- With one exception, approaches must have lane widths in the range of 11 to 13 ft. Any curb lanes that are studied can be up to 14 ft wide to allow for curb and gutter.
- Approaches should not have sharp curves or other unusual horizontal/vertical geometry.
- Approaches should have a grade in the range of −0.5 to +0.5 percent.
- Approaches should not experience queue spillback during the study period.

With regard to the first criterion, if an approach with an active driveway otherwise satisfied the site selection criteria, then it was included; however, queues that experienced impedance from driveway activity were removed from the database. For this study, an "active" driveway was defined as one having an entering (or exiting) volume of 10 veh/hr or more.

With regard to the second criterion, if an approach with a local bus stop was included, queues that experienced impedance from bus stop activity were excluded from the database.

Number of Study Sites

It was determined that a minimum of 30 approach study sites would be needed in the database to collectively represent the desired combinations of approach configuration, area population, and district representation.

Site Selection Process

Candidate intersections were identified by FDOT district staff, county staff, and the research team. The roadway and traffic characteristics for each candidate intersection were collected and tabulated. The area population for each intersection was based on the population of the city (or metropolitan area) within which the intersection was located. For intersections that are outside the city limits, the population was estimated using population estimates of the county and nearby cities.

Each candidate intersection was visited by a member of the research team to determine which of its approaches would be a suitable study site. Once the field visits were completed, 10 approach study sites were selected in each district using the criteria described in the previous section. The most common reasons for which sites were eliminated include: low volume, sharp skew, or short turn bays. The candidate intersections were reviewed for their consistency with the approach configuration and population criteria. The FDOT district within which they were located was also identified.

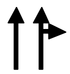
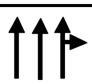
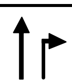
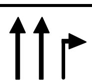
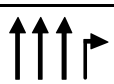
Two goals were established for the site selection process. One goal of the process was to distribute the sites as evenly as possible between the population and approach configuration combinations shown in Table 3. A second goal was to distribute the sites evenly among the FDOT districts. This objective was intended to ensure that "district" effects could be isolated from the effect of other factors (e.g., number of lanes, area type, etc.). After consideration of the site selection criteria and the process goals, a final list of study sites was established. These sites are identified in Table 4. In general, two approaches were selected for study at each intersection.

Table 5 categorizes the study sites by approach configuration and area population. For each site listed, the corresponding district number is indicated first followed by the site location. The site number (assigned in Table 4) is shown in parentheses following each site location. In general, the distribution of sites between the population and configuration combinations is reasonably balanced. However, it should be noted that all of the sites in the population group of "50 to 200" are located in a common district (i.e., District 1). Population in the vicinity of each study site was estimated using 2000 census data for Florida as well as the judgment of the research team and project review team members.

Table 4. Study Sites by Location.

District	No.	Site Location	Cross Street	Travel Direction	Approach Config.	Nearest City (Population)	Speed Limit, mph
5	1	U.S. 17/92	S.R. 436	NB	6	Casselberry (> 500,000)	45
	2			SB	6		45
	3	U.S. 441/Orange Blossom Trail	C.R. 482/ Sand Lake Road	NB	6	Orlando (> 500,000)	45
	4			SB	6		45
	5	S.R. 436/ Semoran Boulevard	S.R. 50/ Colonial Drive	NB	3	Orlando (> 500,000)	45
	6			SB	5		45
	7	S.R. 40/ Silver Boulevard	SW 27th Avenue	EB	2	Ocala (200,000-500,000)	45
	8			WB	5		45
	9	U.S. 92/ W. Intl. Speedway Boulevard	S.R. 483/Cycle Morris Boulevard	EB	6	Daytona (200,000-500,000)	45
	10			WB	6		45
6	11	U.S. 1	Sombrero Beach Boulevard	NB	5	Marathon (5,000-50,000)	45
	12			SB	2		45
	13	U.S. 1	Key Deer Boulevard	NB	2	Big Pine Key (5,000-50,000)	45
	14			SB	4		45
	15	U.S. 441/ S.R. 7	NW 183rd Street	NB	3	Miami (> 500,000)	45
	16			SB	3		45
	17	S.R. 997/ Krome Avenue	S.R. 994/ Quail Roost Drive	NB	1	Homestead (< 5,000)	45
	18			SB	1		45
	19	NW 7th Street	S.R. 953/ LeJeune Road	EB	2	Miami (> 500,000)	40
	20			WB	2		40
7	21	U.S. 19/ S.R. 55	C.R. 490/ Grover Cleveland	NB	2	Crystal River (5,000-50,000)	45
	22			SB	2		45
	23	Ulmerton Road	Belcher Road	EB	5	Clearwater (> 500,000)	40
	24			WB	5		40
	25	Alt. U.S. 19/ 5th Avenue North	U.S. 19/ 34th Street	EB	2	St. Petersburg (200,000-500,000)	40
	26			WB	2		40
	27	S.R. 52	U.S. 41/ S.R. 45	EB	4	Land O' Lakes (< 5,000)	55
	28			WB	4		55
	29	U.S. 301	12th Avenue	NB	1	Zephyrhills (5,000-50,000)	40
	30			SB	1		40
1	31	U.S. 92/ Memorial Boulevard	Massachusetts Ave.	EB	5	Lakeland (50,000-200,000)	45
	32		U.S. 98/N. Florida	EB	3		45
	33	S.R. 659/Combee Rd	U.S. 92/Memorial	NB	4		40
	34	U.S. 98/N. Florida	Parkview/10th Street	SB	2		45
	35	Lake Miriam Drive	Cleveland Hts. Blvd.	EB	1		35

Table 5. Study Sites Categorized by Configuration and Population.

No.	Approach Config.	Area Population (1000's) [1, 2]					Tot. App.[3]
		< 5	5 to 50	50 to 200	200 to 500	> 500	
1		D6 - S.R. 997 @ S.R. 994 (17,18)	D7 - U.S. 301 @ 12th Avenue (29,30)	D1 - Lake Miriam Drive @ Cleveland Hts. Boulevard (35)			5
2			D7 - U.S. 19 @ Grover Cleveland (21,22) D6 - U.S. 1 @ Key Deer Boulevard (13) D6 - U.S. 1 @ Sombrero Beach Boulevard (12)	D1 - U.S.98 @ Parkview (34)	D7 - Alt. U.S. 19 @ 34th St. (25,26) D5 - S.R. 40 @ SW 27th Avenue (7)	D6 - NW 7th St. @ S.R. 953 (19,20)	10
3				D1 - U.S. 92 @ U.S. 98 (32)		D5 - S.R. 436 @ S.R. 50 (5) D6 - U.S. 441 @ NW 183 Street (15,16)	4
4		D7 - S.R. 52 @ U.S. 41 (27,28)	D6 - U.S. 1 @ Key Deer Boulevard (14)	D1 - S.R. 659 @ U.S. 92 (33)			4
5			D6 - U.S. 1 @ Sombrero Beach Boulevard (11)	D1 - U.S. 92 @ Massachusetts Avenue (31)	D5 - S.R. 40 @ SW 27th Avenue (8)	D5 - S.R. 436 @ S.R. 50 (6) D7 - Ulmerton Rd. @ Belcher Rd. (23,24)	6
6					D5 - U.S. 92 @ S.R. 483 (9,10)	D5 - U.S. 17 @ S.R. 436 (1,2) D5 - U.S. 441 @ CR. 482 (3,4)	6
Total Approaches:		4	8	5	6	12	35

Notes:
1 - D1: FDOT District 1; D5: FDOT District 5; D6: FDOT District 6; D7: FDOT District 7.
2 - Number in parenthesis (x) is the site number assigned to the referenced intersection (see Table 4).
3 - Total number of approach study sites for each approach lane configuration.

DATA COLLECTION

This part of the chapter describes the data collection activities including preparation of the condition diagrams, equipment setup, and data collection procedures.

Condition Diagram

A condition diagram was prepared for each intersection at which one or more study sites are located. The diagram showed the following features for each approach study site:

- right-turn radius;
- lane width;
- driveway location, if present;
- bus stop location, if present;
- left-turn bay length and length of bay taper;
- right-turn bay length and length of bay taper;
- posted speed limit;
- adjacent land use characterization (i.e., business, shopping, residential, recreational); and
- any other factors that may influence saturation flow.

In addition to the above features, the lane assignments for all approach and departure legs are shown on the diagram. The data needed for this diagram were collected before or after the field study of saturation flow rate (but not during the study). An example condition diagram is shown in Figure 9.

Traffic Data Collection Equipment

The data collection equipment used in the study of an intersection approach consisted of one camcorder mounted on a tripod and miscellaneous batteries, blank tapes, and clipboards. One technician was deployed with each camcorder. This technician positioned the camcorder on his or her study approach and monitored it during the study period. The camcorder was mounted on a tripod and located just behind the curb and facing the intersection, as shown in Figure 10.

The camcorder was positioned such that its field of view included: (1) at least one controlling signal head for the subject through and right-turn movements and (2) a view of each traffic lane serving the subject movements (up to three lanes). The left edge of the camera field of view allowed a view of at least one full queue storage position in the most distant lane being studied (more storage positions were available in lanes nearer to the camcorder). Limitations of the camera field of view precluded the study of more than three lanes during a common time interval.

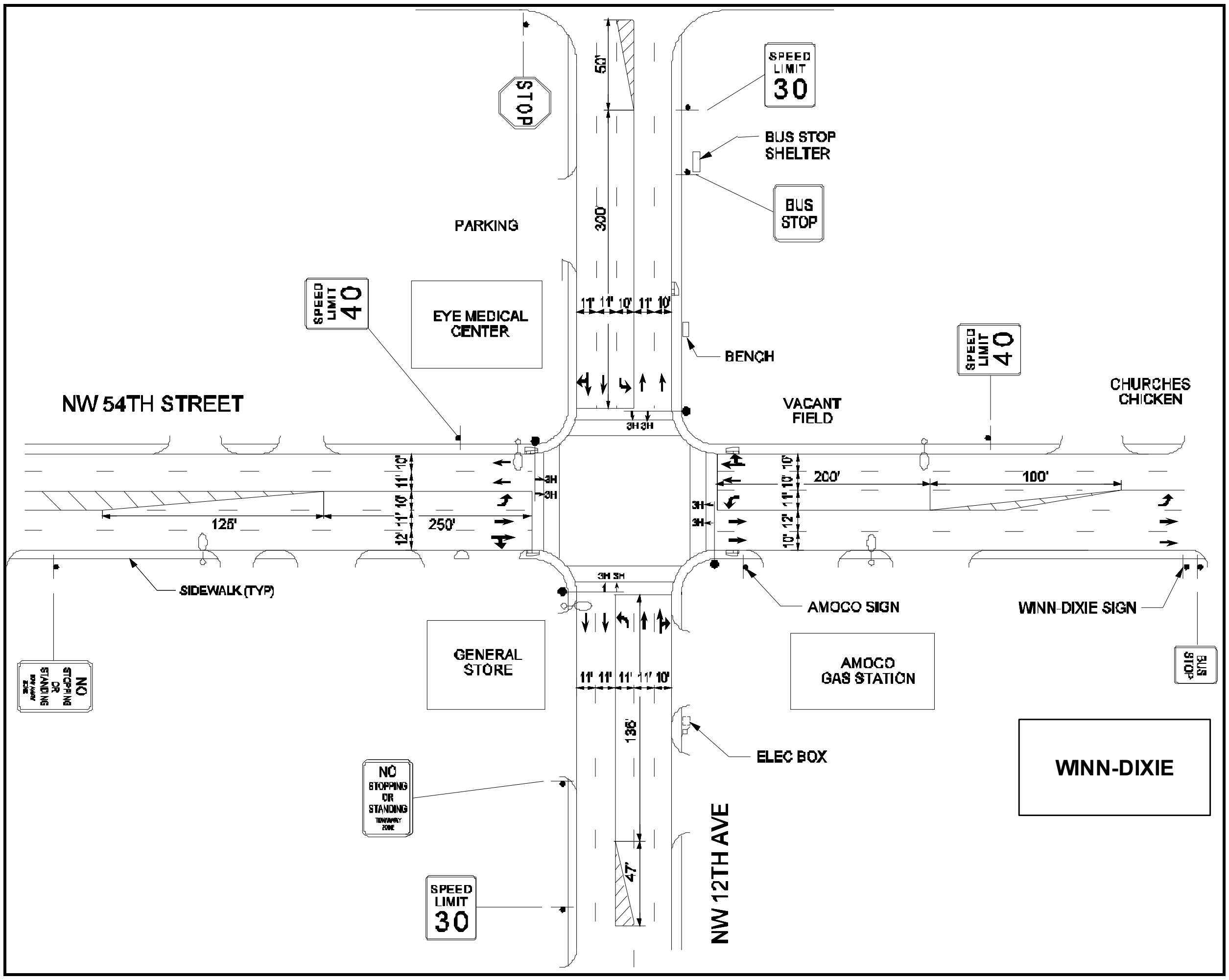


Figure 9. Example Condition Diagram.

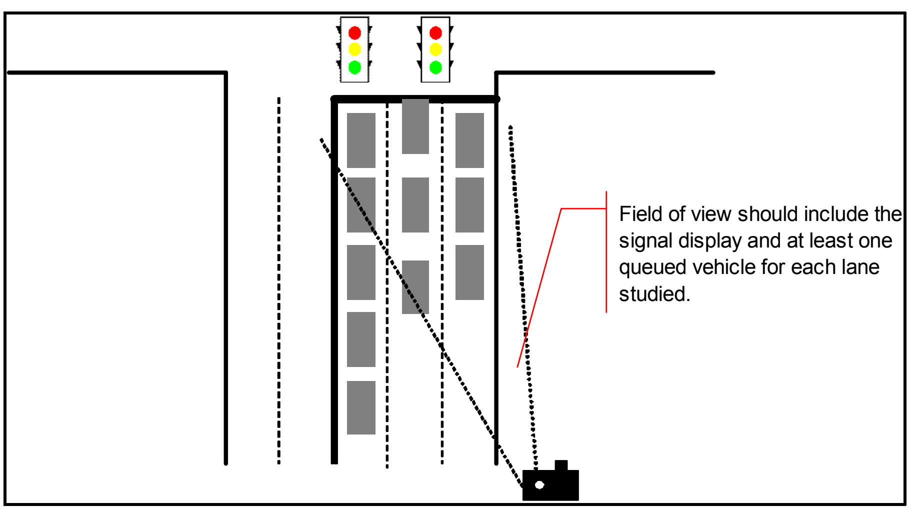


Figure 10. Typical Camcorder Location on the Intersection Approach.

Data Collection Procedure

Most traffic events of interest were recorded on videotape; however, some events were manually recorded during the study by the technician. All traffic events were recorded during the peak traffic hours (e.g., morning, noon, and/or afternoon peak). The equipment was set up at least 15 minutes before the start of each study period. Prior to the start of each study, the technicians synchronized the clocks in each camcorder with a common stopwatch.

During the course of the study, the technicians recorded specific traffic events on a Queue Reporting Worksheet. One worksheet was completed for each lane group studied. The format of this worksheet is illustrated in Table 6.

Table 6. Sample Queue Reporting Worksheet.

Cycle Begin Time[1]	Incident in Lane...? (Y/N)[2]			More than 8 Veh. in Queue in Lane...? (Y/N)[3]			Make/Model/Color of Last Queued Vehicle in Lane...[4]			Valid Obs. in Cycle[5]
	1	2	3	1	2	3	1	2	3	
8:15:12	N	N		Y	Y		Honda/Accord/White	Dodge/Caravan/Tan		2
8:17:10	N	N		Y	Y		VW/Jetta/Black	Toyota/Celica/Red		2
8:19:15	N	N		N	Y		Toyota/Camry/White	Ford/Taurus/White		1

Notes:

1 - Time to nearest second that the green time begins (this time is used to "align" the observations recorded in the field with the data extracted from the videotape when replayed in the lab).

2 - Used to denote when one or more of the following events occur in a lane being studied:
- Driveway access or egress movement.
- Local bus stops and loads/unloads passengers.
- Pedestrians or bicyclists cross the crosswalk that is parallel to the subject direction of travel and located on the same side of the street. This incident is only applicable when the right-turn movement is being studied.

3 - Used to denote whether there are more than eight vehicles in queue at the start of green. A vehicle is considered in queue if it is stopped in queue or is within the queue and moving at a speed of 3.0 mph or less.

4 - Make, model, and color of the last vehicle in queue at the start of green time.

5 - The number of observations where eight or more vehicles are in queue per studied lane at the beginning of green time and no incidents occur. If two lanes on an approach are studied and both lanes have eight or more vehicles in queue at the start of green and there are no incidents, the record of events during this cycle would count as two "cycle observations." The total number of valid cycle observations needs to equal or exceed 67 for each lane group.

Study Time Periods

Data were collected during time periods that are reflective of typical peak traffic periods at each study site. These periods typically occurred during midweek days (i.e., Tuesday, Wednesday, and Thursday) in the morning (7 to 9:00 a.m.), afternoon (11:30 a.m. to 1:30 p.m.), and evening (4 to 6:00 p.m.) peak periods. Data were not collected during holidays, periods of inclement weather, incidents, or construction activity.

Study Duration

A sample size analysis that considered the variability in headway data indicated that a minimum of 268 vehicles in queue positions five through eight were needed to obtain the desired level of confidence in the estimated overall average saturation flow rate for a given lane group. This number equates to 67 "cycle observations" (= 268/4). One cycle observation is defined to consist of the measurement of discharge time for the fourth and eighth queued vehicles in one traffic lane for one signal cycle. Thus, 34 cycle observations are needed for lane groups with two lanes (assuming that measurements are taken in both lanes), and 22 cycle observations are needed for lane groups with three lanes. Approaches with two lane groups (i.e., a through lane group and a right-turn lane group) need 67 cycle observations for each lane group.

Given that the sample size is defined in terms of "cycle observations," study duration is dependent upon signal cycle length and traffic volume. Assuming that queues of eight or more vehicles are present every cycle, an intersection with a cycle length of 100 seconds has 36 cycles per hour. If there are two lanes on the approach to this intersection, the desired 67 cycle observations would be obtained in about 0.9 hours (= 67/36/2). Similar calculations were used to compute the minimum study duration for various combinations of number of lanes and cycle length. The results of these calculations are shown in Table 7.

Table 7. Minimum Study Duration.

Cycle Length, s	Minimum Study Duration, h		
	1 Lane Studied	2 Lanes Studied	3 Lanes Studied
100	1.9	1.0	0.6
120	2.3	1.2	0.8
140	2.7	1.4	0.9
160	3.1	1.6	1.0
180	3.5	1.8	1.2

An analysis of sample size that includes the effect of traffic volume is described in Appendix A. It suggests that lane AADTs in excess of 5000 veh/d/ln are needed to achieve the minimum study durations listed in Table 7. If lane AADTs are only 3800 veh/d/ln, then the trends in Figure A-1 indicate that the durations listed in Table 7 increase by a factor of about 2.0. In fact, if the lane AADTs are less than 3800 veh/d/ln, the minimum study duration increases in an exponential manner due to the relatively few queues of eight or more vehicles found when AADTs drop below 3800 veh/d/ln.

Collection of an adequate sample size for the right-turn lane groups within a four-hour period (i.e., two peak periods) was anticipated to be more challenging than for the through lane groups. Frequent right turns on red tended to minimize queues in the right-turn lanes. Additional hours of study were conducted for the sites with exclusive lanes in those instances where the additional hours were believed to yield a cost-effective increase in sample size.

DATA REDUCTION

Data Reduction Worksheet

Data were extracted manually from the videotapes and recorded on a Data Reduction Worksheet. This worksheet is shown in Figure 11. Data entry fields are indicated by the shaded cells in this worksheet. The format of the worksheet follows that shown in Exhibit H16-1 of the *HCM* (*1*). The worksheet was completed for one lane of the subject approach study site. Multiple sheets were completed, as needed, to record the minimum 67 cycle observations.

Data Reduction Procedure

The data reduction procedure followed that described in Chapter 16, Appendix H of the *HCM.* As indicated in Figure 11, the data extracted for each signal cycle included the following items:

- time of start of green,
- time of end of green,
- discharge time of the 4th queued vehicle,
- discharge time of the 8th queued vehicle,
- discharge time of the last queued vehicle (only used in special circumstances),
- number of heavy vehicles in queue positions 1 through 4,
- number of heavy vehicles in queue positions 1 through 8,
- number of heavy vehicles in queue (only used in special circumstances),
- for shared-lane groups, number of right-turn vehicles in queue positions 1 through 4,
- for shared-lane groups, number of right-turn vehicles in queue positions 1 through 8,
- for shared-lane groups, number of right-turn vehicles in queue (only used in special circumstances),
- number of queued vehicles at the start of green,
- number of vehicles served during the cycle,
- time that queue discharge is blocked by pedestrians,
- time that queue discharge is blocked by a stopped local bus, and
- time that queue discharge is blocked by a bike.

The location at which these data were recorded on the worksheet is shown in Figure 12. As suggested by the sample data in this figure, the row identified as the discharge time of the last queued vehicle was not used. However, in some situations where volumes were unexpectedly light, it was necessary to record the times for the fourth and last queued vehicles. In this situation, the last queued vehicle was in queue position 6 or 7.

SATURATION FLOW RATE DATA REDUCTION WORKSHEET

General Information		*Site Information*	
Analyst:		Intersection:	
Agency:		Area Type:	
Date:		Approach Studied:	NB ☐ SB ☐ EB ☐ WB ☐
Time Period:		Movements Studied:	Left ☐ Thru ☐ Right ☐

Lane Movement Input

N

N-S Street:

Draw lane lines.
Indicate (with arrows) the movements allowed in each lane.

E-W Street:

Input Field Measurement

	Cycle 1			Cycle 2			Cycle 3			Cycle 4		
	Elapsed Time (hh:mm:ss:ff)	HV by 8th veh	T by 8th veh	Elapsed Time (hh:mm:ss:ff)	HV by 8th veh	T by 8th veh	Elapsed Time (hh:mm:ss:ff)	HV by 8th veh	T by 8th veh	Elapsed Time (hh:mm:ss:ff)	HV by 8th veh	T by 8th veh
Start of Green												
4th Vehicle												
8th Vehicle												
Last Vehicle												
End of Green												
No. Queued Veh												
No. Veh. Served												
Ped. Block Time												
Bus Block Time												
Bike Block Time												

	Cycle 5			Cycle 6			Cycle 7			Cycle 8		
	Elapsed Time (hh:mm:ss:ff)	HV by 8th veh	T by 8th veh	Elapsed Time (hh:mm:ss:ff)	HV by 8th veh	T by 8th veh	Elapsed Time (hh:mm:ss:ff)	HV by 8th veh	T by 8th veh	Elapsed Time (hh:mm:ss:ff)	HV by 8th veh	T by 8th veh
Start of Green												
4th Vehicle												
8th Vehicle												
Last Vehicle												
End of Green												
No. Queued Veh												
No. Veh. Served												
Ped. Block Time												
Bus Block Time												
Bike Block Time												

Glossary and Notes

HV = Heavy vehicles (vehicles with more than 4 tires on the pavement).
T = Turning vehicles (L = left, R = right)
Pedestrians, bicycles, and buses that block vehicles should be noted with the time (in seconds) that they block the discharging queue.

Figure 11. Data Reduction Worksheet.

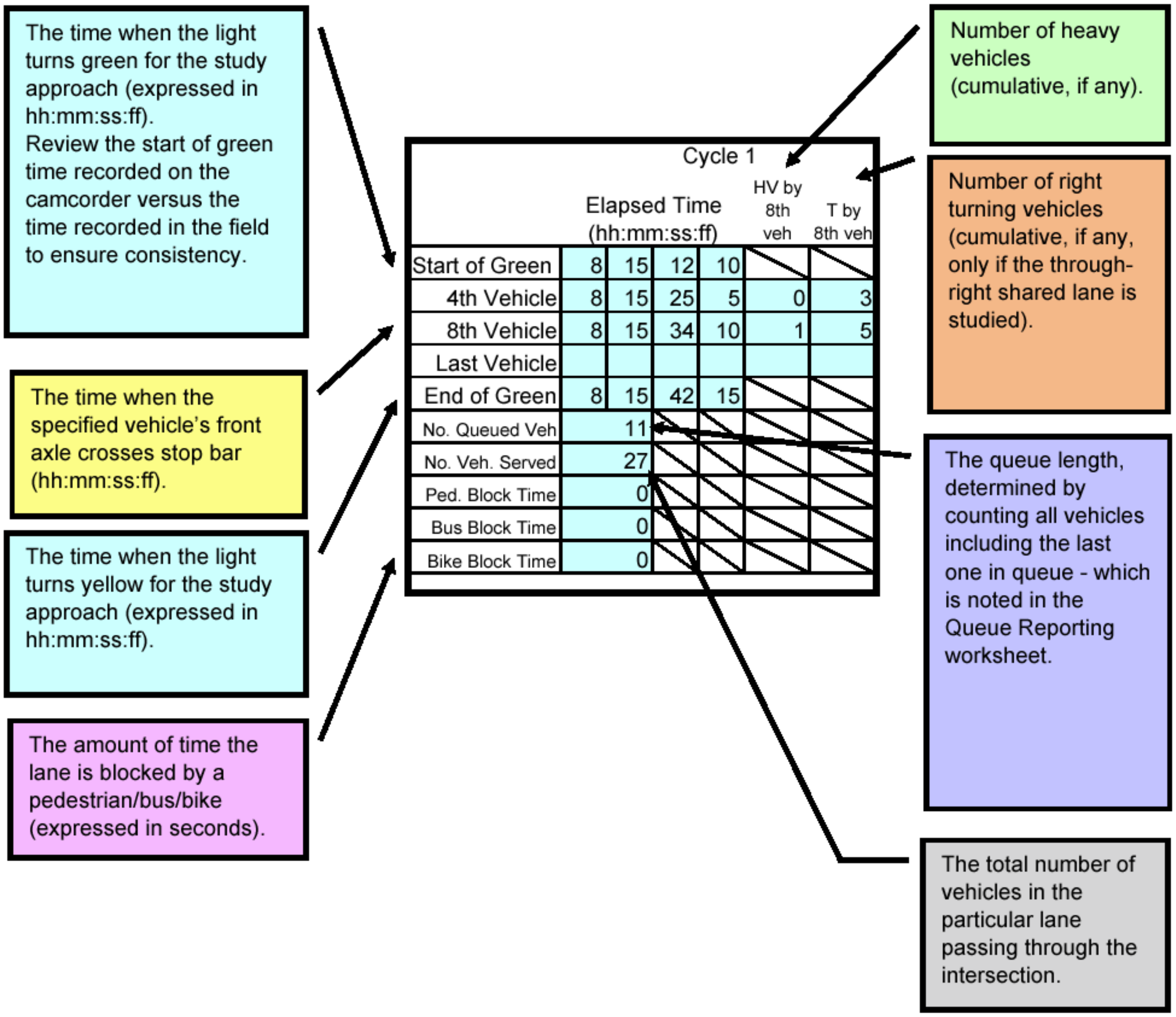


Figure 12. Sample Data Reduction Worksheet Data Input.

The front axle and the stop line are the reference points used for measuring vehicle discharge time. The use of the front axle represents a modification to the headway measurement process that was implemented with the 1994 update to the *HCM*. It represents a change from other studies conducted by FDOT (which used the back axle as a reference point) and will likely produce lower estimates of start-up lost time.

CHAPTER 4. DATA ANALYSIS

OVERVIEW

This chapter describes the development and evaluation of a series of saturation flow adjustment factors that explain the effect of right turns, area population, and number of lanes on saturation flow rate. The effects of these factors on saturation flow rate, as reported in the literature, were described previously in Chapter 2. This chapter also describes the procedure used to estimate the base saturation flow rate that is appropriate for signalized intersections in Florida.

This chapter consists of three parts. The first part summarizes the database assembled for this project. The second part describes the development and calibration of a saturation flow rate prediction model. The components of this model include the various adjustment factors found to have a statistically significant influence on the saturation flow rate measured at several field study sites. The last part documents the procedure used to estimate the base saturation flow rate.

DATABASE SUMMARY

This part of the chapter summarizes the data collected at 35 signalized intersection approaches in 14 Florida cities. Each approach represents one field study site. Database elements consist of general information about the site as well as specific information about its traffic characteristics and geometry. General site information includes: intersecting street names, travel direction, approach lane configuration, area population, and speed limit. Traffic characteristics include: approach volume, heavy-vehicle percentage, and daily traffic volume. Geometric information includes: lane width and curb radius.

This part consists of two sections. Initially, the intersection approach study sites are identified and described using the aforementioned database elements. Then, the saturation flow rates measured at each site are summarized to provide some insight as to the general trends in the data, relative to population and approach configuration.

Field Study Sites

Thirty-five intersection approaches were selected for field study. The site selection criteria were described previously in Chapter 3. The field study at each site consisted of the videotape recording of traffic events for a period of four or more hours, the manual observation of traffic queue length, and the survey of site geometry and traffic control devices. Data reduction involved the replay of the videotape and the extraction of saturation headways of the fourth and eighth vehicle in queue. Study site traffic characteristics and geometry are summarized in Table 8. Additional information about the study sites was previously provided in Tables 4 and 5.

Table 8. Study Site Traffic Characteristics and Geometry.

District	No.	Site	Travel Dir.	App. Config.	AADT,[1] veh/d	Flow Rate,[2] veh/h	Heavy Veh., %	Lane Width, ft	Turn Radius, ft	Cycle Length, s
5	1	U.S. 17/92	NB	6	58,000	1485	2.2	11	55	180 a.m. 200 p.m.
	2		SB	6		1707	2.3	11	45	
	3	U.S. 441/Orange Blossom Trail	NB	6	n.a.	1234	5.6	12	100	130 off 140 p.m.
	4		SB	6		1297	7.8	12	90	
	5	S.R. 436/ Semoran Boulevard	NB	3	53,500	1470	5.2	11	105	150 a.m. 160 p.m.
	6		SB	5		1603	4.0	11	140	
	7	S.R. 40/ Silver Boulevard	EB	2	n.a.	892	9.1	11	45	125 a.m. 135 p.m.
	8		WB	5		749	5.8	12	50	
	9	U.S. 92/ W. Intl. Speedway Boulevard	EB	6	n.a.	1380	2.7	12	120	150-170
	10		WB	6		1730	2.6	12	145	
6	11	U.S. 1	NB	5	32,000	1150	4.7	11.5	30	95-105
	12		SB	2		1176	3.7	12	20	
	13	U.S. 1	NB	2	20,000	736	1.6	12	32	120-140
	14		SB	4		702	2.9	12	60	
	15	U.S. 441/ S.R. 7	NB	3	n.a.	1735	6.3	11	35	125-150
	16		SB	3		1969	2.9	11	35	
	17	S.R. 997/ Krome Avenue	NB	1	15,000	630	12.0	12	50	70-80
	18		SB	1		692	14.1	12	50	
	19	NW 7th Street	EB	2	30,000	859	1.7	10	20	100-120
	20		WB	2		835	2.5	11	30	
7	21	U.S. 19/ S.R. 55	NB	2	31,100	1173	8.9	12.5	30	115-125
	22		SB	2		1117	7.8	12.5	75	
	23	Ulmerton Road	EB	5	50,000	1469	8.0	13	50	140-160
	24		WB	5		1341	9.5	13	50	
	25	Alt. U.S. 19/ 5th Avenue North	EB	2	21,500	726	2.2	11	40	120-140
	26		WB	2		915	1.4	11	40	
	27	S.R. 52	EB	4	15,100	396	12.5	13	75	105 a.m. 115 p.m.
	28		WB	4		420	13.2	12	115	
	29	U.S. 301	NB	1	15,700	694	6.4	12	40	80-90
	30		SB	1		584	4.3	11	30	
1	31	U.S. 92/ Memorial Boulevard	EB	5	37,500	1001	8.1	11	n.a.	130
	32		EB	3	35,000	948	11.7	11	n.a.	
	33	S.R. 659/Combee Rd.	NB	4	19,400	331	3.5	12	n.a.	105
	34	U.S. 98/N. Florida	SB	2	44,000	1180	2.7	12	n.a.	130
	35	Lake Miriam Drive	EB	1	16,300	483	0.5	11	n.a.	120-150

Notes:
1 - n.a.: data not available.
2 - Average approach flow rate during field study.

The data in Table 8 indicate a relatively narrow range of lane widths among the study sites. This narrow range was intentional and intended to limit the effect of influences (i.e., lane width) that are outside the scope of this project. In contrast, there is an intentionally wide range in the right-turn radius and cycle lengths among the sites. This range was intended to facilitate the analysis of these factors and their effect on saturation flow rate. Approach grades and pedestrian volumes were negligible at all study sites. None of the sites had shared through plus left-turn lanes.

Summary Statistics

The saturation flow rate measured at each of the 35 sites is summarized in Table 9. This rate was computed using the discharge times of the fourth and eighth queue positions. The flow rates shown are median values (as opposed to an average). When the data are not normally distributed, the median is influenced less by extremely small (or large) data points than is the average value. In this instance, the median flow rates range from 2 percent lower to 6 percent higher than the average flow rates. On average, the median flow rates are about 2 percent larger than the average rates.

The flow rates listed in Table 9 are categorized by lane use to facilitate some assessment of differences among sites, independent of the effects of right-turning vehicles. They represent the combined flow rate for a mixed stream of passenger cars and heavy vehicles. Collectively, a total of 10,252 vehicles were observed to be in saturation flow at the 35 field study sites. Of these, 7105 vehicles were observed in the through traffic lanes, and 3055 vehicles were observed in shared through plus right-turn lanes. Only 92 vehicles were observed in exclusive right-turn lanes. It was intended that many more observations would be obtained for exclusive right-turn lanes; however, sites with extensive queuing in the right-turn lane are fairly rare in Florida due primarily to right-turn-on-red operation. The desire to conduct the field studies during the winter and spring months (a time when seasonal visitors to Florida are at a peak) limited the window of time available to find more suitable sites with exclusive right-turn lanes.

The statistics in Table 9 indicate that a wide range in saturation flow rate exists among the sites. Values as low as 1376 veh/h/ln and as high as 2149 veh/h/ln were observed. The overall median saturation flow rate for the through traffic lanes studied is 1735 veh/h/ln. The overall median rate for the shared through plus right-turn lanes is lower at 1588 veh/h/ln. This trend likely reflects the fact that right-turning vehicles turn at a lower saturation flow rate. The ratio of these two flow rates is 0.92. In other words, the data suggest that the saturation flow adjustment factor for shared lanes is about 0.92. Figure 2 suggests that this value is consistent with that of a shared lane having about 50 percent right turns.

Table 9. Saturation Flow Rate Categorized by Site and Lane Use.

District	No.	Site	Cross Street	Travel Dir.	App. Config.	Through Lanes		Through+ Right Lanes		Right-Only Lanes	
						Obs.[1]	S.F.[2]	Obs.[1]	S.F.[2]	Obs.[1]	S.F.[2]
5	1	U.S. 17/92	S.R. 436	NB	6	308	1916			55	1854
	2			SB	6	384	1912			16	2149
	3	U.S. 441/Orange Blossom Trail	C.R. 482/ Sand Lake Road	NB	6	364	1735			0	--
	4			SB	6	432	1714			0	--
	5	S.R. 436/ Semoran Blvd.	S.R. 50/ Colonial Drive	NB	3	198	1819	242	1756		
	6			SB	5	289	1955			21	1435
	7	S.R. 40/ Silver Boulevard	SW 27th Avenue	EB	2	87	1770	135	1571		
	8			WB	5	305	1678			0	--
	9	U.S. 92/ W. Intl. Speedway Blvd.	S.R. 483/Cycle Morris Boulevard	EB	6	385	1811			0	--
	10			WB	6	395	1815			0	--
6	11	U.S. 1	Sombrero Beach Boulevard	NB	5	311	1711			0	--
	12			SB	2	102	1793	109	1565		
	13	U.S. 1	Key Deer Boulevard	NB	2	15	1612	9	1649		
	14			SB	4	183	1591			0	--
	15	U.S. 441/ S.R. 7	NW 183rd Street	NB	3	172	1704	200	1668		
	16			SB	3	253	1770	136	1684		
	17	S.R. 997/ Krome Avenue	S.R. 994/ Quail Roost Drive	NB	1			192	1510		
	18			SB	1			166	1518		
	19	NW 7th Street	S.R. 953/ LeJeune Road	EB	2	139	1735	150	1646		
	20			WB	2	160	1708	159	1621		
7	21	U.S. 19/ S.R. 55	C.R. 490/ Grover Cleveland	NB	2	149	1760	168	1516		
	22			SB	2	133	1735	152	1442		
	23	Ulmerton Road	Belcher Road	EB	5	284	1763			0	--
	24			WB	5	320	1808			0	--
	25	Alt. U.S. 19/ 5th Avenue N.	U.S. 19/ 34th Street	EB	2	150	1862	174	1629		
	26			WB	2	130	1694	153	1674		
	27	S.R. 52	U.S. 41/ S.R. 45	EB	4	216	1639			0	--
	28			WB	4	355	1600			0	--
	29	U.S. 301	12th Avenue	NB	1			284	1560		
	30			SB	1			314	1477		
1	31	U.S. 92/ Memorial Blvd.	Massachusetts Ave.	EB	5	374	1516			0	--
	32		U.S. 98/N. Florida	EB	3	164	1376	24	1521		
	33	S.R. 659	U.S. 92/Memorial	NB	4	240	1662			0	--
	34	U.S. 98	Parkview/10th St.	SB	2	108	1594	76	1577		
	35	Lake Miriam Dr.	Cleveland Hts.Blvd.	EB	1			212	1600		
					Total/Median:	7105	1735	3055	1588	92	1937

Notes:
1 - "Obs." number of queued vehicles for which a headway was measured.
2 - "S.F.": median saturation flow rate (in veh/h/ln) for all time periods studied.
"--" - Data not available.

The overall median saturation flow rate for the exclusive right-turn lanes is 1937 veh/h/ln. Intuitively, this rate should be lower than that for the shared lanes because of the lower flow rate associated with the right-turn maneuver. During the field studies, it was noted that the vehicles in exclusive right-turn lanes were rarely stopped during the red indication. Rather, they were taking advantage of the right-turn-on-red option and continuously moving during the red indication. Occasionally, these vehicles would pause at the "yield" point that is around the corner from the stop line and adjacent to the cross street. As a result, there was rarely a queue behind the stop line at the start of green indication, and when a queue was present, it was moving very efficiently over the stop line such that the observed headways were relatively short.

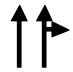
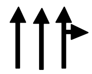
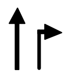
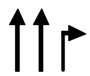
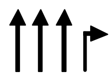
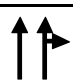
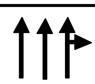
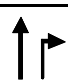
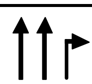
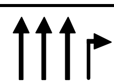
As noted previously, the lowest saturation flow rate for a through movement was found at U.S. 92 and U.S. 98 in District 1 (i.e., Site 32). This flow rate is 1376 veh/h/ln. The reason for this exceptionally low flow rate was explored by evaluating the traffic characteristics and geometry associated with the site. The saturation flow rate data exhibited wide variability (i.e., a standard deviation of 610 veh/h/ln); however, there was no apparent reason for this variability. Curiously, the next lowest saturation flow rate was recorded at a neighboring intersection (i.e., U.S. 92 and Massachusetts Avenue, Site 31). Lacking additional information about possible unique features of these two intersections, and given the large number of observations available for the through movement at other sites, it was decided that the data from these two sites should not be used in the calibration of the saturation flow adjustment factors.

The distribution of median saturation flow rates by approach configuration and population is shown in Table 10. The saturation flow rates in the last column of Table 10 that are associated with approach configurations 4, 5, and 6 for the "through lanes only" category indicate that there is a correlation between the number of lanes and saturation flow rate. This trend can also be seen in the rates for the shared through plus right-turn lane category. This finding is consistent with that found by McMahon et al. (*7*), as shown in Figure 4.

The weighted median flow rates for each of the three lane use categories provide some insight as to the correlation between population and saturation flow rate. The general trend is that flow rate increases with an increase in population. The increase is greatest from the "< 5,000" to the "5,000 to 50,000" categories. The increase associated with a change from "200,000 to 500,000" to "> 500,000" is more subtle. This trend is consistent with that found by other researchers (*3, 6*), as noted previously in Figure 3.

This examination of trends in Table 10 is only cursory; a more thorough assessment of factor effects is provided in the next part of this chapter.

Table 10. Saturation Flow Rate Categorized by Configuration and Population.

Lane Use	Config. No.	Approach Config.	Median Saturation Flow Rate, veh/h/ln					
			Area Population (1000's)					Overall Median
			< 5	5 to 50	50 to 200	200 to 500	> 500	
Through Lanes Only	2		--	1742	1594	1815	1708	**1742**
	3		--	--	1376	--	1763	**1708**
	4		1612	1591	1662	--	--	**1624**
	5		--	1711	1516	1678	1831	**1714**
	6		--	--	--	1815	1815	**1815**
	Overall Median:		**1612**	**1714**	**1554**	**1785**	**1793**	**1735**
Shared Through plus Right-Turn Lane	1		1510	1516	1600	--	--	**1521**
	2		--	1521	1577	1649	1639	**1588**
	3		--	--	1521	--	1711	**1704**
	Overall Median:		**1510**	**1516**	**1588**	**1649**	**1681**	**1588**
Right-Turn Only	4		--	--	--	--	--	**--**
	5		--	--	--	--	1435	**1435**
	6		--	--	--	--	1942	**1942**
	Overall Median:		**--**	**--**	**--**	**--**	**1937**	**1937**

Note:
"--" - Data not available.

MODEL DEVELOPMENT

This part of the chapter describes the development of a series of saturation flow adjustment factors that explain the effect of right turns, area population, and number of lanes on saturation flow rate. Initially, the database described in the previous part is analyzed to better understand the relationship between the various database variables and saturation flow rate. Then, adjustment factor functions are derived for use in the model calibration activity. Next, regression analysis is used to calibrate the adjustment factors using the field data. Then, the adjustment factors are validated. Finally, a sensitivity analysis is conducted using the calibrated factors.

Analysis of Factor Effects

The relationship between selected factors and saturation flow rate is analyzed graphically in this section. In general, the analysis of these relationships considered a wide range of factors related to intersection geometry, traffic character, and environment. The specific factors considered include: right-turn percentage, number of through lanes, area population, traffic pressure, heavy-vehicle percentage, speed limit, lane width, and curb radius. Those factors that were found to be visibly correlated saturation flow rate are discussed in this section.

The effect of right-turn percentage on saturation flow rate for the 18 sites with shared through plus right-turn lanes is shown in Figure 13. Each data point in the figure corresponds to the average saturation flow rate for the shared lane at one site. The one solid data point represents the average saturation flow rate of the through lanes adjacent to the shared lane at the sites (i.e., 1715 veh/h/ln). Only data from sites in areas with a population of 50,000 or more are shown. Data from sites with lower population tended to have a much lower overall saturation flow rate, which tended to obscure visualization of the effect of right-turn vehicle presence on saturation flow rate.

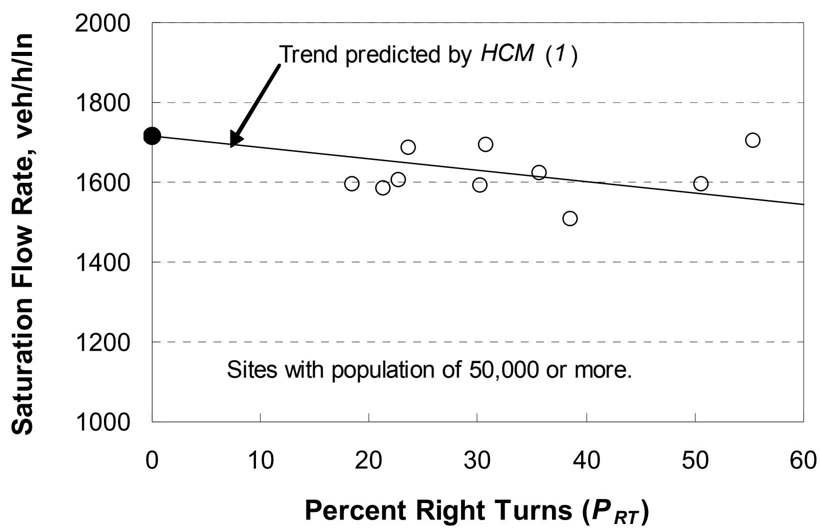


Figure 13. Effect of Right-Turn Percentage on Saturation Flow Rate.

Also shown in Figure 13 is the effect of right-turn vehicle presence on saturation flow rate, as obtained using the right-turn adjustment factor (f_{RT}) described in Exhibit 16-7 of the *HCM* (*1*) and using a base saturation flow rate of 1715 veh/h/ln. This effect is illustrated using a trend line. The downward slope in the trend line tends to be in general agreement with the downward trend in the saturation flow rate data points that represent the various study sites.

An examination of the saturation flow rate data revealed a tendency for a lower saturation flow rate in the curb lane of the shared-lane approaches. As noted previously, this trend may be partly explained by the tendency for heavy vehicles and/or right-turn vehicles to use this lane.

Further examination of this trend indicated that it also existed on intersection approaches with only through lanes (an exclusive right-turn lane was provided on these approaches to serve turning vehicles). This effect is shown in Figure 14.

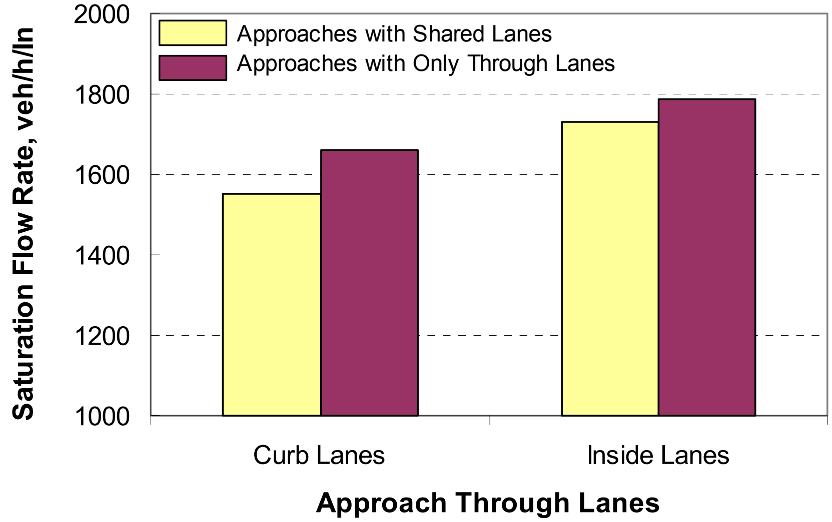


Figure 14. Effect of Lane Location on Saturation Flow Rate.

The trends shown in Figure 14 indicate that the curb lane has an overall average saturation flow rate that is about 10 percent lower than that of the adjacent through lanes. The lower flow rate may be due to the prevalence of heavy vehicles (as noted above) as well as various sources of "friction" associated with the edge of the roadway (e.g., driveways, pedestrians, utility poles, etc.). The trends in the figure also suggest that approaches with a shared lane tend to have an overall saturation flow rate that is 5 percent lower than that of approaches with only through lanes. This trend is likely due to the presence of right-turn vehicles in the shared lane. As noted in the discussion associated with Figure 13, right-turning vehicles tend to reduce the overall saturation flow rate of a lane.

The data in Table 10 suggest that saturation flow rate is correlated with area population such that higher flow rates are found in areas with higher population. This relationship was previously discussed in Chapter 2 in an examination of data reported by Zegeer (*3*) and by Agent and Crabtree (*6*). The relationship between average saturation flow rate and population found in the field data is shown in Figure 15.

Figure 15 shows the relationship between saturation flow rate and population for the shared-lane and the through-lane-only approaches found in the database. A best-fit curve is also shown to illustrate the trend in the data. The shape of the curve is consistent with the trends reported by Zegeer (*3*) and by Agent and Crabtree (*6*). The effect of population is most notable for areas with less than 200,000 persons. Intersections in areas above 200,000 persons are not significantly influenced by area population.

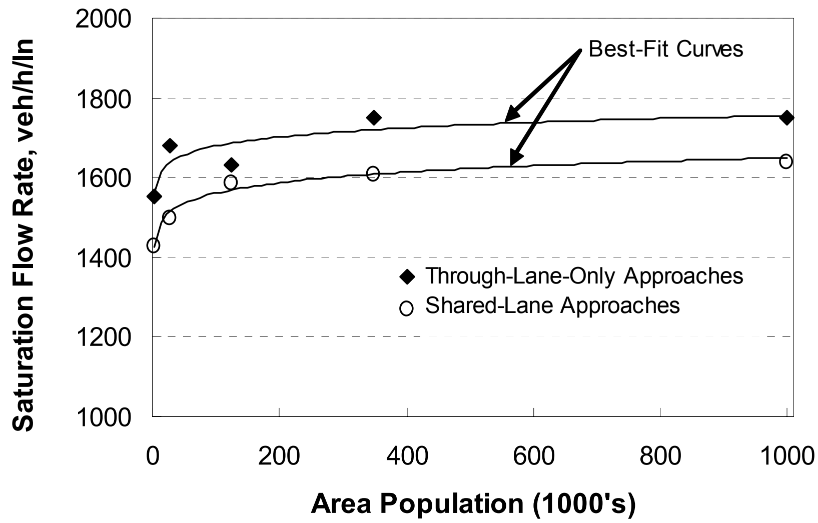


Figure 15. Relationship between Population and Saturation Flow Rate.

The relationship between number of approach lanes and average saturation flow rate is shown in Figure 16. This figure indicates that the saturation flow rate of an approach increases with the number of through lanes on the approach. The trend is apparent in both the shared-lane approaches and those with only through lanes. This relationship was also found in the data reported by McMahon et al. (*7*), as shown in Figure 4.

In a study of saturation flow rate at high-type intersections and interchanges, Bonneson (*8*) found that the number of vehicles in queue had an effect on saturation flow rate. Specifically, he found that the saturation headway measured for the fifth car in queue decreased with increasing queue length. This same trend was found for the sixth, seventh, eighth, etc. queue positions. He called this effect that of "traffic pressure." It is believed to result from the presence of aggressive drivers (e.g., commuters) who are anxious to minimize their travel time in otherwise high-volume conditions. They achieve their goal by accepting relatively short headways during queue discharge.

The relationship between traffic pressure (as measured in terms of vehicles served per signal cycle) is shown in Figure 17. Best-fit trend lines are also shown in the figure for the shared-lane and through-lane-only configurations. The trend lines are consistent among the two lane uses and show saturation flow rate increasing with an increase in traffic pressure. The effect is very similar to that reported by Bonneson and Messer (*9*), as shown in Figure 5.

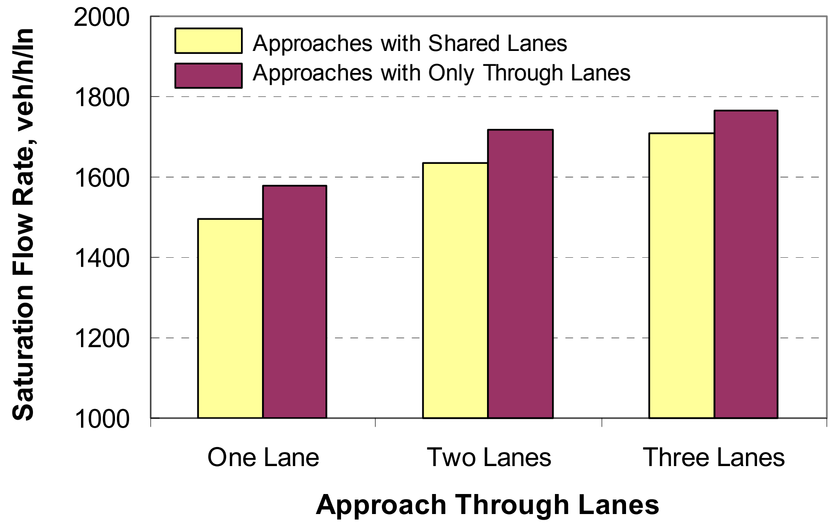


Figure 16. Effect of Number of Lanes on Saturation Flow Rate.

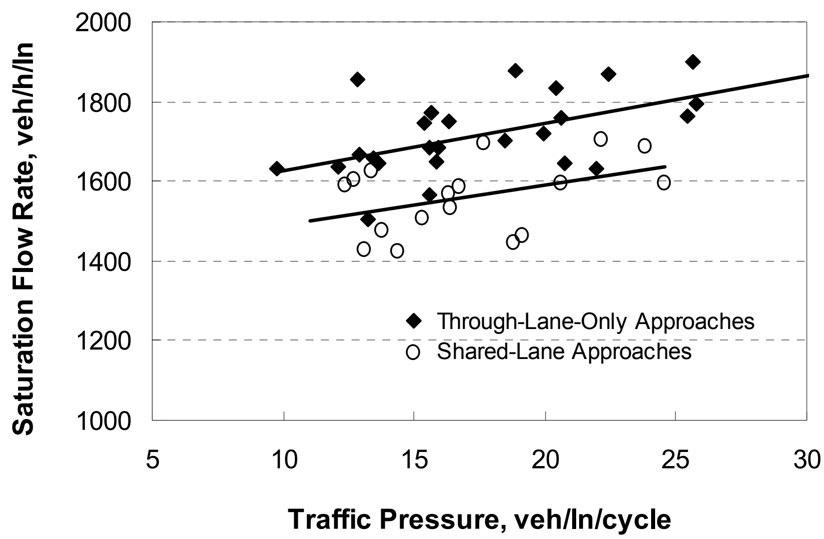


Figure 17. Effect of Traffic Pressure on Saturation Flow Rate.

The relationship between average saturation flow rate and the percentage of heavy vehicles is shown in Figure 18. Best-fit lines are also shown to illustrate the trend in the data. These trend lines indicate that saturation flow rate decreases with an increasing percentage of heavy vehicles. This trend is consistent with that predicted using the heavy-vehicle adjustment factor (f_{HV}) described in Exhibit 16-7 of the *HCM* (*1*).

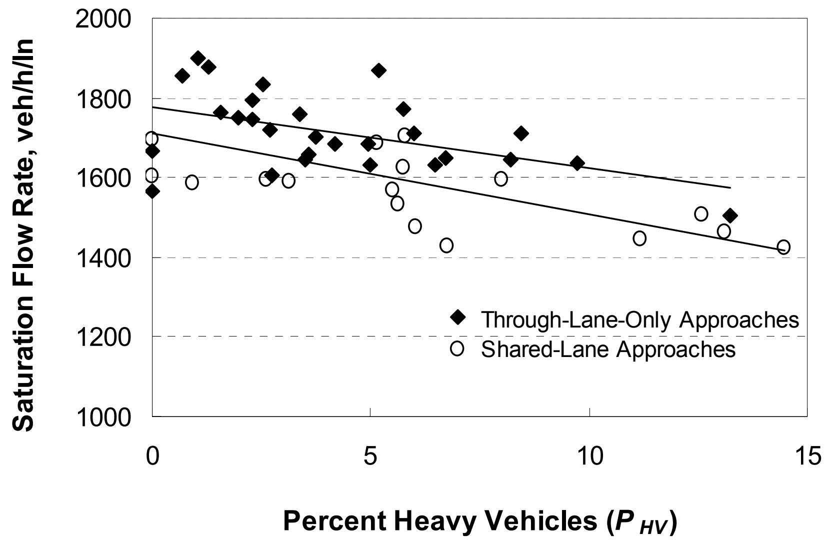


Figure 18. Effect of Heavy-Vehicle Percentage on Saturation Flow Rate.

The relationship between speed limit and saturation flow rate is shown in Figure 19. A best-fit line is also shown to illustrate the trend in the data. The trend line indicates that saturation flow rate increases with an increase in speed limit. A preliminary examination of the data indicated that the relationship between speed and saturation flow rate was obscured by the effect of more influential variables (e.g., population and traffic pressure). To eliminate these influences, the data shown in Figure 19 are based on saturation flow rates observed for signal cycles with only passenger car traffic streams in areas with population of 50,000 persons or more and with traffic pressures between 13 and 22 veh/ln/cycle.

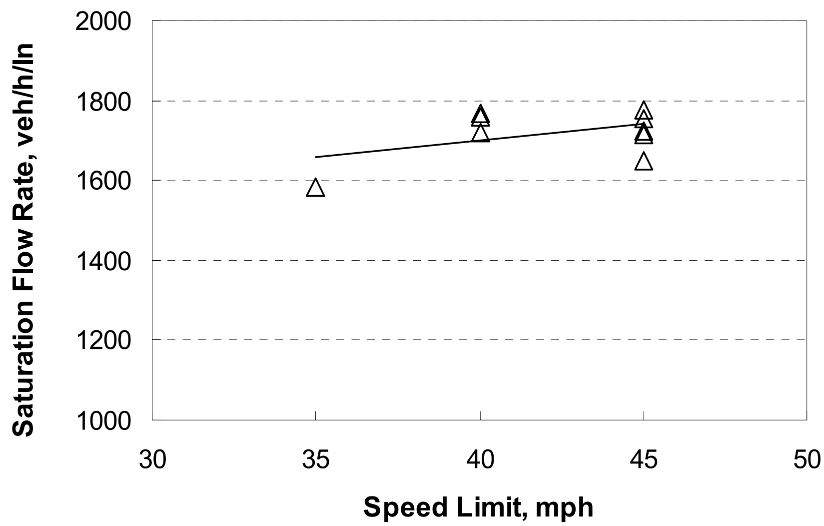


Figure 19. Relationship between Speed Limit and Saturation Flow Rate.

Theoretic models of the queue discharge process indicate that saturation flow rate increases with speed (*8*). However, the relationship between speed limit and saturation flow rate may also be a reflection of driveway density, roadside development, pedestrian activity, signal spacing, etc. in the vicinity of the intersection. According to Exhibit 10-4 of the *HCM* (*1*), lower speed limits are associated with frequent driveways, high density roadside development, frequent pedestrian activity, and short segment lengths. If these characteristics are present in the vicinity of the intersection, they are likely to have an adverse effect on saturation flow rate. In addition, there is additional evidence of a perceived relationship between speed limit and saturation flow rate in the Generalized Tables developed by FDOT for the 2002 *Quality/Level of Service Handbook* (*10*). Specifically, the "local adjustment factors" listed in Tables 4-1 to 4-9 vary with speed limit in a manner consistent with the trend in Figure 19 (for streets with otherwise common numbers of lanes, median type, and left-turn treatment).

Saturation Flow Rate Adjustment Factors

This section describes the development of several factors intended for use in estimating the saturation flow rate of signalized intersections in Florida. The factors developed herein include:

- adjustment factor for right turns, f_{RT};
- adjustment factor for number of lanes, f_N;
- adjustment factor for area population, f_{AP};
- adjustment factor for traffic pressure, f_{TP};
- adjustment factor for heavy vehicles, f_{HV}; and
- adjustment factor for speed limit, f_{SL}.

Equations for computing each of the aforementioned factors are described in this section. The calibration of each equation is the subject of the regression analysis described in the next section. In application, these factors would be used with Equation 1 to estimate the saturation flow rate for a lane group.

Adjustment factors for right turns and heavy vehicles currently exist in the *HCM*. However, these factors are reexamined in this section. In the next section, they are calibrated to conditions in Florida.

For each adjustment factor listed in Equation 1, the *HCM* provides a table of values or an equation (or equations) that can be used to determine the value of the adjustment factor. These equations are not repeated in this chapter but can be found in Chapter 16 of the *HCM*.

Adjustment Factor for Right Turns

The effect of right-turn vehicles on saturation flow rate is reported in several research and engineering reference documents (*2, 3, 4*). The general trend is one of decreasing saturation flow rate with increasing right-turn percentage. When the outside through lane is shared with right-turning vehicles, the resulting saturation flow rate can be estimated as a weighted average of the discharge headways of the two traffic movements (*2*). This averaging technique results in the following equation for adjusting saturation flow rate:

$$f_{RT} = \frac{1}{1 + P_{RT}(E_{RT} - 1)} \tag{15}$$

where:

P_{RT} = proportion of right-turn vehicles in the lane group and

E_{RT} = right-turn equivalency factor expressed in terms of an equivalent number of through passenger cars (= average right-turn headway divided by average through headway).

Adjustment Factor for Number of Lanes

As discussed with regard to Figure 14, the curb lane was found to have a lower saturation flow rate than any of the through lanes that exist to its left. An adjustment factor that accounts for this effect is:

$$f_N = \frac{1}{1 + \frac{1}{N}(E_{CL} - 1)} \tag{16}$$

where:

N = number of lanes in lane group and

E_{CL} = curb-lane equivalency factor expressed in terms of an equivalent number of through passenger cars (= average curb-lane headway divided by average headway in a non-curb through lane).

Adjustment Factor for Area Population

The curved shape of the best-fit trend line in Figure 15 suggests the following functional form for an area-population adjustment factor:

$$f_{AP} = \frac{1}{Pop^{\,b_5}} \tag{17}$$

where:

Pop = area population in millions of persons (= population × 10^{-6}); and

b_5 = empirical regression coefficient.

Adjustment Factor for Traffic Pressure

The equation for the traffic-pressure adjustment factor is based on the following equation form described by Bonneson and Messer (*9*):

$$f_{TP} = \frac{1}{1 + b_6(v_l - 20)} \tag{18}$$

where:

v_l = average flow rate per lane (i.e., traffic pressure) for the lane group, veh/ln/cycle; and
b_6 = empirical regression coefficient.

The equation offered by Bonneson and Messer (*9*) includes a traffic pressure of 15 veh/ln/cycle for the base saturation condition. Thus, using their equation, an approach with an average traffic pressure of 15 veh/ln/cycle would have an adjustment factor of 1.0. The range of volumes found in the data assembled for this project suggests a base value of 20 veh/ln/cycle. This value is reflected in the divisor of Equation 18.

Adjustment Factor for Heavy Vehicles

The factor for heavy vehicles is represented by the following equation. It is the same equation used in Chapter 16 of the *HCM* to adjust for heavy vehicles.

$$f_{HV} = \frac{1}{1 + P_{HV}(E_{HV} - 1)} \qquad \textbf{(19)}$$

where:

P_{HV} = proportion of heavy vehicles in the lane group and
E_{HV} = heavy-vehicle equivalency factor expressed in terms of an equivalent number of through passenger cars (= average heavy-vehicle headway divided by average through headway).

Adjustment Factor for Speed Limit

The factor for speed limit is represented by the following equation. Its form was suggested by the trends in the data in Figure 19. A speed limit of 50 mph was selected as the base saturation condition. Thus, approaches with a speed limit of 50 mph have an adjustment factor of 1.0.

$$f_{SL} = \frac{1}{1 + b_7(S - 50)} \qquad \textbf{(20)}$$

where:

S = approach speed limit, mph; and
b_7 = empirical regression coefficient.

Model Calibration

The model calibration activity was based on regression analysis using the following model form:

$$T_s = b_1 \times n \times P_1 \times P_2 \qquad \textbf{(21)}$$

with

$$P_1 = [1 + I_{CL}(b_2 - 1)] \times [1 + P_{RT}(b_3 - 1)] \times [1 + P_{HV}(b_4 - 1)] \qquad \textbf{(22)}$$

and

$$P_2 = Pop^{b_5} \times [1 + b_6(v_l - 20)] \times [1 + b_7(S - 50)] \qquad \textbf{(23)}$$

where:

T_s = saturated discharge time, s;
n = number of queued vehicles represented in saturated discharge time, veh;
I_{CL} = indicator variable for curb lane (= 1 if applied to a curb lane, 0 otherwise); and
b_i = empirical regression coefficients (i = 1, 2, 3, 4, 5, 6, 7, 8).

The form of this model is consistent with the method described in Chapter 16, Appendix H, of the *HCM* (*1*) for computing saturation flow rate. Most recently, this model form was used by Perez-Cartagena and Tarko (*11*) to estimate factors affecting saturation flow in Indiana.

Several of the regression coefficients have a theoretical interpretation. The b_1 coefficient represents the base saturation headway h_o. The reciprocal of this headway is the base saturation flow rate s_o. The b_2 coefficient represents the curb-lane equivalency factor E_{CL}. The b_3 coefficient represents the right-turn equivalency factor E_{RT}. Finally, the b_4 coefficient represents the heavy-vehicle equivalency factor E_{HV}.

The nonlinear regression procedure (NLIN) in the SAS software was used to estimate the coefficients for the model shown in Equations 21, 22, and 23 (*12*). This procedure was used because the model structure was multiplicative (as opposed to linear). NLIN uses an iterative search criterion to find the best-fit coefficients based on a maximum-likelihood approach using a user-specified error distribution. For this analysis, the error distribution was specified as the normal distribution.

Data for the shared lanes and the exclusive through lanes were combined and used to calibrate the regression model. It was rationalized that the exclusive right-turn lanes would require a separate calibration effort given the fundamental differences between their traffic character and that of the shared or exclusive through lanes. However, the calibration effort for the exclusive right-turn lane did not reveal useful relationships due to the paucity of data collected for this lane use. Thus, the findings reported in this section pertain entirely to the regression analysis of the shared lanes and the exclusive through lanes.

For this analysis, an "observation" was defined to be the saturated discharge time measured in one traffic lane during one signal cycle. The saturated discharge time T_s is computed as the difference between the discharge time of the last queued vehicle and that of the fourth queued vehicle. The appropriate traffic characteristics, geometry, and environment information were associated with each observation for statistical analysis of correlation. The database includes a total of 2648 observations. The model calibration was based on a randomly selected sample of 2387 observations; the remaining 261 observations were subsequently used to validate the calibrated model.

The statistics related to the calibrated prediction model are shown in Table 11. The calibrated coefficient values can be used with Equations 21, 22, and 23 to predict the saturated discharge time of a traffic queue during an average signal cycle. Alternatively, these values can be substituted in Equations 15 through 20 to obtain the calibrated adjustment factors. This substitution is the subject of discussion in the next section. The coefficient of determination R^2 is 0.74, indicating that about three-fourths of the variability in the data is explained by the model variables.

Table 11. Calibrated Model Statistical Description.

Model Statistics		Value		
	R^2:	0.74		
	Observations n_o:	2387 cycles		
	Standard Error:	±1.3 s		
Range of Model Variables				
Variable	**Variable Name**	**Units**	**Minimum**	**Maximum**
T_s	Saturated discharge time	s	1	16
P_{RT}	Portion right-turn vehicles	none	0.0	1.0
P_{HV}	Portion heavy vehicles	none	0.0	1.0
v_l	Traffic pressure	veh/ln/cycle	5	42
S	Speed limit	mph	35	55
Calibrated Coefficient Values				
Variable	**Definition**	**Value**	**Std. Dev.**	**t-statistic**
b_1	Base saturation headway (= 1905 pc/h/ln)	1.89	0.014	138.3
b_2	Effect of curb lane	1.03	0.006	164.4
b_3	Effect of right turns	1.07	0.015	72.8
b_4	Effect of heavy vehicles	1.74	0.023	77.0
b_5	Effect of area population	-0.018	0.0017	-10.7
b_6	Effect of traffic pressure	-0.0032	0.00040	-8.0
b_7	Effect of speed limit	-0.0066	0.00078	-8.4

The values associated with each of the calibrated coefficients are shown in the bottom half of Table 11. The values shown are consistent with the model development and the trends found in the literature review discussed in Chapter 2. The base saturation flow rate in the database is estimated as 1905 pc/h/ln (= 3600/1.89). The *t*-statistic for each calibrated coefficient is listed in the last column of Table 11. The statistics shown indicate that all of these values are statistically significant with a 99 percent level of confidence.

The fit of the model was also assessed through graphical comparison of the observed and predicted discharge times. This comparison is provided in Figure 20. The trend line in this figure does *not* represent the line of best fit. Rather, it is a "$y = x$" line. The data would fall on this line if the model predictions exactly equaled the observed data. The trends shown in the figure indicate that the model is able to predict discharge time without bias.

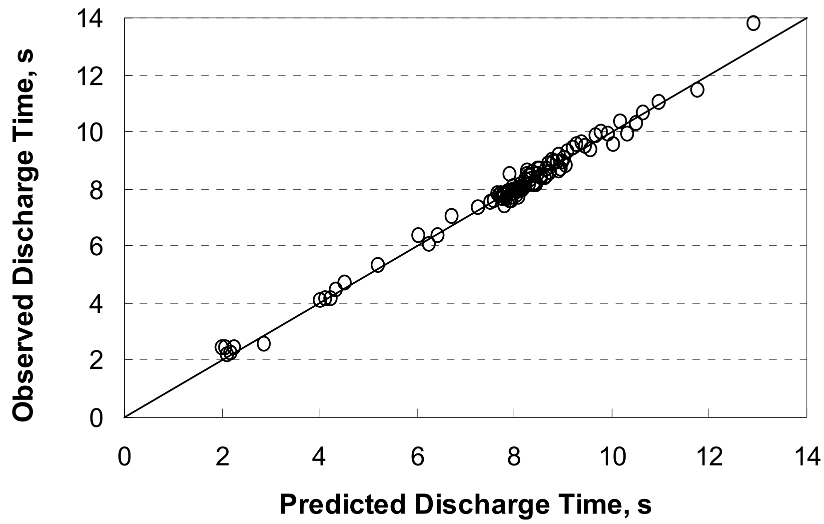


Figure 20. Comparison of Observed and Predicted Discharge Times.

Model Validation

Two tests were conducted to verify the validity of the calibrated adjustment factors. The first test was based on the use of a small portion of the data collected for this project. The second test was based on the use of the data reported by McMahon et al. (*7*), as reproduced in Table 2. The first test provides a more realistic indication of the factor's predictive ability because all the data needed for the various factor inputs were available and the data were disaggregate (i.e., not averaged over multiple hours). The latter test provides an indication of the transferability of the factors but yields a less realistic indication of predictive ability. One reason this test is less realistic is that some of the factor inputs (e.g., flow rate per cycle) had to be estimated from the reported data. Another reason for a reduced level of realism in the second test is that the reported data represented an average of conditions observed during nine hours of data collection. In this latter case, averaging over such extended periods tends to obscure the effect of a factor by reducing the range of values for the individual factor input variables.

Validation Based on Collected Data

For this validation test, 261 observations from the original database were used to evaluate the predictive ability of the calibrated adjustment factors. These 261 observations were not used for factor calibration. They were randomly selected from the original database and held in reserve for the validation test. Thirty-nine sites were represented in the validation database, of which 23 sites had observations of average saturation headway for six or more signal cycles. For each observation, all six of the calibrated adjustment factors were computed and multiplied together to obtain a "combined adjustment factor." The overall average saturation headway was also computed for each site (as an average of the cycle-based headways) and then converted into an estimate of the overall average saturation flow rate. These two variables are compared in Figure 21.

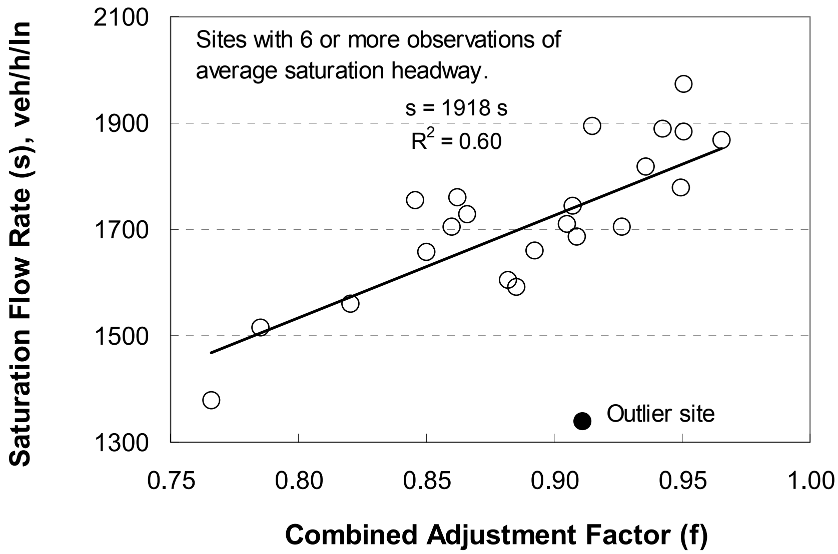


Figure 21. Adjustment Factor Validation Based on Collected Data.

As shown in Figure 21, the combined adjustment factors show good correlation with the saturation flow rate for 22 of 23 sites. A coefficient of determination R^2 of 0.60 was computed for the validation data. It suggests that the adjustment factors are able to explain 60 percent of the variability in saturation flow rate among the 22 sites. The saturation flow rate at one site did not follow the trend in the other 22 sites and was excluded from the analysis.

The best-fit line to the validation data is also shown in Figure 21, as is the corresponding equation (i.e., $s = 1918 f$). The constant in this equation is an estimate of the base saturation flow rate s_o for the validation sites. The value of 1918 pc/h/ln is slightly larger than the 1905 pc/h/ln estimated from the calibration data. The distribution of data points around the trend line is essentially uniform for the range of factor values which suggests that the factor is not biased.

Validation Based on McMahon Data

The data reported by McMahon et al. (*7*) were used for this validation test. These data were previously listed in Table 2. A combined adjustment factor was computed for each of the sites listed in Table 2 for which a speed limit was available. This factor was computed as the product of the area population, number of lanes, traffic pressure, and speed-limit adjustment factors described in a previous section. The computed factors are listed in Table 12. The average local adjustment factor is 0.902 with a range of 0.788 to 0.980.

The local adjustment factors listed in Table 12 were compared with the corrected base saturation flow rates listed in Table 2. The results of this comparison are shown in Figure 22. A best-fit trend line is also shown in the figure. The associated equation is shown, as well as the coefficient of determination R^2.

Table 12. McMahon Saturation Flow Rate Adjustment Factor Analysis.

County	County Pop. (1000's)	Site [1]	Approach Travel Direction	Traffic Pressure, veh/cy/ln	Speed Limit, mph	Approach Through Lanes	Adjustment Factors				
							f_{AP}	f_N	f_{TP}	f_{SL}	f_{all}
Palm Beach	1131	1	WB	18	45	2	1.002	0.985	0.994	0.968	0.951
Palm Beach	1131	1	NB	16	40	3	1.002	0.990	0.986	0.938	0.918
Palm Beach	1131	1	EB	12	45	2	1.002	0.985	0.974	0.968	0.931
Palm Beach	1131	1	SB	21	40	3	1.002	0.990	1.003	0.938	0.934
Palm Beach	1131	2	WB	18	40	2	1.002	0.985	0.993	0.938	0.920
Palm Beach	1131	2	EB	9	40	2	1.002	0.985	0.965	0.938	0.894
Palm Beach	1131	3	WB	26	45	3	1.002	0.990	1.020	0.968	0.980
Palm Beach	1131	3	NB	9	45	3	1.002	0.990	0.965	0.968	0.927
Palm Beach	1131	3	EB	17	45	3	1.002	0.990	0.991	0.968	0.952
Broward	1623	4	WB	5	45	3	1.009	0.990	0.954	0.968	0.923
Broward	1623	4	NB	6	45	3	1.009	0.990	0.956	0.968	0.925
Broward	1623	4	EB	4	45	3	1.009	0.990	0.950	0.968	0.919
Broward	1623	4	SB	6	45	3	1.009	0.990	0.956	0.968	0.924
Broward	1623	6	WB	6	45	3	1.009	0.990	0.957	0.968	0.925
Broward	1623	6	NB	9	45	3	1.009	0.990	0.966	0.968	0.934
Broward	1623	6	EB	5	45	3	1.009	0.990	0.953	0.968	0.921
Broward	1623	6	SB	9	45	3	1.009	0.990	0.966	0.968	0.934
Martin	127	7	WB	9	40	1	0.963	0.971	0.965	0.938	0.847
Martin	127	7	NB	27	45	2	0.963	0.985	1.024	0.968	0.941
Martin	127	7	EB	8	40	1	0.963	0.971	0.964	0.938	0.846
Martin	127	7	SB	28	45	2	0.963	0.985	1.027	0.968	0.943
Martin	127	8	WB	5	45	2	0.963	0.985	0.954	0.968	0.877
Martin	127	8	NB	28	45	3	0.963	0.990	1.026	0.968	0.948
Martin	127	8	SB	19	45	3	0.963	0.990	0.997	0.968	0.921
St. Lucie	193	9	WB	6	30	1	0.971	0.971	0.956	0.883	0.796
St. Lucie	193	9	NB	5	30	1	0.971	0.971	0.955	0.883	0.795
St. Lucie	193	9	EB	8	35	1	0.971	0.971	0.962	0.910	0.825
St. Lucie	193	9	SB	8	45	1	0.971	0.971	0.964	0.968	0.879
St. Lucie	193	10	NB	24	45	3	0.971	0.990	1.013	0.968	0.943
St. Lucie	193	10	EB	12	30	1	0.971	0.971	0.975	0.883	0.812
St. Lucie	193	10	SB	19	45	3	0.971	0.990	0.998	0.968	0.929
Indian River	113	11	WB	14	45	2	0.961	0.985	0.981	0.968	0.900
Indian River	113	12	WB	5	30	1	0.961	0.971	0.955	0.883	0.788
Indian River	113	12	SB	8	45	1	0.961	0.971	0.962	0.968	0.869

Note:
1 - Each site number represents one intersection.

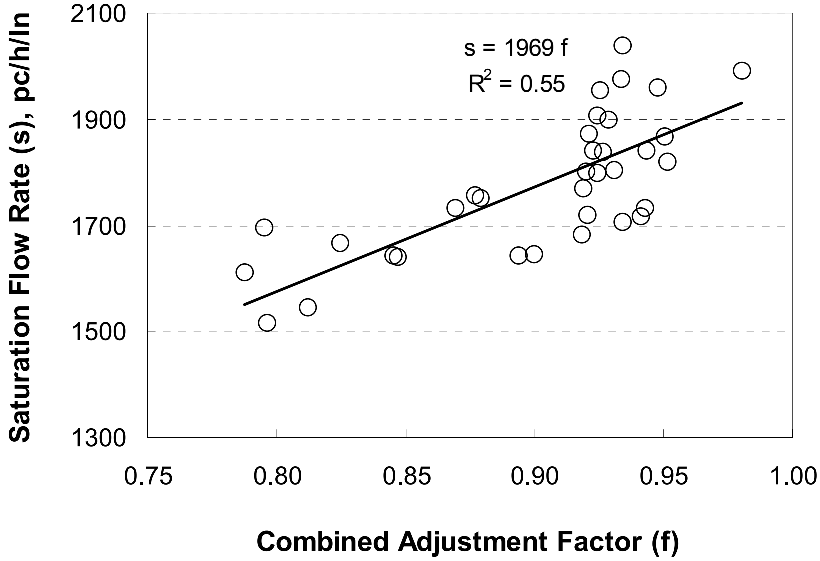


Figure 22. Adjustment Factor Validation Based on McMahon Data.

As indicated by the R^2 in Figure 22, the adjustment factors are able to explain 55 percent of the variability in the McMahon database. This is a relatively good fit to the data, given that they represent nine-hour averages.

The best-fit line to the validation data is also shown in Figure 22, as is the corresponding equation (i.e., $s = 1969 f$). The constant in this equation is an estimate of the base saturation flow rate s_o for the validation sites. The value of 1969 pc/h/ln is slightly larger than the 1905 pc/h/ln estimated from the calibration data.

Validation Summary

The findings from the two validation tests indicate that the calibrated factors are unbiased estimators of saturation flow rate. The R^2 of 0.74 in the calibration database is probably an indication of the systematic variability in the saturation flow data. The remaining 26 percent of variability (= 100 × [1.0 − 0.74]) is likely due to random variation. From this analysis, it is concluded that the calibrated adjustment factors can be extended to other locations in Florida without bias and with the expectation that they will explain at least 74 percent (= 55/74 × 100) of the variation in saturation flow rate that is "explainable" (i.e., due to systematic variability).

Sensitivity Analysis

This section describes the calibrated adjustment factors and illustrates their effect on saturation flow rate graphically. In some instances, the trend obtained from the calibrated factor is compared with similar factors reported in the literature.

Adjustment Factor for Right Turns

The calibrated adjustment factor for right turns is:

$$f_{RT} = \frac{1}{1 + P_{RT}(1.07 - 1)} \quad \textbf{(24)}$$

The relationship between right-turn percentage and the calibrated adjustment factor value is shown in Figure 23. The trend obtained from Equation 24 is designated as the "best-fit" line in the figure. Also shown is the relationship obtained from the right-turn adjustment factor described in Chapter 16 of the *HCM* (*1*). Both relationships indicate a reduced saturation flow rate with increasing turn percentage. The *HCM* relationship suggests that the effect of right turns is more significant than was measured at the Florida intersections.

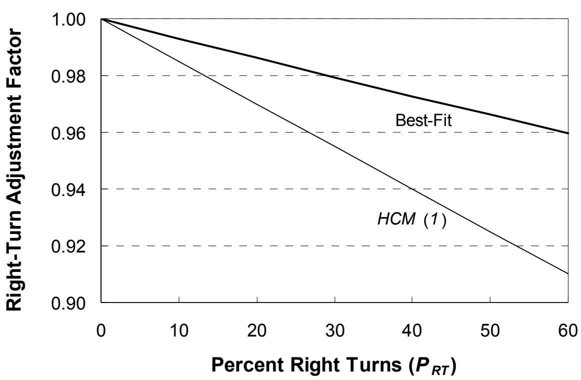


Figure 23. Right-Turn Adjustment Factor.

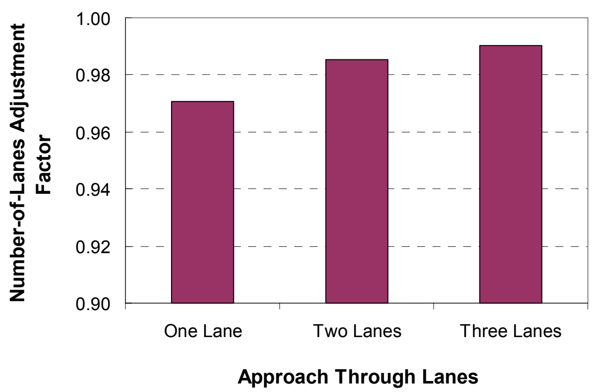


Figure 24. Number-of-Lanes Factor.

Adjustment Factor for Number of Lanes

The calibrated adjustment factor that accounts for the curb-lane effect is:

$$f_N = \frac{1}{1 + \frac{1}{N}(1.03 - 1)} \quad \textbf{(25)}$$

The relationship between number of lanes in the lane group N and the calibrated adjustment factor value is shown in Figure 24. The trend shown indicates that the overall saturation flow rate increases with an increasing number of lanes. However, this trend is due to the lower saturation flow rate in the curb lane, as is averaged over the remaining through lanes. The saturation flow rate in the inside through lanes is not affected by curb-lane friction.

The number-of-lanes adjustment factor and the right-turn adjustment factor should be used together to estimate the effect of a shared, through plus right-turn lane on lane group saturation flow rate. Consider a lane group with two through lanes, one of which is a through plus right-turn lane. The shared lane serves 120 veh/h turning right and 380 veh/h traveling through the intersection. The inside lane serves 500 veh/h through the intersection. These volumes equate to 12 percent right turns. The right-turn adjustment factor is 0.992; the number-of-lanes factor for a two-lane lane group is 0.985. The combined adjustment factor is 0.977.

Now, consider the same approach but with the inside through lane removed. The lane group now has one through-plus-right-turn lane. The same 120 veh/h turn right and the same 380 veh/h travel through the intersection. These volumes equate to 24 percent right turns. The right-turn adjustment factor is 0.983; the number-of-lanes factor for a one-lane lane group is 0.971. The combined adjustment factor is 0.954. A comparison of this factor with that for the two-lane lane group indicates that the multilane approach has a saturation flow rate that is 2.4 percent (= 0.977/0.954) larger than the single-lane approach, given the same traffic volume in the shared lane.

Adjustment Factor for Area Population

The calibrated adjustment factor for area population is:

$$f_{AP} = \frac{1}{Pop^{-0.018}} \tag{26}$$

The relationship between area population and the calibrated adjustment factor value is shown in Figure 25. Also shown in this figure is the curve fit through the adjustment factors reported by Zegeer (*3*) and by Agent and Crabtree (*6*). This curve was shown previously in Figure 3. In general, there is good agreement between the two curves. The best-fit curve obtained from the Florida data suggests that area population has less effect on saturation flow rate than is suggested by Zegeer or by Agent and Crabtree.

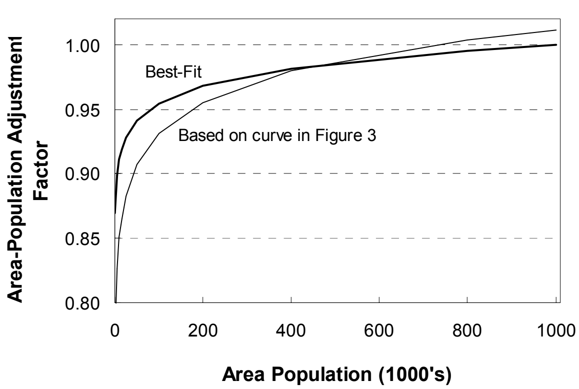


Figure 25. Area-Population Factor.

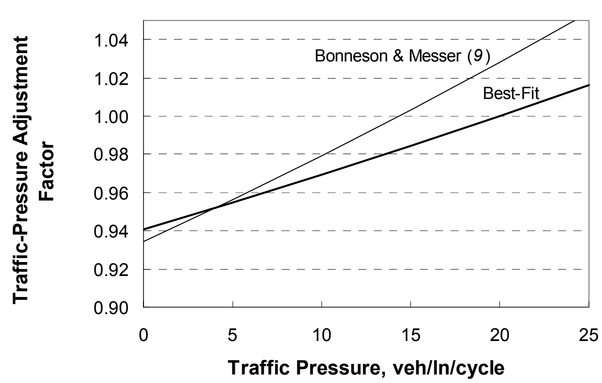


Figure 26. Traffic-Pressure Factor.

Because of its empirical nature, it is important to bound the area-population adjustment factor to the range of population reflected in the database. Specifically, a lower limit on population of 500 persons is appropriate for this factor. If it is applied to intersections in an area with a population less than 500 persons, the adjustment factor associated with a population of 500 persons should be used. Similarly, an upper limit on population of 2.0 million persons is appropriate. If the factor is applied to intersections in an area with a population in excess of 2.0 million persons, the adjustment factor associated with a population of 2.0 million persons should be used.

Adjustment Factor for Traffic Pressure

The calibrated traffic pressure adjustment factor is:

$$f_{TP} = \frac{1}{1 - 0.0032\,(v_l - 20)} \qquad \textbf{(27)}$$

The relationship between traffic pressure and the calibrated adjustment factor value is shown in Figure 26. As with the other figures, it is labeled as "best fit." Also shown in this figure is the effect of traffic pressure reported by Bonneson and Messer (*9*). The two trend lines show good agreement and confirm that saturation flow rate increases when the volume of vehicles served each cycle increases.

Because of its empirical nature, it is important to bound the traffic-pressure adjustment factor to the range of flow rates reflected in the database. Specifically, an upper limit on v_l of 35 veh/ln/cycle is appropriate for this factor. If the factor is applied to intersections with a flow rate in excess of 35 veh/ln/cycle, the adjustment factor associated with a flow rate of 35 veh/ln/cycle should be used.

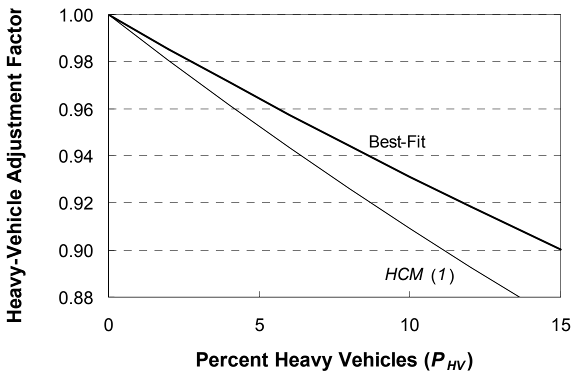
Adjustment Factor for Heavy Vehicles

The calibrated heavy-vehicle adjustment factor is:

$$f_{HV} = \frac{1}{1 + P_{HV}\,(1.74 - 1)} \qquad \textbf{(28)}$$

The heavy-vehicle equivalency factor of 1.74 in Equation 28 represents the mix of heavy vehicles at the intersections during the study periods. This factor is smaller in value than the 2.0 recommended in Chapter 16 of the *HCM* (*1*), and it is larger than the value of 1.5 used to develop the heavy-vehicle adjustment factor for Chapter 9 of the 1985 *HCM* (*13*).

The relationship between heavy-vehicle percentage and the calibrated adjustment factor value is shown in Figure 27. Also shown is the relationship obtained from the heavy-vehicle adjustment factor described in Chapter 16 of the *HCM* (*1*). Both relationships indicate a reduced saturation flow rate with an increasing percentage of heavy vehicles. The *HCM* relationship shown suggests that the effect of heavy vehicles is more significant than was measured at the Florida intersections.



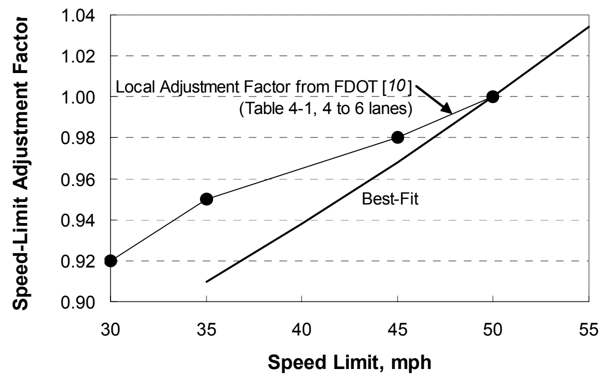


Figure 27. Heavy-Vehicle Adjustment Factor. **Figure 28. Speed-Limit Adjustment Factor.**

Adjustment Factor for Speed Limit

The adjustment factor for speed limit is represented by the following equation:

$$f_{SL} = \frac{1}{1 - 0.0066\,(S - 50)} \qquad \textbf{(29)}$$

The relationship between speed limit and the calibrated adjustment factor value is shown in Figure 28. Also shown is the relationship obtained from the local adjustment factors found in Table 4-1 of FDOT's 2002 *Quality/Level of Service Handbook* (*10*). The factors shown apply to four-lane and six-lane arterials; however, similar trends exist for the other cross sections.

As noted previously, theoretic models of the queue discharge process indicate that saturation flow rate increases with speed (*8*). However, the speed-limit adjustment factor may also be reflecting the effect of driveway density, roadside development, pedestrian activity, signal spacing, etc. in the vicinity of the intersection. Hence, this adjustment factor can be used to indirectly evaluate the effect of the street or highway environment on performance. It should not be used to explicitly assess the impact of speed (or a change in speed limit) on performance.

Because of its empirical nature, it is important to bound the speed-limit adjustment factor to the range of speeds reflected in the database. Specifically, a lower limit on speed of 30 mph is appropriate for this factor. If it is applied to intersection approaches with a speed limit less than 30 mph, the adjustment factor associated with a speed of 30 mph should be used. Similarly, an upper limit on speed of 55 mph is appropriate. If the factor is applied to intersection approaches with a speed limit in excess of 55 mph, the adjustment factor associated with a speed of 55 mph should be used.

BASE SATURATION FLOW RATE

This part of the chapter describes the method used to estimate base saturation flow rate. The b_1 coefficient in the calibrated regression model indicates a base saturation flow rate of 1905 pc/h/ln. The regression equations in Figures 21 and 22 suggest base saturation flow rates of 1918 and 1969 pc/h/ln, respectively. These differences are resolved in the following sections for the purpose of estimating a single recommended base saturation flow rate for all Florida intersections.

Influence of Queue Position

As shown by Bonneson (*14*), headways do *not* reach a constant saturation value after the fourth queue position. Rather, they tend to be shorter with each subsequent queue position until the eighth, or possibly even higher, queue position. The reduction in headway after the fourth queue position tends to be small, and for purposes of facilitating the collection of sufficient sample sizes in field studies, the fourth queue position is recommended for use by the *HCM* for measurements of saturation flow rate.

The implication of the influence of queue position is that researchers attempting to estimate base saturation flow rate may have a slight bias in their estimate if they include headways from the fourth, fifth, or even sixth queue position. This bias would be in a direction of yielding a smaller base saturation flow rate than is ultimately achieved by queue positions eight and above. It should be noted that the calibration of saturation flow rate adjustment factors is not as sensitive to this issue because the influence of most adjustment factors is the same, regardless of whether the vehicle is in the fourth queue position or the tenth queue position.

The queue discharge model developed by Bonneson (*14*) was used to estimate the effect of using headway data recorded for only the fourth through eighth queue positions, as opposed to the fourth through tenth queue position (as was used by McMahon et al.[*7*]). Specifically, Equation 20 in the paper by Bonneson (*14*) was used to estimate the average minimum discharge headway for queue positions four through eight and for positions four through ten. The findings from this analysis indicate that the minimum discharge headway using queue positions four through ten is about 0.02 s/veh shorter than that found when using queue positions four through eight. This difference translates into a base saturation flow rate ratio of 1.3 percent. Specifically, if queue positions four through ten are used to estimate base saturation flow rate, the resulting flow rate will be 1.3 percent larger than that found when using queue positions four through eight.

The statistics in Table 11 indicate that the base saturation headway headway is 1.89 s/veh ±0.014 s/veh. If the average headway is reduced by 1.3 percent, the resulting adjusted base saturation headway is estimated as 1.87 s/veh.

McMahon Data Analysis

The regression analysis conducted to develop Figure 22 revealed that the base saturation flow rate in the McMahon data was 1969 pc/h/ln ±16 pc/h/ln. These values convert into a base saturation headway of 1.83 s/veh ±0.015 s/veh.

Base Saturation Flow Rate Estimate

This section describes the method used to estimate the base saturation flow rate using the databases described in the two previous sections. Specifically, the base saturation flow estimate was obtained by using a weighted average of the saturation headway estimates from the two studies. The value used to weigh the two estimates is the standard deviation obtained from their respective regression analyses. The analysis is illustrated in Table 13 (values shown are rounded).

Table 13. Calculation of Base Saturation Flow Rate.

Database	Saturation Headway (*H*), s/veh	Standard Deviation (*S*), s/veh	Weight (*W*) [1]	*H* × *W*
Table 11	1.87	0.014	5102	9519
McMahon	1.83	0.015	4258	7785
		Total:	9360	17,304
Average:	1.85 [2]		**Base Saturation Flow Rate, pc/h/ln:**	**1950**

Notes:
1 - Weight, $W = 1/S^2$
2 - Average headway, $H = \sum (H \times W) / \sum W$.

As indicated in the last row of Table 13, the weighted average saturation headway is 1.85 s/veh. The corresponding base saturation flow rate is 1950 pc/h/ln. This value is recommended for use with the calibrated adjustment factors at intersections in Florida. It reflects the saturation flow rate that would be measured when using the measured discharge times of queue positions four and ten. Slightly different values will be obtained if different queue positions are used as a basis for the estimate of base saturation flow rate.

CHAPTER 5. APPLICATION PROCEDURE

OVERVIEW

The findings from a review of the literature, as documented in Chapter 2, indicate that some areawide influences and local conditions have been found to be correlated with saturation flow rate but are not included in the *HCM* (*1*). Areawide influences reflect differences between drivers in small towns and those in large cities. Local conditions reflect differences between drivers during off-peak and peak traffic hours. Local conditions also reflect the effect of constraints on the driving environment, as may be characterized by speed limit or the number of approach traffic lanes.

Chapter 16 of the *HCM* (*1*) describes a procedure for estimating the saturation flow rate of a given intersection approach. This procedure includes numerous saturation flow rate adjustment factors that are intended to adjust the base saturation flow rate such that the resulting saturation flow estimate accurately reflects conditions on the intersection approach. These factors were listed previously in connection with Equation 1.

Saturation flow adjustment factors reflecting the effect of population, traffic pressure, number of lanes, and speed limit were developed for this research. The methods used to develop these factors were described in Chapter 4. The objective of this chapter is to document a procedure for describing the correct use of these adjustment factors. This procedure is intended to be used with the *HCM* or FDOT's *Quality/Level of Service Handbook* (*10*) to estimate the saturation flow rate for specific intersections, based on consideration of areawide influences and local conditions. This chapter is concluded with a list of recommended research topics.

PROCEDURE

A procedure for computing the saturation flow rate of a signalized intersection lane group is described in Chapter 16 of the *HCM*. This procedure is part of a larger methodology for estimating the control delay incurred by, and level of service provided to, the intersection lane group and the intersection as a whole. The signalized intersection analysis methodology is shown in Figure 29; the saturation flow rate estimation procedure is a second step of this methodology (it is identified by dashed outline). As indicated by the flow of calculations in this figure, the saturation flow rate is subsequently used to estimate lane group capacity, which is then used to estimate delay, level of service (LOS), and the distance to the maximum back of queue. Hence, an accurate estimation of saturation flow rate is key to the accurate estimation of intersection performance.

This part of the chapter describes several factors that can be used to estimate the effect of area population and three local conditions on saturation flow rate. These factors include:

- adjustment factor for number of lanes,
- adjustment factor for area population,
- adjustment factor for traffic pressure, and
- adjustment factor for speed limit.

The next section identifies the input variables needed for these factors. The section after that describes the equations to be used to compute the factor values. The last section illustrates the use of the adjustment factors in an example application.

Required Input Data

The local adjustment factors and their required input data are discussed in this section, including a discussion of default input values where appropriate. The input data needed for each factor are summarized in Table 14.

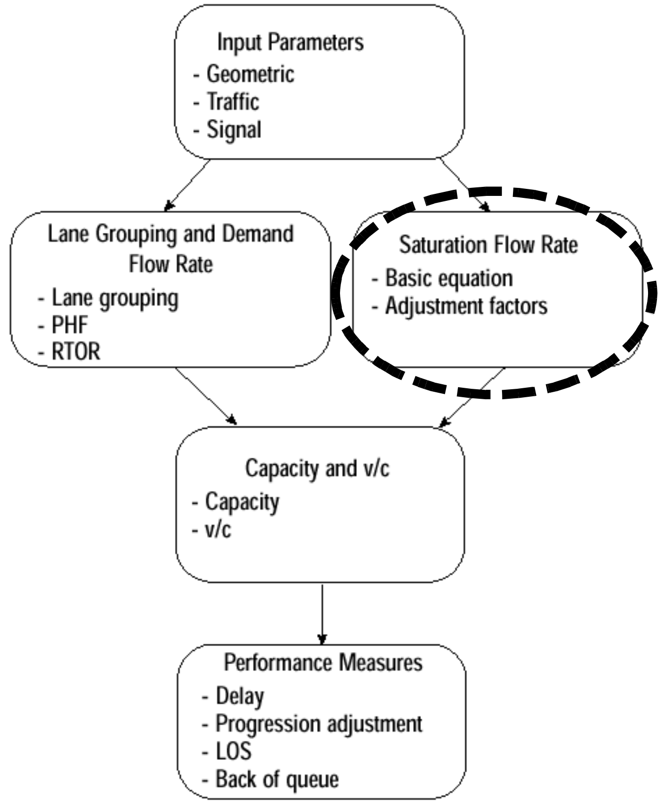


Figure 29. *HCM* Signalized Intersection Analysis Methodology.

Table 14. Input Data for Local Adjustment Factors.

Adjustment Factor	Input Data	Applicable Lane Groups [1]
Adjustment factor for number of lanes	Number of lanes in lane group	Through
Adjustment factor for area population	Area population	Left, through, right
Adjustment factor for traffic pressure	Lane group volume, cycle length	Through
Adjustment factor for speed limit	Speed limit	Left, through, right

Note:
1 - Lane groups are established using the guidelines provided in the *HCM* (*1*, p. 16-6). In general, all turn movements served by an exclusive lane are designated as separate lane groups. All remaining lanes are grouped together as the "through" lane group. An approach will have three lane groups if it has exclusive left and right-turn lanes (or bays).

Number of Lanes

A curb-lane adjustment factor is appropriate for the through lane group on the intersection approach. The number of lanes input is based on information obtained during field inspection and relates to the number of through lanes present. The factor applies to all approach cross sections, including those with an exclusive right-turn lane, paved shoulder, parking lane, or bicycle lane.

Area Population

Area population is the number of people in the surrounding area of the subject intersection/facility. FDOT's *Quality/Level of Service Handbook* section on "Area Type" should be consulted for more specific guidance on determining an intersection's area population.

Traffic Pressure

Saturation flow rate tends to increase slightly when there are more vehicles in queue. This effect is referred to as "traffic pressure." It results from the presence of aggressive drivers who are anxious to minimize their travel time in otherwise high-volume conditions. They achieve their goal by accepting relatively short headways during queue discharge.

For practical application, the degree of pressure exerted on traffic can be estimated as the average number of vehicles served per lane during each signal cycle. This value is computed as:

$$v_l = \frac{v\ C}{N \times 3600} \qquad \textbf{(30)}$$

where:

v_l = average flow rate per lane (i.e., traffic pressure) for the lane group, veh/ln/cycle;
v = lane group volume, veh/h;
C = cycle length, s; and
N = number of lanes serving the lane group.

Because of the empirical nature of the adjustment factor for traffic pressure, an upper limit of 35 veh/ln/cycle is recommended for v_l when it is used to compute the adjustment factor.

Direct measurement of vehicle counts per lane for a sample of signal cycles is the best method of estimating traffic pressure. In the absence of field counts, the default values listed in Table 15 may be used for a given cycle length and target level of service. They were computed using Equation 30 and a typical volume for each of the service levels indicated.

Table 15. Default Traffic Pressure Values.

Cycle Length, s	Traffic Pressure by Estimated Level of Service, veh/ln/cycle			
	B	C	D	E
90	3	12	19	21
100	4	14	22	23
120	5	17	26	28
140	5	19	30	33
160	6	22	34	35
180	7	25	35	35
200	8	28	35	35
220	8	30	35	35
240	9	33	35	35

Speed Limit

The four urban street design categories identified in the *HCM* characterize combinations of physical features and traffic activity that are typically found together on an urban street. These characteristics are summarized in Exhibit 10-4 of the *HCM*; they are repeated herein as Table 16. Of particular note is the range of speed limits typically associated with urban streets of a specific category. If speed limit cannot be obtained from the field, a default value can be selected from the range offered in Table 16.

Table 16. Urban Street Design Categories and Typical Speed Limits.

Characteristic	**Design Category**			
	High Speed	**Suburban**	**Intermediate**	**Urban**
Driveway density	Very low density	Low density	Moderate density	High density
Arterial type	Multilane divided; undivided or two lane with shoulders	Multilane divided; undivided or two lane with shoulders	Multilane divided or undivided; one way, two lane	Undivided one way, two way, two or more lanes
Parking	No	No	Some	Significant
Separate left-turn lanes	Yes	Yes	Usually	Some
Signals per mile	0.5-2	1-5	4-10	6-12
Speed limit	45-55 mph	40-45 mph	30-40 mph	25-35 mph
Pedestrian activity	Very little	Little	Some	Usually
Roadside development	Low density	Low to medium density	Medium to moderate density	High density

Determining Saturation Flow Rate

This section describes several newly-developed factors that can be used to estimate the saturation flow rate of a signalized intersection. These factors include:

- adjustment factor for number of lanes, f_N;
- adjustment factor for area population, f_{AP};
- adjustment factor for traffic pressure, f_{TP}; and
- adjustment factor for speed limit, f_{SL}.

Equations for computing each of the aforementioned factors, as well as proposed revisions to the right-turn and heavy-vehicle adjustment factors in the *HCM*, are presented in Table 17 and discussed in the subsequent paragraphs. These equations are intended to be used with Equation 1 to estimate the saturation flow rate for a specific intersection lane group.

In addition to the four newly-developed factors, two additional modifications will be needed to FDOT's ARTPLAN conceptual planning procedure to incorporate the findings of this research. Specifically, the right-turn adjustment factor listed in Table 17 will need to be added to ARTPLAN and the heavy-vehicle equivalency factor E_{HV} in ARTPLAN will need to be updated.

Table 17. Adjustment Factors for Saturation Flow Rate.

Factor	Formula	Definition of Input Variables	Constants & Boundaries [1]
Number of lanes	$f_N = \frac{1}{1 + \frac{1}{N}(E_{CL} - 1)}$	N = number of lanes in the lane group	E_{CL} =1.03
Area population	$f_{AP} = \frac{1}{Pop^{-0.018}}$	Pop = area population in millions of persons (= population × 10^{-6})	$0.0005 \le Pop \le 2.0$ $0.872 \le f_{AP} \le 1.013$
Traffic pressure	$f_{TP} = \frac{1}{1 - 0.0032\,(v_l - 20)}$	v_l = average flow rate per lane for the lane group, veh/ln/cycle	$v_l \le 35$ $f_{TP} \le 1.050$
Speed limit	$f_{SL} = \frac{1}{1 - 0.0066\,(S - 50)}$	S = approach speed limit, mph	$30 \le S \le 55$ $0.883 \le f_{SL} \le 1.034$
Right turns	$f_{RT} = \frac{1}{1 + P_{RT}(E_{RT} - 1)}$	P_{RT} = proportion of right-turn vehicles in the lane group	E_{RT} = 1.07
Heavy vehicles	$f_{HV} = \frac{1}{1 + P_{HV}(E_{HV} - 1)}$	P_{HV} = proportion of heavy vehicles in the lane group	E_{HV} = 1.74

Note:

1 - The area-population, traffic-pressure, and speed-limit adjustment factors are limited to the values shown. If an input variable has a value less than the lower limit shown, then the factor should be computed using the lower limit and the value obtained used for any case where the input value is less than the lower limit. If an input variable has a value more than the upper limit, then the factor should be computed using the upper limit and the value obtained used for any case where the input value is more than the upper limit.

Base Saturation Flow Rate

The base saturation flow rate used in Equation 1 is that of a passenger car traffic stream traveling in a through lane at a busy intersection in an urbanized area where lane widths average 12 ft and there is negligible grade, no on-street parking, no local buses, and negligible pedestrian and bicycle traffic. Under these conditions, a base saturation flow rate of 1950 pc/h/ln is appropriate.

Base Saturation Flow Rate Adjustment Factors

The adjustment factors listed with Equation 1 are documented in Chapter 16 of the *HCM*. The factors developed for this research are summarized in Table 17 and should also be used in Equation 1.

Adjustment Factor for Number of Lanes. The curb lane of a through lane group typically has a lower flow rate than that of the adjacent through lanes. This lower rate is likely due to various sources of "friction" associated with the edge of the roadway (e.g., driveways, pedestrians, signs, poles, and other street furniture) and its proximity to moving traffic. The ratio of the saturation headway in the curb lane to that in an adjacent lane is 1.03 at the typical intersection.

Adjustment Factor for Area Population. Intersections in densely populated areas tend to have higher saturation flow rates than those in lightly populated areas. This trend has been found in several studies and may be a surrogate for driver aggressiveness—where more aggressive drivers are found in areas with higher population and associated congestion. At a population of 5000 persons, this factor equals 0.90. In contrast, at a population of 1,000,000 persons, it equals 1.0.

Adjustment Factor for Traffic Pressure. Measurements of saturation flow rate at busy intersections indicate that drivers tend to adopt shorter headways when discharging from a lengthy queue. This phenomenon is believed to reflect greater anxiety by these drivers during peak traffic hours. A surrogate measure for the traffic-pressure effect is the number of vehicles served per lane during the average signal cycle, v_l. Equation 30 can be used to compute v_l. Default values of v_l are provided in Table 15. An adjustment factor of 1.0 is obtained when v_l equals 20 veh/ln/cycle. This factor was developed using data for through traffic movements. Research is needed to determine the extent to which traffic pressure affects turn movements.

Adjustment Factor for Speed Limit. As suggested by Table 16, the physical elements of the urban street environment tend to be correlated with the posted speed limit. These elements include driveway density, roadside development, sidewalks, on-street parking, and street segment length. When these elements are present in the vicinity of the intersection, they are likely to influence saturation flow rate. This influence has been observed in field measurements of saturation flow rate at intersections on streets having a range of speed limits. Saturation flow rates tend to be lower on low-speed streets. The speed-limit adjustment factor should be used to indirectly evaluate the effect of the urban street environment on performance. It should not be used to explicitly assess the impact of speed (or a change in speed limit) on performance. Also, it should not be used as a basis for recommending a speed limit for a given street. A factor of 1.0 is obtained when the speed limit is 50 mph.

Adjustment Factor for Right Turns. The right-turn adjustment factor is intended to reflect the influence of turn geometry on the saturation flow rate of the right-turn movement. In general, saturation flow rates decrease with an increase in the proportion of right turns in the lane group. The right-turn adjustment factor is based on a right-turn equivalency factor E_{RT} that represents the ratio of the saturation headway of right-turning vehicles to that of through vehicles. Field measurements indicate that this ratio is 1.07 for typical intersection turn geometry.

Adjustment Factor for Heavy Vehicles. The heavy-vehicle adjustment factor is intended to reflect the influence of heavy-vehicle presence on the saturation flow rate of the lane group. In general, saturation flow rates decrease with an increase in the proportion of heavy vehicles in the lane group. The heavy-vehicle adjustment factor is based on a heavy-vehicle equivalency factor E_{HV} that represents the ratio of the saturation headway of heavy vehicles to that of through vehicles. Field measurements indicate that this ratio is 1.74 for the mix of heavy vehicles typically found at intersections in Florida. This value is smaller than the value of 2.0 recommended in Chapter 16 of the *HCM* (*1*) and larger than the value of 1.5 used to develop the heavy-vehicle adjustment factor for Chapter 9 of the 1985 *HCM* (*13*).

Example Application

An example application of the recommended adjustment factors is described in this section. This application compares the estimated saturation flow rate for the through lane group at four intersection approaches. The focus of this application is on the adjustment factors described in the previous section. Any other factors, as listed in Equation 1, are assumed to equal 1.0 (e.g., no grade, no local buses, 12 ft lanes, etc.).

The four intersections selected for this application are hypothetical; they were selected to illustrate the range of conditions represented by the recommended set of factors. One intersection reflects conditions found in a rural area. A second intersection reflects conditions found on a street in a small city. The third and fourth intersections reflect conditions on an arterial street in mid-sized and large cities, respectively. The specific characteristics for each intersection are listed in Table 18.

Table 18. Input Values and Estimates for Four Intersections.

Category	Characteristic	Intersection			
		A	B	C	D
General	Area population (1000's)	3	30	400	1500
Signal	Cycle length, s	90	120	120	120
Roadway	Number of through lanes (both ways)	2	4	6	4
	Exclusive right-turn lane	No	No	No	No
	Design category [1]	High speed (I)	Suburban (II)	Suburban (II)	Intermediate (III)
	Posted speed limit, mph	50	45	45	35
Traffic	Percent heavy-vehicles, %	3	3	2	1.5
	Percent right turns, %	12	12	12	12
	Base saturation flow rate, pc/h/ln	1950	1950	1950	1950
Com-putations	Traffic pressure, veh/ln/cycle [2]	19	26	26	26
	Adjust. factor for number of lanes	0.971	0.985	0.990	0.985
	Adjustment factor for area population	0.901	0.939	0.984	1.007
	Adjustment factor for traffic pressure	0.998	1.019	1.019	1.019
	Adjustment factor for speed limit	1.000	0.968	0.968	0.910
	Adjustment factor for right turns	0.992	0.992	0.992	0.992
	Adjustment factor for heavy vehicles	0.978	0.978	0.985	0.989
	Combined adjustment factor	0.847	0.885	0.939	0.903
	Saturation flow rate, veh/h/ln	**1650**	**1730**	**1830**	**1760**

Notes:
1 - Roman numerals indicate the corresponding arterial class, as defined in the *Quality/Level of Service Handbook* (*10*).
2 - Traffic pressure values based on Table 15 and a target level of service of "D" during the analysis period.

The traffic pressure values were obtained from the defaults provided in Table 15. The equations in Table 17 were used to compute the adjustment factors listed in the bottom half of Table 18. The saturation flow rate at intersections A and B is most influenced by the area-population factor. These intersections are located in areas of relatively small population. The saturation flow rate for intersection D is most influenced by speed limit. This intersection has a relatively low speed limit. The influence is likely an indication of relatively frequent driveways, adjacent sidewalks, on-street parking, and dense roadside development—all of which are factors commonly found in large urbanized areas.

The proposed adjustment factors were also used to estimate the saturation flow rate for a wider range of intersections than depicted in Table 18. For this analysis, the default roadway and traffic characteristics used were those listed in the generalized service tables in FDOT's *Quality/Level of Service Handbook* (*10*). Class I, II, III, and IV streets were considered, with area population ranging from 1000 persons to 1.5 million persons. The number of through lanes ranged from one to four, and speeds varied from 30 to 55 mph. To facilitate comparison, the saturation flow rate obtained using the proposed factors (and recommended base saturation flow rate of 1950 pc/h/ln) was compared with the rate obtained using the factors listed in the *Handbook*, for common combinations of population, roadway, and traffic conditions.

The results of the comparison between the proposed and existing procedures indicate that saturation flow rates in urbanized areas are, on average, about 0.2 percent lower using the proposed factors, relative to those rates obtained using the existing factors. In contrast, the rates in urban and transitioning areas are about 4.8 percent lower using the proposed factors. Finally, the saturation flow rates in rural areas are about 5.7 percent lower using the proposed factors. The rates varied by several percentage points within each area-type category. Detailed results from this analysis are provided in Appendix B.

RECOMMENDED RESEARCH

Four topics of recommended future research were identified during the course of this project. These four topics are outlined in the list below:

- Research is needed to determine if traffic pressure has an effect on left-turn drivers and associated headways. If found, the research should quantify the effect in the form of a saturation flow rate adjustment factor for left-turn lane groups.

- Research is needed to quantify the effect of driveway activity (i.e., vehicles turning into or out of a driveway) on the saturation flow rate of a signalized intersection. The driveways of interest are those located within the functional area of the intersection.

- Research is needed to quantify the effect of heavy vehicles on intersection saturation flow rate. The research would go beyond that conducted for this project by developing a saturation flow rate factor (or factors) that is sensitive to a range of heavy vehicle types (e.g., single-unit truck, tractor-trailer, etc.), their loading, and their distribution in the traffic stream.

CHAPTER 6. REFERENCES

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APPENDIX A.

SAMPLE SIZE ANALYSIS
AND
CORRESPONDING MINIMUM AADT FOR SITE SELECTION

SAMPLE SIZE ANALYSIS

This sample size analysis is based on the measurement of discharge times for the eighth and fourth vehicles. The difference of these times, divided by 4, equals the average saturation headway h_i for signal cycle i. The average of these "cycle averages" is the average saturation headway for the subject approach $\bar{h}$.

Definitions

An "observation" is defined as the average saturation headway for one signal cycle. It is computed as:

$$h_i = \frac{T_8 - T_4}{8 - 4} = \frac{\sum_1^8 h_j - \sum_1^4 h_j}{8 - 4} = \frac{\sum_5^8 h_j}{8 - 4} \qquad \textbf{(A-1)}$$

where:

h_i = average saturation headway of the 5th through 8th queue positions for signal cycle i, s/veh; and

T_j = discharge time of the vehicle in queue position j, s.

Assumptions

Examination of Equation A-1 indicates that h_i represents the sum of four random headways. For sample size estimation, a conservative assumption is that the four random values are independent and identically distributed. Thus, the standard deviation of h_i can be computed as:

$$S_{\bar{h}} = \frac{S_h}{\sqrt{8 - 4}} = \frac{0.5}{\sqrt{4}} = 0.25 \qquad \textbf{(A-2)}$$

where:

$S_{\bar{h}}$ = standard deviation of $\bar{h}$, s/veh; and

S_h = standard deviation of saturation headway (= 0.5 s/veh).

Analysis

The minimum sample size is computed using the following equation:

$$
\begin{aligned}
N_o &= \left(\frac{z \, S_{\bar{h}_{58}}}{e} \right)^2 \\
&= \left(\frac{1.96 \times 0.25}{0.06} \right)^2 \\
&= 67 \quad (= n_h / 4)
\end{aligned}
\tag{A-3}
$$

where:

N_o = minimum number of signal cycles for which a value of h_i is computed;
z = standard normal variate (= 1.96 for two-tail 95 percent confidence interval); and
e = allowable error in estimate of mean headway (= 0.06 s/veh).

Equation A-3 indicates that 67 signal cycles need to be observed to obtain an estimate of the average saturation headway $\bar{h}$ with a 95 percent level of confidence. At least eight queued vehicles would have to be observed each cycle in the traffic lane of interest. The average saturation headway for these 67 cycles will be within ±0.06 s/veh of the true mean headway, with a 5 percent chance of it being outside this range. This error range equates to ±52 veh/h/ln for the corresponding true mean saturation flow rate. Collectively, the 67 cycle observations of h_i are the equivalent to 268 vehicle-observations from queue positions five through eight (i.e., 268 = 4 × 67).

MINIMUM AADT FOR SITE SELECTION

The field survey method requires the movement being studied to experience 67 eight-vehicle (or longer) queues during the study period. However, it is possible that some cycles will not experience the requisite number of 8 or more vehicle queues because of: (1) temporal volume variation or (2) randomness in arrivals each cycle.

Based on an assumption of Poisson arrivals, the expected number of observations during any one cycle is:

$$
n_o = P(n \geq N_q) \times \frac{3600}{C} \times (N_q - 4) \tag{A-4}
$$

with

$$
P(n \geq N_q) = 1 - \sum_{n=0}^{n=N_q-1} \frac{q^n e^{-q}}{n!} \tag{A-5}
$$

where:

n_o = number of vehicles for which a value of h_i is computed, veh/h;
$P(n \geq N_q)$ = probability that the number of vehicles in queue n is N_q or more;
N_q = queue position used to estimate h_i (= 8 for this study), veh/cycle;

C = cycle length, s;
q = arrivals on red (= Q / 3600 × [C - g]), veh/cycle;
Q = subject lane volume (= AADT /N_c × k), veh/h/ln;
N_c = number of lanes in the cross section at the intersection that serve through vehicles;
k = ratio of average hour volume to AADT; and
g = green interval duration, s.

The flow rate was estimated for each of the daylight hours during the day using the AADT for the subject lane as input and a set of hourly volume distribution factors for urban intersections. The volumes were sorted from highest to lowest. For a given lane AADT, Equation A-4 was used to estimate the number of vehicles observed during each of the daylight hours, starting with the highest volume hour. The number of hours of study was then estimated as the minimum number of hours that, in total, yield the desired sample size of 268 vehicle-observations. This process is illustrated in Table A-1.

Table A-1. Study Duration Analysis Based on Sample Size Requirement.

Hour	AADT Distribution (k)	Lane Volume, veh/h/ln [1]	Arrivals on Red, veh/cycle [2]	P(n≥N)	Vehicle Observations (n_o), veh/h	Cumulative Vehicle-Obs., veh
1	0.097	359	8.0	0.55	66	66
2	0.094	348	7.8	0.51	62	128
3	0.090	333	7.4	0.47	56	184
4	0.085	315	7.0	0.40	49	232
5	0.079	292	6.5	0.33	40	272
6	0.072	266	6.0	0.25	30	302
7	0.064	237	5.3	0.17	20	322
8	0.055	203	4.5	0.09	11	332

Notes:
1 - Based on a lane AADT of 3700 veh/d/ln.
2 - Based on a cycle length of 120 s and a green interval duration of 40 s.

The last column of Table A-1 indicates the cumulative number of vehicle-observations for each successive hour of study. It is based on a lane AADT of 3,700 veh/d/ln. As indicated by the first row of the table, if the study duration is limited to the peak traffic hour, a total of 66 vehicles in queue positions 5 through 8 would be observed in the subject lane. If the study duration includes two peak hours, then a total of 128 vehicles would be observed (66 in the first hour and 62 in the second hour). This process continues such that the study of the highest-volume five hours is expected to yield 272 vehicle-observations. Because this value exceeds the desired sample size of 268 vehicles (as computed in the previous section), it is concluded that a minimum AADT of 3700 veh/d/ln is needed if the study is to be completed in five hours.

The analysis described in the preceding paragraphs was repeated for a range of AADTs and approach lane configurations. The results of this analysis are shown in Figure A-1.

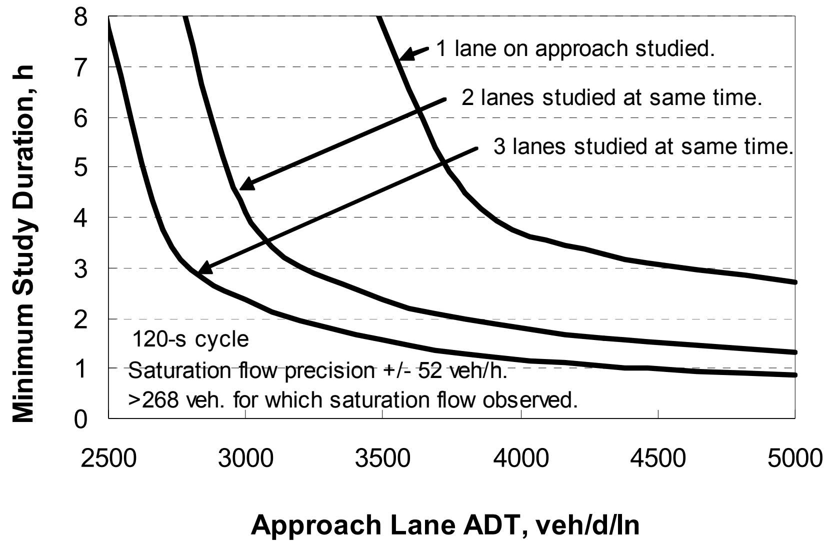

Figure A-1. Minimum Study Duration.

Figure A-1 is based on a 120-s cycle and a *g/C* ratio for the subject movement of 0.33. A longer cycle length will reduce the minimum study duration, hence this figure is conservative when applied to traffic movements controlled by longer cycle lengths.

The "knee" of each curve in Figure A-1 is about four hours, regardless of the number of lanes studied. This threshold duration was used to define the minimum lane AADT, lower AADTs would not likely be cost-effective to study. The minimum lane AADTs from Figure A-1 are listed below by approach configuration:

- exclusive right-turn lane: 4000 veh/d/ln;
- one through lane: 4000 veh/d/ln;
- two through lanes: 3200 veh/d/ln;
- three through lanes: 2900 veh/d/ln; and
- a single lane with mixed through and right turns: 4000 veh/d/ln.

It may be difficult to find sites that have an exclusive right-turn lane with a volume of 4000 veh/d/ln, especially if the right-turn movement is on an approach with only one through lane, if right turn on red is heavily used, or if overlap right-turn phasing is provided.

APPENDIX B.

ESTIMATED SATURATION FLOW RATES FOR INTERSECTIONS ON FLORIDA STREETS

Table B-1. Saturation Flow Rates Estimated Using Proposed and Existing Adjustment Factors.

Area Type & Cycle Length, s	Population (1000s)	Class	Lanes	Speed Limit, mph	Percent Heavy Vehicles	Traffic Pressure veh/ln/cyc	f_RT	f_N	f_AP	f_TP	f_SL	f_HV	Combined f	Sat. Flow veh/h/ln	FDOT f_local	f_hv	Combined f	Sat. Flow veh/h/ln	Percent Change
							PROPOSED FACTORS								EXISTING FDOT Q/LOS HANDBOOK				
					Target LOS:	D	Base Sat:	1950	pc/h/ln							Base Sat:	1900		
							% Rights	12.00	%										
Urbanized	1500	1	2	45	2	26	0.992	0.971	1.007	1.019	0.968	0.985	0.943	**1840**	1.000	0.980	0.980	**1860**	**-1.1**
120	1500	1	4	50	2	26	0.992	0.985	1.007	1.019	1.000	0.985	0.988	**1930**	1.000	0.980	0.980	**1860**	**3.8**
	1500	1	8	50	2	26	0.992	0.993	1.007	1.019	1.000	0.985	0.996	**1940**	0.950	0.980	0.931	**1770**	**9.6**
	1500	2	2	45	2	26	0.992	0.971	1.007	1.019	0.968	0.985	0.943	**1840**	0.980	0.980	0.961	**1830**	**0.5**
	1500	2	4	45	2	26	0.992	0.985	1.007	1.019	0.968	0.985	0.957	**1870**	0.980	0.980	0.961	**1830**	**2.2**
	1500	2	8	45	2	26	0.992	0.993	1.007	1.019	0.968	0.985	0.964	**1880**	0.950	0.980	0.931	**1770**	**6.2**
	1500	3	2	35	1.5	26	0.992	0.971	1.007	1.019	0.910	0.989	0.889	**1730**	0.950	0.985	0.936	**1780**	**-2.8**
	1500	3	4	35	1.5	26	0.992	0.985	1.007	1.019	0.910	0.989	0.903	**1760**	0.950	0.985	0.936	**1780**	**-1.1**
	1500	3	8	35	1.5	26	0.992	0.993	1.007	1.019	0.910	0.989	0.909	**1770**	0.920	0.985	0.906	**1720**	**2.9**
	1500	4	2	30	1.5	26	0.992	0.971	1.007	1.019	0.883	0.989	0.863	**1680**	0.920	0.985	0.906	**1720**	**-2.3**
	1500	4	4	30	1.5	26	0.992	0.985	1.007	1.019	0.883	0.989	0.876	**1710**	0.920	0.985	0.906	**1720**	**-0.6**
	1500	4	8	30	1.5	26	0.992	0.993	1.007	1.019	0.883	0.989	0.883	**1720**	0.900	0.985	0.887	**1680**	**2.4**
	200	1	2	45	2	26	0.992	0.971	0.971	1.019	0.968	0.985	0.909	**1770**	1.000	0.980	0.980	**1860**	**-4.8**
	200	1	4	50	2	26	0.992	0.985	0.971	1.019	1.000	0.985	0.953	**1860**	1.000	0.980	0.980	**1860**	**0.0**
	200	1	8	50	2	26	0.992	0.993	0.971	1.019	1.000	0.985	0.960	**1870**	0.950	0.980	0.931	**1770**	**5.6**
	200	2	2	45	2	26	0.992	0.971	0.971	1.019	0.968	0.985	0.909	**1770**	0.980	0.980	0.961	**1830**	**-3.3**
	200	2	4	45	2	26	0.992	0.985	0.971	1.019	0.968	0.985	0.923	**1800**	0.980	0.980	0.961	**1830**	**-1.6**
	200	2	8	45	2	26	0.992	0.993	0.971	1.019	0.968	0.985	0.929	**1810**	0.950	0.980	0.931	**1770**	**2.3**
	200	3	2	35	1.5	26	0.992	0.971	0.971	1.019	0.910	0.989	0.858	**1670**	0.950	0.985	0.936	**1780**	**-6.2**
	200	3	4	35	1.5	26	0.992	0.985	0.971	1.019	0.910	0.989	0.870	**1700**	0.950	0.985	0.936	**1780**	**-4.5**
	200	3	8	35	1.5	26	0.992	0.993	0.971	1.019	0.910	0.989	0.877	**1710**	0.920	0.985	0.906	**1720**	**-0.6**
	200	4	2	30	1.5	26	0.992	0.971	0.971	1.019	0.883	0.989	0.833	**1620**	0.920	0.985	0.906	**1720**	**-5.8**
	200	4	4	30	1.5	26	0.992	0.985	0.971	1.019	0.883	0.989	0.845	**1650**	0.920	0.985	0.906	**1720**	**-4.1**
	200	4	8	30	1.5	26	0.992	0.993	0.971	1.019	0.883	0.989	0.851	**1660**	0.900	0.985	0.887	**1680**	**-1.2**
Urban or	45	1	2	45	3	26	0.992	0.971	0.946	1.019	0.968	0.978	0.879	**1710**	0.980	0.971	0.951	**1810**	**-5.5**
Transition	45	1	4	50	3	26	0.992	0.985	0.946	1.019	1.000	0.978	0.921	**1800**	0.980	0.971	0.951	**1810**	**-0.6**
120	45	2	2	45	3	26	0.992	0.971	0.946	1.019	0.968	0.978	0.879	**1710**	0.950	0.971	0.922	**1750**	**-2.3**
	45	2	4	45	3	26	0.992	0.985	0.946	1.019	0.968	0.978	0.892	**1740**	0.950	0.971	0.922	**1750**	**-0.6**
	45	3	2	35	2	26	0.992	0.971	0.946	1.019	0.910	0.985	0.832	**1620**	0.920	0.980	0.902	**1710**	**-5.3**
	45	3	4	35	2	26	0.992	0.985	0.946	1.019	0.910	0.985	0.844	**1650**	0.920	0.980	0.902	**1710**	**-3.5**
	5	1	2	45	3	26	0.992	0.971	0.909	1.019	0.968	0.978	0.845	**1650**	0.980	0.971	0.951	**1810**	**-8.8**
	5	1	4	50	3	26	0.992	0.985	0.909	1.019	1.000	0.978	0.885	**1730**	0.980	0.971	0.951	**1810**	**-4.4**
	5	2	2	45	3	26	0.992	0.971	0.909	1.019	0.968	0.978	0.845	**1650**	0.950	0.971	0.922	**1750**	**-5.7**
	5	2	4	45	3	26	0.992	0.985	0.909	1.019	0.968	0.978	0.857	**1670**	0.950	0.971	0.922	**1750**	**-4.6**
	5	3	2	35	2	26	0.992	0.971	0.909	1.019	0.910	0.985	0.800	**1560**	0.920	0.980	0.902	**1710**	**-8.8**
	5	3	4	35	2	26	0.992	0.985	0.909	1.019	0.910	0.985	0.811	**1580**	0.920	0.980	0.902	**1710**	**-7.6**
Rural	5	1	2	45	3	19	0.992	0.971	0.909	0.998	0.968	0.978	0.827	**1610**	0.920	0.971	0.893	**1700**	**-5.3**
90	5	1	4	45	3	19	0.992	0.985	0.909	0.998	0.968	0.978	0.839	**1640**	0.920	0.971	0.893	**1700**	**-3.5**
	1	1	2	45	3	19	0.992	0.971	0.883	0.998	0.968	0.978	0.804	**1570**	0.920	0.971	0.893	**1700**	**-7.6**
	1	1	4	45	3	19	0.992	0.985	0.883	0.998	0.968	0.978	0.815	**1590**	0.920	0.971	0.893	**1700**	**-6.5**
														Average				**Average**	**Average**
Urbanized														**1773**				**1777**	**-0.2**
Transition														**1673**				**1757**	**-4.8**
Rural														**1603**				**1700**	**-5.7**