Districtwide Miscellaneous PD&E Studies

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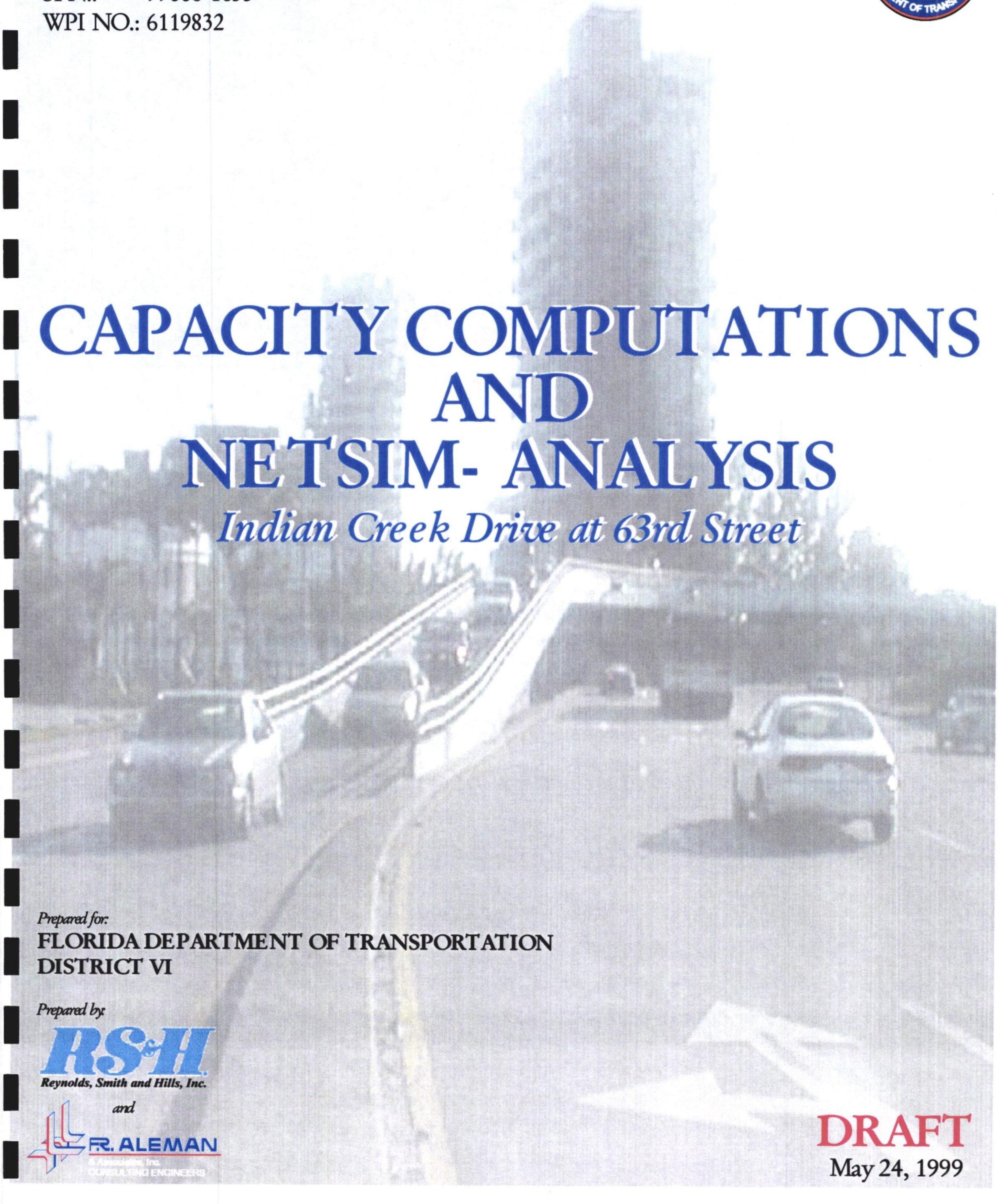

CAPACITY COMPUTATIONS AND NETSIM- ANALYSIS

Indian Creek Drive at 63rd Street

Prepared for:
FLORIDA DEPARTMENT OF TRANSPORTATION
DISTRICT VI

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1.0 INTRODUCTION

The purpose of this technical memorandum was to develop capacity estimates for the eastbound left-turn movements at 63rd street and Indian Creek Drive for three alternative traffic operating conditions. The three alternative operating conditions are as follows:

- Existing Flyover
- New Flyover
- At-grade, Triple-Left-Turn Lanes

The three alternative operating conditions are described in detail in the FDOT report titled 63rd Street at Indian Creek Drive Intersection, Final Preliminary Analysis, Evaluation, and Recommendation Report, January 29, 1999. An animation model was also developed for the three alternatives using TRAF-NETSIM.

2.0 CAPACITY COMPUTATIONS

The lane group capacities for eastbound left turn movements at the study intersection can be estimated using equations 9-3 and 9-12 from the Highway Capacity Manual (HCM). These equations specifies the procedures for computing lane group capacities and saturation flow rates at an intersection. The relationships are as follows:

$$c_i = s_i\ (g_i/C) \quad \text{..(9-3)}$$

where

c_i = capacity of lane group i, vph;
s_i = saturation flow rate for lane group i, vphg;
g_i/C = effective green ratio for lane group i;

$$s = s_o\ N\ f_w\ f_{HV}\ f_g\ f_p\ f_{bb}\ f_a\ f_{RT}\ f_{LT} \quad \text{....................(9-12)}$$

where

s = saturation flow rate for the subject lane group, vphg.
s_o = ideal saturation flow rate per lane, pcphgpl;
N = number of lanes in the lane group;
f_w = adjustment factor for lane width (12-ft lanes are standard);
f_{HV} = adjustment factor for heavy vehicles in the traffic stream;
f_g = adjustment factor for approach grade;
f_p = adjustment factor for the existence of a parking lane adjacent to the lane group and the parking activity in that lane;
f_{bb} = adjustment factor for the blocking effect of local buses that stop within the intersection area;
f_a = adjustment factor for area type;
f_{RT} = adjustment factor for right turns in the lane group;
f_{LT} = adjustment factor for left turns in the lane group.

In Table 5-6 of the Highway Capacity Manual, the saturation flow rate for a single lane ramp with free flow of 30 m.p.h. is estimated at 1900 pcph (see Appendix A). In studies conducted by the ITE the capacities for triple left turn lanes were estimated at 1830 pcphgpl (see Appendix B, Capacities of Triple Left Turn Lanes, ITE, 1995). The ITE study also found no significant correlation between the geometric factors and the calculated saturation flow rate. Applying these saturation flow estimates to equations 9-3 and 9-12, the capacity for the alternative operating conditions can be computed. Table 1 shows the computed capacities for the intersection.

TABLE 1
CAPACITY ANALYSIS WORKSHEET- HCM PROCEDURE
63rd STREET AT INDIAN CREEK DRIVE - EASTBOUND LEFT TURNS

FACTOR	EXISTING FLYOVER	NEW FLYOVER	AT-GRADE TRIPLE LEFT
s_o	1900[1]	1900[1]	1830[2]
N	1	1	3
f_w	0.900	1.000	0.933
f_{HV}	0.98	0.98	0.98
f_g	0.960	0.970	1.00
f_p	1.00	1.00	1.00
f_{bb}	1.00	1.00	1.00
f_a	1.00	1.00	1.00
f_{RT}	1.00	1.00	1.00
f_{LT}	1.00	1.00	1.00
s (vehs/hr of green)	1668	1806	5211
g_i/C[3]	1.0	1.0	0.57
Capacity (vehs/hr)	1609	1806	2861
Demand (year 2021)	1700	1700	1700
volume/capacity	1.06	0.94	0.59

Notes:
1. Saturation flow rate per HCM Table 5-6.
2. Saturation flow rate per ITE, Capacities of Triple Left-Turn Lanes, 1995.
3. g/C ratio for PM peak year 2021, per Traffic Report, Indian Creek at 63rd Street, January 27, 1999.

3.0 TRAF-NETSIM ANALYSIS

An analysis of the three alternative traffic operating conditions was conducted using TRAF-NETSIM. The analysis was conducted for the PM peak conditions in year 2021. The NETSIM analysis was used to produce traffic animation models for the three operating conditions. Exhibits 1 through 3 show typical operating conditions from the TRAF-NETSIM models. Table 2 shows the measures of effectiveness generated by the models for the three alternative traffic operating conditions.

TABLE 2
SUMMARY NETSIM ANALYSIS

ALTERNATIVE 1 - AT-GRADE TRIPLE LEFT

Measure of Effectiveness	Pine Tree Dr. to Alton Rd.	Alton Rd. to Indian Creek
Maximum Queue Length (vehs)	10	7
Delay (secs/veh)	7.5	15.5

ALTERNATIVE 2 - NEW FLYOVER

Measure of Effectiveness	Pine Tree Dr. to Alton Rd.	Alton Rd. to Indian Creek
Maximum Queue Length (vehs)	27	5
Delay (secs/veh)	89.2	30.9

ALTERNATIVE 3 - EXISTING FLYOVER

Measure of Effectiveness	Pine Tree Dr. to Alton Rd.	Alton Rd. to Indian Creek
Maximum Queue Length (vehs)	27	5
Delay (secs/veh)	91.6	33.9

EXHIBIT 1
NETSIM ANALYSIS
ALTERNATIVE 1 - TRIPLE LEFT

EXHIBIT 2
NETSIM ANALYSIS
ALTERNATIVE 2 - NEW FLYOVER

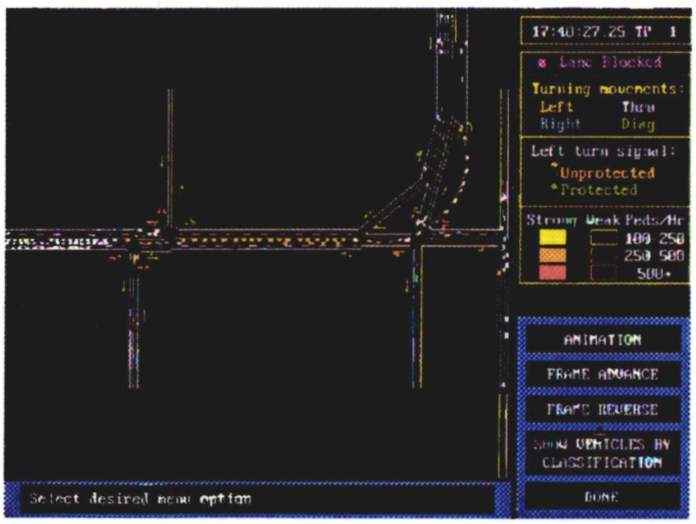

EXHIBIT 3
ALTERNATIVE 3 -EXISTING FLYOVER

CONCLUSIONS

Based on the computations above the estimated capacities for the eastbound left turn movements in the three operating conditions are as follows:

TABLE 3
CAPACITY ANALYSIS SUMMARY
EASTBOUND LEFT TURN MOVEMENT

ALTERNATIVE	CAPACITY(vehs/hr.)	V/C RATIO	Max Queue (vehs)*
Triple-Left Turn	2850	1.06	10
New Flyover	1800	0.94	27
Existing Flyover	1600	0.59	27

* Maximum queue eastbound at Allison Road

APPENDIX A

Extract from HCM

Table 5-6

CAPACITY OF RAMP ROADWAYS

Because most operational problems occur at ramp terminals (either the ramp-freeway terminal or the ramp-street terminal), there is little information regarding the operational characteristics of ramp roadways themselves. Some basic design standards exist in AASHTO policies (*1*), but they are not related to specific operational characteristics. In the 1970s, Leisch (*3*) adapted this material to provide a broader set of criteria that were, again, unrelated to specific operational characteristics. Thus, information presented in this section is for general guidance only.

Ramp roadways differ from the freeway mainline in the following ways:

1. Ramps are roadways of limited length and width (often just one lane).
2. The free-flow speed of the ramp is frequently lower than that of the roadways it connects, particularly the freeway.
3. On single-lane ramps, where passing is not possible, the adverse effect of trucks and other slow-moving vehicles is more pronounced than on a multilane roadway.
4. Acceleration and deceleration often take place on the ramp itself.
5. At ramp-street junctions, queueing may develop on the ramp, particularly if the ramp-street junction is signalized.

Table 5-6 gives approximate criteria for the capacity of ramp roadways. These capacities are based on recent studies (*2*) and previously noted work conducted in the 1970s (*3*).

Table 5-6 gives the approximate capacity of the ramp roadway itself, *not* the ramp-freeway terminal. There is no evidence, for example, that a two-lane on-ramp freeway terminal can accommodate any more vehicles than a one-lane ramp terminal without the addition of a lane (in which case the configuration becomes a major merge area).

Thus, it is unlikely that two-lane on-ramps can accommodate more than 2,200 pcph through the merge area itself. The two-lane configuration will achieve a merge with less turbulence and a higher level of service but will not increase the capacity of the merge, which is controlled by the capacity of the downstream freeway section. For higher on-ramp flows, a two-lane on-ramp must be used in conjunction with a lane addition and a major merge configuration.

TABLE 5-6. APPROXIMATE CAPACITY OF RAMP ROADWAYS

FREE-FLOW SPEED OF RAMP, S_{FR} (MPH)	CAPACITY (PCPH)	
	SINGLE-LANE RAMPS	TWO-LANE RAMPS
>50	2,200	4,400
41-50	2,100	4,100
31-40	2,000	3,800
21-30	1,900	3,500
<21	1,800	3,200

Two-lane off-ramps can accommodate higher ramp flows through the diverge area than single-lane off-ramps, although high observations are in the 4,000-pcph range. Such high off-ramp flows, however, often leave the continuing freeway section with relatively low per-lane flow rates. A major diverge configuration can be considered and may more effectively balance the per-lane flows on each departing leg.

Even where a single-lane merge or diverge configuration is used, there are several reasons to consider widening the ramp to two lanes outside the terminal areas, including the following:

1. When the ramp is longer than 1,000 ft, a second lane allows drivers to pass stalled or slow-moving vehicles. This can also be accomplished with a single-lane ramp and a paved shoulder of 8 ft or more.
2. When queues are expected to form at signalized and other ramp-street terminals, an additional ramp lane provides additional storage capacity.
3. When the ramp has a steep grade or other minimal geometrics, a second ramp lane again allows drivers to pass slow-moving vehicles.

In such cases, the two-lane ramp is tapered to a single lane in advance of the ramp-freeway terminal.

IV. SAMPLE CALCULATIONS

CALCULATION 1: ISOLATED ON-RAMP

Problem

An on-ramp on a four-lane freeway with standard 11.8-ft (3.6-m) lane widths and adequate clearances serves a demand of 550 vph (5 percent trucks). The freeway mainline approaching the ramp carries 2,500 vph (10 percent trucks). The terrain is level, PHF is 0.90, and the ramp has an acceleration lane with a total length of 750 ft. Free-flow speeds are 60 mph for the freeway and 45 mph for the ramp. Drivers are primarily regular users of the facility. At what level of service would this ramp be expected to operate?

Solution

A sketch of this section is shown in Figure 5-10, the worksheet for this calculation.

The first computation must be the conversion of all demand volumes to flow rates in passenger cars per hour under ideal conditions. For each demand flow, the PHF is given as well as information that allows the determination of f_{HV} and f_p. These factors are selected according to the procedures in Chapter 3. The driver population factor, f_p, is 1.00, because no special characteristics are noted. For level terrain, the passenger car equivalent for trucks is 1.5 per truck, yielding an f_{HV} of $1/[1 + 0.10\ (1.5 - 1)] = 0.952$ for freeway volume and $1/[1 + 0.05\ (1.5 - 1)] = 0.976$ for ramp

APPENDIX B

Extract from ITE Study

Capacities of Triple Left Turn Lanes

An Informational Report of the
Institute of Transportation Engineers

Capacities of Triple Left-Turn Lanes

Prepared by ITE Technical Council Committee 5P-5A

April 1995

Summary of Findings

The objective of ITE Committee 5P-5A was to investigate the capacities of triple left-turn lanes and correlate the findings with roadway geometric features. Technical Committee 5P-5A was formed to extend the work of Committee 5P-5 (Capacities of Multiple Left-Turn Lanes), which examined the capacities and operating characteristics of double left-turn lanes. The findings of Committee 5P-5 are documented in the ITE Informational Report, *Capacities of Multiple Left-Turn Lanes* (Publ No. IR-065)[1].

The committee's objective was accomplished through a review of the literature and the collection and analysis of data at 17 intersections with triple left-turn lane installations. The key findings of the literature review, the data collection and analysis efforts, and the conclusions drawn by the Committee are summarized below.

LITERATURE REVIEW

The literature review revealed only one study concerning the capacities of triple left-turn lanes. That study was conducted by Leonard[2] for the Division of Traffic Operations, California Department of Transportation in 1993. Leonard's sample data consisted of 4,742 lane-cycles and 34,898 vehicles from five triple left turn sites in Orange County, California. Leonard reported an overall saturation flow rate of 1,928 passenger cars per hour of green per lane (pcphgpl) for the five study sites. Leonard[2] also examined the variations in the calculated saturation flows at the study sites. The analyses revealed no significant differences in saturation flow rates when categorized by site, by weekday (Monday through Friday), or by observer. Significant differences were observed between lanes (inner, outer and middle), by time-of-day (AM, midday, and PM), and weekday vs. weekend[2].

The literature search also identified two studies[4,5] concerning the design and operating characteristics of triple left-turn lanes.

DATA COLLECTION AND ANALYSIS

The data collection and analysis efforts followed the basic procedures recommended by the *Highway Capacity Manual* (HCM)[3]. The data analysis phase consisted of 1) calculating saturation flow rates for the triple left-turn lanes and adjacent through lanes; 2) calculation and analysis of triple left-turn factors; 3) analysis of variance tests to investigate differences in flow rates between the approach lanes; and, 4) a preliminary examination of geometric factors affecting left-turn saturation flow rates.

CONCLUSIONS

The results of this limited study indicate an overall saturation flow rate for triple left-turn lanes of approximately 1,830 pcphgpl. This saturation flow rate is within about 5 percent of the rates reported by Leonard[2] for triple left-turn lanes and by ITE Committee 5P-5[1] for double left-turn lanes. The results reported by Leonard[2] for triple left-turn lanes and by ITE Committee 5P-5[1] for double left-turn lanes suggest multiple left-turn lane saturation flow rates of 1,928-1,950 pcphgpl, respectively. The results of this study also indicate that there may be no significant difference in the saturation flow rates between each of the three turn lanes, or between any of the turn lanes and the adjacent through lane. This result suggests a left-turn adjustment factor (fLT) of 1.00 may be appropriate for triple left-turn lanes. ITE Committee 5P-5[1] reported similar results for double left-turn lanes. Leonard[2], however, reported a significant difference in the saturation flow rates between each of the left-turn lanes (inner, middle and outer lanes).

Due to the small sample size and missing geometric data for many of the intersections, only very preliminary investigations of the effects of geometric factors on triple left-turn lane saturation flow rates were possible. Based on the limited database of this study, no geometric factors were found to be related to triple left-turn lane saturation flow.

The results of this study are based on a small sample. The study sites were not randomly selected and many of the sites have missing data. Also, some of the sites may not be typical triple left-turn lane installations with regards to driver behavior (most of the study sites are in California), grade, proximity of adjacent intersections, and intersection configuration (for instance, off-ramps, and so forth). The results of this study, however, should not be summarily dismissed as entirely exploratory in nature. The small, nonrandom sample collected in this study, for example, is probably largely attributable to the relative scarcity of triple left-turn lane installations. The basic approach was to collect data from as many triple left-turn lane installations as could be identified by committee members in their local areas. In addition, many of the triple left-turn lane installations investigated in this study were relatively new and, in some cases, never reached saturation flow conditions. Finally, several committee members suggested that the greatest influence on triple left-turn lane use may be downstream conditions (major generator, the close proximity of another intersection, geometric restriction, and so forth) that might cause drivers to pre-position themselves in the turn lane most favorable to their immediate downstream destination. Assmus[6] refers to this phenomenon as "downstream attraction bias." The lack of evidence found in this study concerning any significant relationships between intersection geometry and triple left-turn lane saturation flow rates may be attributable, in part, to the existence of this downstream attraction bias.

In any event, additional research should be conducted to develop more precise estimates of triple left-turn lane saturation flow rates and to quantify the geometric factors which may affect these flow rates.