Operational Characteristics of Triple Left Turns

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A study of the characteristics of triple left turns was performed. Five triple-left-turn sites in Orange County, California, were identified. Manual saturation flow rate studies at each site were performed using electronic counter boards. Queue discharge times and vehicle arrivals were compiled for all vehicles by lane and by signal cycle. Across all five sites, a sample consisting of 4,742 lane cycles and 34,898 vehicles was compiled for analysis. On average, these triple left turns supported flows of 795 vehicles per hour, received a 19 percent split of the total cycle time and spent 57 percent of that split time servicing the queue. Computing the saturation flow rate using the method suggested by the 1985 *Highway Capacity Manual*, the average saturation flow rate observed was 1,930 vehicles per hour of green per lane. Variations in the saturation flow rate observed at the triple-left-turn sites were explored. Results reveal no significant differences in saturation flow rates when categorized by site, by weekdays (e.g., Monday through Friday), or by observer. Significant differences were observed between lanes (e.g., inner and middle versus outer), time of day (e.g., morning, midday, and evening) and weekday versus weekend.

A triple left turn is a left-turning movement with three lanes available to the turning vehicles. U-turns may or may not be allowed from the innermost lane, and through movements may or may not be allowed from the outermost lane. In theory, a triple left turn provides additional capacity at the stop line, providing for a greater discharge of turning vehicles over a shorter amount of time. This may result in shorter cycle times at the intersection and additional green time for other traffic movements within a fixed time budget.

Traffic professionals have expressed several concerns over the installation of triple left turns. In general, these concerns may be categorized into safety issues and operational issues. Safety-related questions focus on driver traversal of the triple left turn. For example, special road bumps ("cat tracks") have been installed to channel the drivers correctly through the left turn and into the adjoining approach. Other safety concerns include truck and bus negotiation through such turns.

Operational concerns related to triple left turns focus on the perceived increase in capacity at the stop line. Stop line capacity is usually described as "saturation flow." Teply and Jones (*1*) present an excellent review of different descriptions of saturation flow. For purposes of this paper, the definition presented in Chapter 9 of the *Highway Capacity Manual* (HCM) (*2*) is adopted.

Although it is recognized that a triple left turn provides an absolute increase in the number of turn lanes for the left-turning movement, this increased capacity may be offset by a reduction in the discharge (saturation flow) rates due to driver "cautiousness" during turn traversal, especially of drivers in the center lane. For example, the HCM (*2*, Table 9–12) suggests that adjustment factors of 0.95 and 0.92 be applied to the ideal saturation flow rate to exclusive, protected single and dual left-turning movements, respectively.

The principal goal of this research is to provide an assessment of the current operational experience with triple left turns in California. The objectives of this research are to document operating characteristics associated with triple left turns, including flows serviced by the turns, saturation flow rates, and various signal timing characteristics.

DATA

Five triple-left-turn sites were selected on the basis of discussions with the California Department of Transportation (Caltrans); all of the sites were located in Orange County, in Southern California. Table 1 presents a summary of these sites along with other pertinent data. Figures 1 through 4 present aerial photographs of four of the triple left turn sites; aerial photographs of the Paseo De Valencia at Los Alisos site (Site 3) were not available. Sites 1, 4, and 5 are located within 2 km of each other and are located on the Pacific Coast Highway (PCH) near Newport Bay within the city of Newport Beach. As-built plans were available for these three sites; lane widths for these three sites are given in Table 1. Geometric plans for Sites 2 and 3 were unavailable. Lane usages were obtained through field observations at each site.

Signal phasings at each site are varied. All sites present protected-only signal phasing to the triple left turns. Sites 2 and 3, because of restricted geometrics (e.g., freeway off-ramp and T-intersection), display five and four signal phases, respectively. Site 5 uses six signal phases, displaying green to combined left and through movements to the triple-left-turn approach (i.e., split phase). Sites 1 and 4 display eight traffic signal phases.

Field observations were collected during daylight hours; the weather during all observation periods was clear and dry. Approaches at all sites are generally level (e.g., zero grade.) On-street parking is prohibited at all sites. All sites are located in generally suburban regions. Truck and other heavy vehicle percentages were not explicitly recorded, although all sites could be best characterized as having relatively low percentages (e.g., 0 to 2 percent) of large vehicles.

METHODOLOGY

Data were collected manually using an electronic counting board (*3*) capable of recording keystroke-time stamp combinations to a resolution of 1/64 sec; using this information, it is possible to construct the time histories of vehicles discharging from the stop line. These types of counting boards also enable one observer to record

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TABLE 1 Summary of Triple-Left-Turn Sites

Site Code	Description	City	Short Name	State Route?	Signal Phases	Lane Usage	Lane Widths Inner	Outer	Middle
1	EB Dover to SB Pacific Coast Highway	Newport Beach	PCH/Dover	Yes	8	L-L-L	11'	11'	11'
2	SB Interstate 5 to EB Lake Forest Road	Lake Forest	I-5/LakeF	Yes	5	L-L-LT	*	*	*
3	NB Paseo De Valencia to WB Los Alisos	Laguna Niguel	PDV/LosAl	No	4	L-L-L	*	*	*
4	EB Pacific Coast Highway to NB Jamboree Blvd	Newport Beach	PCH/Jamb	Yes	8	L-L-L	11'	11'	11'
5	NB Bayside Drive to WB Pacific Coast Highway	Newport Beach	PCH/BaySi	Yes	6	L-L-LT	11'	13'	10'

* Lane widths for Sites 2 and 3 were unavailable.

the discharge patterns of all three lanes of the triple left turn movement simultaneously. Results of the field observations may then be translated into cycle-by-cycle summaries of the discharge patterns of the vehicles.

The overall project methodology is as follows:

1. Perform field observations at each site,
2. Construct cycle-by-cycle summaries from the raw data files, and
3. Perform statistical analysis of data.

Field Observation Methodology

The data collection procedure adopted for this field survey is very similar to the manual method described in the HCM (*2*), the principal differences being that a single observer may record data over several lanes and that the discharge times of all vehicles may be recorded. For this field survey, a custom data collection procedure was designed and custom software for data reduction was developed.

Reference point selection and observer positioning follow the recommendations of the HCM (*2*, Chapter 9, Appendix IV). The observer first selects a reference point near the stop bar that is common to all lanes of the triple left turn and then positions himself or herself at the intersection such that the reference points in each lane are visible. For this study of triple left turns, the rear bar of the pedestrian crosswalk was selected as the reference point.

Construction of Cycle-by-Cycle Summaries

Given that a series of keystroke data files have been uploaded from the electronic counter board, the next step of the study methodology involves the construction of cycle-by-cycle data summaries. (The author will provide a detailed description of this procedure on request.) These summaries will be used as input into the next step (statistical analysis). For purposes of this study, a cycle is defined

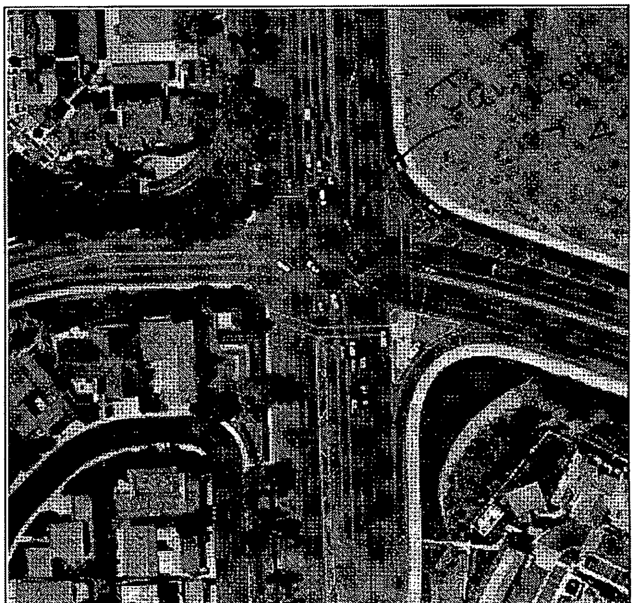

FIGURE 1 Pacific Coast Highway at Jamboree Road.

FIGURE 2 Pacific Coast Highway at Bayside Drive.

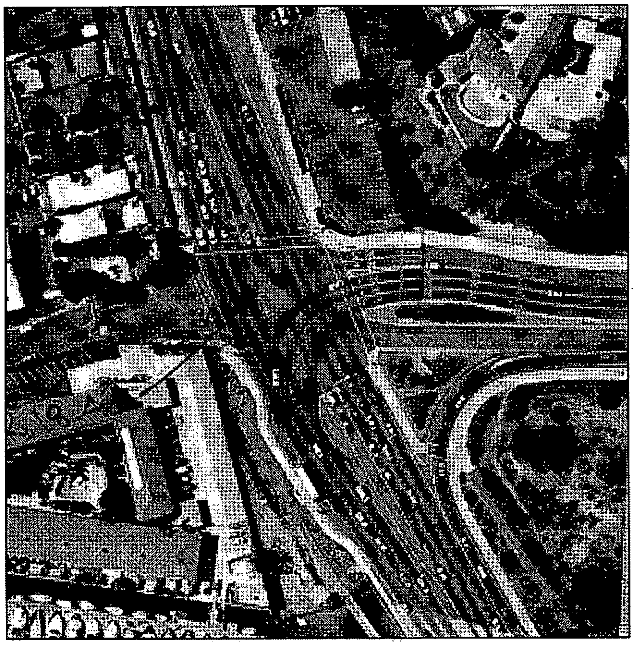

FIGURE 3 Pacific Coast Highway at Dover Drive.

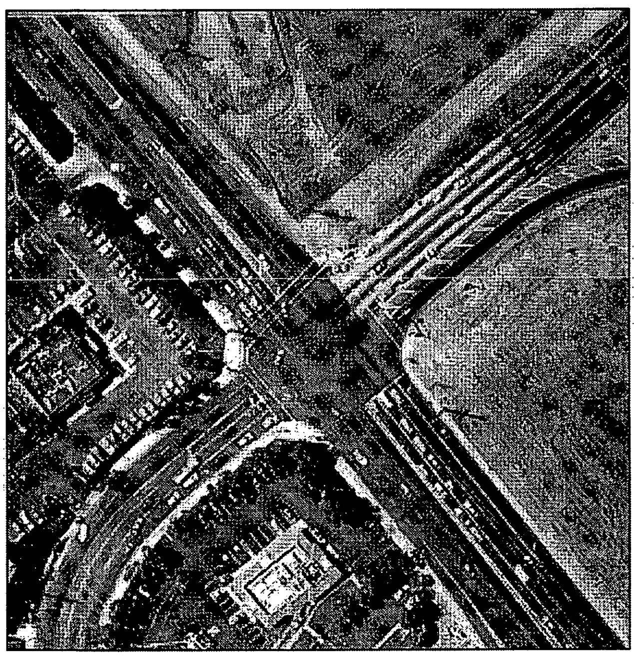

FIGURE 4 Southbound Interstate 5 at Lake Forest Road.

as the period starting at the onset of green and ending with the onset of the next green. Each cycle is divided into three time intervals: green, yellow, and red. Queues are discharged during the green and are accumulated during the yellow and red. Vehicles arriving during the red portion of a cycle are accumulated and then discharged during the succeeding cycle.

Some errors may be introduced during the data collection process. These errors generally result from unobserved, late, or miscoded events (e.g., the observer missed a vehicle arriving into the queue, pressed the button marking the onset of yellow, or pressed the wrong button upon the onset of red). During the construction of the event stream, an attempt is made to identify these types of errors. If the errors are identified, what appear to be good data are salvaged. If inconsistencies cannot be resolved, the data associated with the current cycle are not stored.

Statistical Methodology

The cycle-by-cycle data file constructed may be analyzed using any of a number of standard statistical packages. Two-tailed tests of statistical significance were evaluated using a 95 percent confidence interval.

RESULTS

Observed Vehicle and Cycle Summaries

Field observations were collected at five triple-left-turn sites during the morning, midday, and evening peak travel periods. Twenty-nine field surveys were performed.

A total of 34,898 vehicles over 4,742 lane cycles were observed at all triple-left-turn sites. A lane cycle is defined as the period associated with a traffic signal cycle for a single lane of traffic. For example, three lane cycles are observed at a triple-left-turn site for each traffic signal cycle, one lane cycle for each lane. However, there does not necessarily exist an exact 3:1 correspondence between the number of lane cycles and signal cycles. During periods of low traffic demand (e.g., just before or just after a peak period), one or more of the lanes of the triple left turns would not develop queues. In some cases (e.g., data collected during the first few days of observation) queues with fewer than two or three vehicles queued were arbitrarily omitted by the observer. In later field surveys the observer recorded events for all cycles observed, whether a vehicle was present or not; lane cycles without queued vehicles are designated as missing values for all tabulations. (Unless otherwise stated, use of the word "cycle" is understood to represent a "lane cycle" throughout the rest of the paper.)

Table 2 presents a tabulation of the cycle observations by lane (e.g., inner, outer, and middle) and by time of day (e.g., morning, midday, and evening, or AM, MD, and PM, respectively). Cycles are classified into time-of-day categories using the clock time at onset of the green: AM, before 10:00 a.m.; PM, after 3:00 p.m.; and MD, from 10:00 a.m. to 3:00 p.m. One site (PCH and Jamboree) provides data collected over all three periods, and one site (PCH and Dover) provides data collected over two periods (AM and MD); data at the remaining sites were collected during one period only.

Cycle observations are distributed equally across all three lanes of travel. For the inner and middle lanes, 1,583 and 1,587 valid lane cycles were observed. Cycles without queues were omitted from this sample. Differences in the number of observed cycles would suggest that vehicles would tend to queue first by using the inner and middle lanes and then by using the outer lane.

Table 3 categorizes the cycle observations by day of week. At one site (PCH and Jamboree) data were collected on all days of the week and during all times of day. Data at other sites are not as complete. One reason is that some sites did not exhibit any queueing at specific times of day (e.g., weekend mornings.) A limited project budget also contributed to this reduced coverage. However, all sites

TABLE 2 Cycle Summaries by Lane and Time of Day

Site	Total Vehicles Observed	Lane			Time-Of-Day			
		Inner	Outer	Middle	AM	MD	PM	Total
PCH/Dover	1219	56	49	55	68	92	.	160
I5/LakeF	2370	70	69	76	.	215	.	215
PDV/LosAl	4541	206	210	211	.	.	627	627
PCH/Jamb	26392	1226	1221	1222	1525	772	1372	3669
PCH/Baysi	376	25	23	23	.	.	71	71
All Sites	34898	1583	1572	1587	1593	1079	2070	4742

TABLE 3 Cycle Summaries by Day of Week

Site Name	Day Of Week							Total
	Sun	Mon	Tue	Wed	Thu	Fri	Sat	
PCH/Dover	92	.	.	68	.	.	.	160
I5/LakeF	.	.	132	.	83	.	.	215
PDV/LosAl	.	.	.	290	250	87	.	627
PCH/Jamb	246	618	541	670	752	752	90	3669
PCH/BaySi	.	.	71	.	.	.	.	71
All Sites	338	618	744	1028	1085	839	90	4742

TABLE 4 Queueing Characteristics

Site Name	Vehicles In Queue at Onset of Green												Total
	1	2	3	4	5	6	7	8	9	10	11	12+	
PCH/Dover	.	.	.	19	42	45	31	19	3	1	.	.	160
I5/LakeF	.	1	.	5	33	47	44	35	25	21	4	.	215
PDV/LosAl	35	73	85	89	113	74	56	37	24	14	8	19	627
PCH/Jamb	184	346	513	544	605	479	440	319	121	82	23	13	3669
PCH/BaySi		.	2	46	16	6	1	.	.	.	.		71
All Sites	219	420	600	703	809	651	572	410	173	118	35	32	4742

Mean: 5.16 Vehicles, Mode: 5 Vehicles, Median: 5 Vehicles, Maximum: 17 Vehicles

include at least one period of data collected on a midweek day (e.g., Tuesday, Wednesday, or Thursday.)

Table 4 presents a tabulation of the frequency of lane cycles categorized by numbers of vehicles in the queue. Lane cycles are assigned categories on the basis of number of vehicles that were observed in the standing queue at the onset of green. Each entry in the table represents the number of cycles with the observed queue size. Across all sites, the mode queue size is 5 vehicles, the mean queue size is 5.16 vehicles, the median queue size is 5 vehicles, and the maximum queue size is 17 vehicles.

Saturation Flow Rates

Using the HCM definition, saturation flow rate is the flow in vehicles per hour that could be accommodated by a lane group assuming that the green phase was always available. In general, the maximum flow rate (e.g., saturation flow) may be observed during queue discharge from the stop bar. The HCM defines the period of saturation flow as beginning when the rear axle of the fourth vehicle in the queue crosses the reference point and ending when the rear axle of the last queued vehicle at the beginning of the green crosses the same reference point. For this study of triple left turns, given that the time at discharge was recorded for each vehicle observed during each cycle and that the backs of queues were tracked, alternative saturation flow rate computations using the fields observations may be evaluated.

Table 5 presents a summary of saturation flow rates at each site computed using alternative calculation techniques. Each calculation method is associated with a designation "Drop X," where X represents the number of vehicles dropped from the front of the queue when determining the mean discharge headway. For example, the mean discharge headways for scenario Drop 4 are computed by subtracting the discharge time recorded for the fourth vehicle from the discharge time recorded for the last vehicle to clear and dividing this total headway by the number of gaps between the fourth through the last vehicles—specifically, $n - 4$ where n represents the total vehicles in the queue at the onset of green.

The "None" scenario represents the saturation flow rates computed using all vehicles in the queue and the total discharge time

TABLE 5 Alternative Saturation Flow Rate Computations

Site Name	Vehicles Dropped from Front of Queue						
	None	Drop 1	Drop 2	Drop 3	Drop 4*	Drop 5	Drop 6
PCH/Dover	1673	1754	1835	1871	1939	1981	2077
I5/LakeF	1651	1703	1775	1824	1877	1923	1942
PDV/LosAl	1963	1831	1874	1951	1989	2020	2083
PCH/Jamb	1797	1805	1829	1868	1921	1952	2023
PCH/Baysi	1539	1632	1739	1823	1997	2157	1477
All Sites	1804	1799	1831	1875	1928	1959	2024

*Drop 4 corresponds with the suggested HCM calculation method.

from the onset of green. These values are typically lower than the other values, which may be attributed to start-up loss times associated with vehicles near the beginning of the queue. As one would expect, as these initial vehicles are discarded from the calculation, the saturation flow rate increases. The Drop 4 scenario corresponds with the HCM definition of saturation flow rate period, specifically, the period measured from the fourth through the last vehicle in the queue at onset of green.

As additional vehicles are ignored from the calculation, saturation flow rates averaged over all sites increase from about 1,800 to 2,030 vehicles per hour of green per lane (vphgpl). The HCM-suggested method (e.g., Drop 4) results in an observed saturation flow rate of about 1,930 vphgpl over all sites.

Using the Drop 4 value in Table 5, saturation flow rate values at each site range from 1,880 vphgpl at Site 2 (the freeway off-ramp) to 2,000 vphgpl at Site 5. However, an analysis of variance (Table 6) reveals that these means are not significantly different from the overall group mean of 1,930 vphgpl.

The observed value of 1,930 vphgpl is much larger than the "ideal" saturation flow rate of 1,800 vphgpl suggested by the HCM. As such, computation of a left-turn adjustment factor using only these data is not possible. Additional data collected at the same sites for single and double left turns and for through movements would allow explicit identification of this adjustment factor. However, by accepting several conservative assumptions one may estimate the value of the ideal saturation flow rate at these sites. Assume a lane width of 11 ft, a heavy truck percentage of 2 percent, and the same left-turn adjustment factor applied to exclusive double-left-turn lanes. The HCM saturation flow rate equation (2, Equation 9–8) may be written as

$$s_{lt3} = s_o * f_w * f_{HV} * f_{LT}$$

where

s_{lt3} = saturation flow rate for triple left turns under prevailing conditions,
s_o = ideal saturation flow rate,
f_w = lane width adjustment factor (0.97),
f_{HV} = heavy vehicle adjustment factor (0.99), and
f_{LT} = left-turn lane adjustment factor (0.92).

Under these assumptions, and using the factors obtained from the appropriate tables of the HCM, the ideal saturation flow rate for these five sites would be approximately 2,180 vphgpl.

Variations in Saturation Flow

The influences of site, lane, time of day, and day of week on the observed saturation flow rates of triple left turns were investigated. Table 6 gives a summary of the results of this series of analysis of variance tests. The table presents the explanatory variable under study, the sample size used in the calculation of the F-scores, the degrees of freedom (e.g., categories in explanatory variable less 1), the F-score and a level of significance determined using the F-score, and the degrees of freedom within the sample. All tests of significance are evaluated at the .05 level.

TABLE 6 Saturation Flow Rate Analysis of Variance

Primary Effect	Sample Size	Deg.Of Freedom	F-Score	Significance of F	Reject Null?
Site Code (1-5)	2784	4	2.063	0.083	No
Lane (Inner,Outer,Middle)	2784	2	4.113	0.016	Yes
Time of Day (AM,MD,PM)	2874	2	15.344	0.000	Yes
Weekdays Only (Monday-Friday)	2441	4	1.964	0.097	No
Weekday/Weekend (M-F vs. Sat-Sun)	2784	1	21.769	0.000	Yes
Observer (Two alternates)	2784	1	0.142	0.706	No

All Secondary Interaction Effects (e.g., Lane with TOD, etc.) are not significant at 0.05 Level.

Lane Utilization

Saturation flow rates categorized by lane ranged from 1,890 vphgpl in the outer lane to about 1,950 for the inner and middle lanes (Table 7). These differences are statistically significant and suggest that the outer lane of a triple-left-turn group will exhibit a reduced capacity from the inner and middle lanes. A lane utilization factor based on the mean value of saturation flow for the entire triple-left-turn lane group may be proposed. Let

$$s_{lg} = f_u * s_{lt3}$$

where s_{lg} is the saturation flow rate of the lane group within a triple left turn and f_u is the lane utilization factor. Substituting values of 1,930 vphgpl for the saturation flow rate under prevailing conditions and 1,950 and 1,890 vphgpl for inner/middle and outer lane groups, respectively, lane utilization factors of 1.01 for the inner/middle lane group and 0.98 for the outer lane group may be computed.

One might attribute the lower saturation flow rate of the outer lane to the fact that some sites share left and through movements from this lane. However, an examination of Site 4, which consists of three exclusive left-turn lanes, each with similar lane widths, reveals the same distribution of saturation flow rates between lanes.

Time-of-Day Variations

The data suggest statistically significant variations in saturation flow rates by time of day (e.g., morning, midday, and evening). Values of 1,990, 1,860, and 1,920 vphgpl were observed for the morning, midday, and evening periods (Table 7). A time-of-day adjustment factor may be proposed as

$$s_t = f_t * s_{lt3}$$

where s_t is the saturation flow rate for a specific time-of-day period and f_t is the adjustment factor. Corresponding adjustment factor values of 1.03, 0.96, and 0.99 may be computed for the morning, midday, and evening periods.

These results support the hypothesis that the population of drivers may be classified into two subpopulations: commuting and noncommuting, with each subpopulation exhibiting significantly different driving characteristics (e.g., acceptable discharge headways as measured by saturation flow rate). Different times of day would be composed of different proportions of commuting and noncommuting drivers. Commuting drivers concentrate their activity during the morning and evening peaks; they tend to be more aggressive and accept smaller headways. Noncommuting drivers (e.g., shoppers and tourists) concentrate their activity in the midday and evening periods; they tend to be less aggressive and accept larger headways. The a.m. peak period (with an observed mean saturation flow rate of 1,990 vphgpl) consists of primarily commuter drivers, the midday period (1,860 vphgpl) consists of primarily noncommuting drivers, and the p.m. peak consists of a mix of commuting and noncommuting drivers and exhibits a saturation flow rate of 1,920 vphgpl, approximately midway between the morning and midday periods.

Day-of-Week Variations

The data also suggest significant variations in saturation flow rates observed on weekends (Saturday and Sunday) versus weekdays (Monday through Friday). The weekday saturation flow rates over all sites was observed to be 1,940 vphgpl, whereas the group mean for weekends was observed to be 1,810 vphgpl. Differences in saturation flow rates between individual weekdays and the overall weekday group mean were not statistically significant. A day-of-week utilization factor may be proposed:

$$s_d = f_d * s_{lt3}$$

where s_d is the saturation flow rate for a particular day of week (e.g., either weekday or weekend) and f_d is the day-of-week adjustment factor. Adjustment factor values of 1.01 and 0.94 are calculated for weekdays and weekend days, respectively.

Other Variations

As a consistency check, the saturation flow rates measured by two different observers were compared. Results suggest that the saturation flow rates measured by each observer are not significantly different from the population mean estimated for the entire sample.

All secondary interaction effects (e.g., lane utilization by time of day, or time of day by day of week) were not significant at the 0.05 level. The lack of significance between interactions of these variables simplifies implementation of the proposed adjustment factors.

TABLE 7 Observed Saturation Flow Rates by Lane and Time of Day

	Lane			Time-Of-Day			
Site	**Inner**	**Outer**	**Middle**	**AM**	**MD**	**PM**	**Overall**
PCH/Dover	1938	1894	1979	1977	1908	.	1939
I5/LakeF	1888	1913	1834	.	1877	.	1877
PDV/LosAl	1954	2005	1994	.	.	1989	1989
PCH/Jambo	1948	1868	1954	1992	1838	1888	1921
PCH/Baysi	2209	1942	1655	.	.	1997	1997
All Sites	1946	1891	1950	1991	1856	1921	1928

Values computed using *n-4* vehicles in queue (HCM suggested)

TABLE 8 Signal Timing Characteristics of Triple Left Turns

Site Name	Flow (vphgpl)	Sat Flow (vphgpl)	Green (secs)	Yellow (secs)	Cycle (secs)	Split (%)	Busy (%)
PCH/Dover	274	1939	21	3.3	100	21	65
I5/LakeF	382	1877	29	3.8	104	28	56
PDV/LosAl	298	1989	21	4.1	87	25	46
PCH/Jamb	253	1921	18	3.0	103	17	59
PCH/BaySi	188	1997	15	2.8	103	15	71
All Sites	265	1928	19	3.2	101	18	57

It should be noted, however, that during the weekend mornings, significant queues did not form at any of the sites. As a result one should restrict application of the time-of-day factor to weekdays only.

Signal Timing Characteristics

By using the discharge times of vehicles in the queue in conjunction with the observed times of onset of the green, yellow, and red timing intervals, summary statistics of the signal timing characteristics of the observed triple left turns may be compiled.

Table 8 presents a summary of the signal timing characteristics for triple left turns. For each site the observed flow, saturation flow rate, observed green time, observed yellow time, observed cycle time, observed split time, and observed busy time are presented. Values of flow represent averaged per-lane volumes. Saturation flow rates are measured in vehicles per hour of green per lane and represent the per-lane saturation flow rates determined earlier. Values of green, yellow, and cycle represent the mean observed times of the respective signal timing intervals. For the sample of triple left turns, average interval lengths of 19, 3, and 101 sec were observed for the green, yellow, and cycle times, respectively. The observed split time (in percentage) represents the ratio of green time to cycle time.

The busy time represents the proportion of the green time spent discharging the queue. It is very similar to the degree of saturation, the main difference being that the degree of saturation is computed using all vehicles discharged during the cycle, whereas the busy time is computed using only the vehicles discharging from the queue. In general, the busy time will be less than the degree of saturation value, as some vehicles pass through the intersection after the queue has completely discharged. In the limiting cases, when all vehicles arrive on the red and discharge from the queue, the busy time will equal the degree of saturation. Conversely, when all vehicles arrive on green and no queue was accumulated during the preceding cycle, the busy time will be 0.

For the sample of triple left turns observed, the triple-left-turning movement services 795 vehicles per hour (vph) while receiving only on average 19 percent of the total cycle time. This demonstrates the potential capacity improvement at intersections without adversely affecting the length of the cycle. A busy time of less than 60 percent suggests the possibility of further reducing the split time without hurting the throughput of the triple left turn.

SUMMARY AND CONCLUSIONS

A study of the characteristics of triple left turns was performed. Five sites in Orange County, California, were identified. Manual saturation flow rates at each site were studied using electronic counter boards. Queue discharge times were collected for all vehicles by lane and by cycle.

Across the five sites, a sample consisting of 4,742 lane cycles and 34,898 vehicles was compiled for analysis. On average, these triple left turns supported flows of 795 vph, received a 19 percent split of the total cycle time, and spent 57 percent of that split time servicing the queue. The average saturation flow rate observed over all sites under prevailing conditions was approximately 1,930 vphgpl.

Variations in the saturation flow rate observed at the triple-left-turn sites were also investigated. Results reveal no significant differences between saturation flow rates by site, between weekdays, or by observer. Significant differences were observed between lanes (e.g., inner and middle versus outer), time of day (morning, midday, and afternoon), and time of week (weekday versus weekend). Appropriate saturation flow rate adjustment factors have been suggested.

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REFERENCES

1. Teply, S., and A. Jones. Saturation Flow: Do We Speak the Same Language? In *Transportation Research Record 1320*, TRB, National Research Council, Washington, D.C., 1991.
2. *Special Report 209: Highway Capacity Manual*. TRB, National Research Council, Washington, D.C., 1985.
3. *IMC-IV User's Manual*. Jamar Sales Company, Ivyland, Pa., 1989.

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