Field Validation of Intersection Capacity Factors

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Presented are the results of a series of saturation flow surveys conducted throughout the United States at signalized intersections. The purpose of this research was to verify the saturation flow rates and traffic volume adjustment factors used in various capacity analysis procedures by collecting a relatively extensive data base. Saturation flow headways for more than 20,000 observations were collected for a series of 12 geometric, traffic characteristic, and environmental factors and compared with baseline saturation flow headways for various signal cycle length and phase combinations. Vehicle blockage and lane distribution surveys were conducted for 19,000 additional observations. Based on the results of these surveys, a series of modified adjustment factors is suggested to allow the analyst to determine modified saturation flow rates when calculating signalized intersection capacity.

The signalized intersection chapter of the 1985 *Highway Capacity Manual* (HCM) *(1)* contains a capacity analysis procedure that is based on vehicle delay (stopped delay per vehicle) as the principal measure of effectiveness for levels of service. A critical component used in this procedure is the determination of basic saturation flow rates and adjustment factors used to modify these flow rates.

The purpose of this research effort was to collect an extensive data base that could be used to verify the adjustment factors and the basic saturation flow rate values to improve the reliability of the signalized intersection capacity analysis technique. In addition, these measured saturation flow rates and adjustment factors can be used in other capacity analysis procedures. The saturation flow data collection procedure reported in this paper can be duplicated at other locations.

Results of this research are presented in two sections:

1. Saturation flow rates are provided for the range of common signal cycle lengths and phases, taking into account lost times surveyed.
2. Critical lane volume adjustment factors are presented based on extensive field surveys.

INTERSECTION SATURATION FLOW RATES

Intersection saturation flow rates can be determined for a given level of service based on observed vehicle headways and a design cycle length, as described in Equation 1:

$$SV_E = (3{,}600/h)\ (C - nL)/C \qquad (1)$$

where

SV_E = maximum sum of critical lane volumes,
h = average vehicle headway,
C = cycle length,
n = number of phases per cycle, and
L = lost time per phase.

Although this equation takes into account each of the variables that influence the number of vehicles that can be processed by an intersection *(2)*, and this relationship is consistent with the values measured empirically in the Australian signalized intersection capacity method, there are a number of similar formulas that yield reasonably consistent results. [See Table 1, *Transportation Research Circular 212* (*3*, p.7).]

Adjusted critical lane volumes for various cycle lengths and number of phases per cycle can be determined from average vehicle headways and lost times per phase for through lanes.

Typical Saturation Flow Rates

In studies conducted between 1947 and 1979, through-lane saturation flow rates of between 1,500 and 1,800 vehicles per hour of green (vphg) have been reported *(4)*. In these studies, various vehicles in a queue were identified as representing the first vehicle not incurring a significant amount of lost time. The most extensive data base recently reported in the United States was collected in Kentucky in medium and smaller sized communities. An average saturation flow rate of 1,650 vehicles per hour (vph) was found for lanes with widths between 10 and 15 ft and with approach grades between −3 percent and +3 percent *(5)*.

In *Transportation Research Circular 212*, a value of 1,800 passenger cars per hour of green for optimum roadway conditions is recommended as a base value for saturation flows for the fifth vehicle in queue and beyond *(3)*. NCHRP research *(6)* for the 1985 HCM included a review of 2,926 vehicles following the fourth vehicle in queue (contained in film from a 1975 FHWA delay study). A saturation flow rate of 1,827 vphg was measured.

Survey Procedures

A series of surveys of vehicles in saturation flow was conducted throughout the United States at signalized intersection approaches that contained baseline geometric conditions—those approaches not influenced by geometric factors that have been found to significantly reduce saturation flow rates. A description of these conditions is given in Table 1.

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TABLE 1 BASELINE INTERSECTION APPROACH CONDITIONS

Characteristic	Condition or Value
Lane width	12 ft
Percent heavy vehicles	0 percent (preferred); less than 2 percent (required)
Approach grade	0 percent
Curb parking condition	No curb parking allowed, no illegal curb parking present
Local bus stops	No nearside or farside bus stop activity or signs designating the potential for buses to stop
Area type	Suburban area or outlying commercial; not central business district or residential area
Permitted movements	Straight (or through) movements only; no turn movements permitted in exclusive or optional lane
Metro area size	Greater than 1,000,000 population in metropolitan area
One-way or two-way operation	Two-way operation

Source: Barton-Aschman Associates, Inc.

The data collection techniques used in these surveys were identical to the procedures described in Appendix IV of the 1985 HCM (*1*, Chapter 9). As described in those procedures, the last vehicle stopped in the queue before the onset of green is noted for the signal phase surveyed. At the onset of green, a stopwatch is started. When the rear wheels of the fourth vehicle in queue cross the stop line, saturation flow is considered to begin and the stopwatch time is noted. Saturation flow ends when the rear wheels of the last vehicle (in the queue before the onset of green) cross the stop line. The average saturation flow headway for that signal phase is calculated by dividing the time elapsed for processing n – 4 vehicles by the number of saturation flow vehicles processed. When divided into 3,600, the saturation flow rate for this phase is determined. Only those vehicles crossing the stop line after the fourth queued vehicle were considered to be in saturation flow conditions. In addition, only those vehicles already waiting in a queue at the onset of the green phase were surveyed. Measurements were taken by cycle and by lane at each baseline location.

A total of 3,687 saturation flow vehicles were surveyed at 7 intersection approaches containing baseline conditions in the Chicago, Houston, and Los Angeles metropolitan areas, which was a subset of approximately 20,000 saturation flow headways surveyed. The saturation flow headway for this sample of baseline conditions was 1.92 sec, which is equivalent to a saturation flow rate of 1,875 vphg. Thus, for suburban intersection approaches in large metropolitan areas, saturation flow rates higher than typically reported in the literature can be achieved when good geometric conditions are present. Field surveys at representative intersections (with baseline geometric conditions) should be conducted whenever possible to identify local variations in this value.

Lost Time per Phase

During a green and amber signal phase, there are two periods during which saturation flow headways are not achieved: at the beginning of the phase (startup lost time) and at the end of the phase (clearance lost time).

In an analysis of 1,428 headways for vehicles positioned first, second, or third in queue in Kentucky, the average startup lost time was found to be 1.40 sec *(7)*. The largest community surveyed, Louisville, had an average startup lost time of 1.01 sec.

The average startup lost time per phase was determined for the 3,687 vehicles surveyed for the research discussed in this paper on the baseline condition intersection approaches in the Chicago, Houston, and Los Angeles metropolitan areas. The total time elapsed between the onset of green and the time at which the rear wheels of the fourth vehicle in queue crossed the stop line was measured. A value of four times the average saturation flow headway (1.92 sec) was subtracted from the travel time for the first four vehicles to determine the startup lost time for each phase. The average startup lost time thus calculated was 1.31 sec.

The lost time experienced at the end of each phase (clearance lost time) can vary based on the length of the amber phase (clearance phase), width of the intersection, and vehicle approach speed. Most clearance phase lengths range between 3 and 5 sec. However, the clearance lost time rarely is as great as the length of the amber phase. Typically, the clearance loss time (before the onset of green on the cross street) is about one-half of the length of the amber. Clearance lost times at 334 loaded cycles in Kentucky averaged 1.67 sec *(7)*. The length of the signal cycle and the change interval (yellow plus all red) had a significant impact on the values observed.

Observations have indicated that the portion of the change interval devoted to an all-red phase is virtually all lost time. Total lost time per phase includes the sum of the average startup lost time and clearance lost time. The total was set as a percent of cycle length. The resulting values of total lost time per phase were used to calculate lane volumes as follows:

- 60-sec cycle—5 percent of cycle—3.0 sec;
- 75-sec cycle—4 percent of cycle—3.0 sec;
- 90-sec cycle—4 percent of cycle—3.5 sec;
- 105-sec cycle—3⅓ percent of cycle—3.5 sec; and
- 120-sec cycle—3 percent of cycle—3.5 sec.

These values of 3.0 and 3.5 sec were selected based on (*a*) the measured startup and clearance lost times reported in the surveys described previously, and (*b*) the recognition that the length of signal cycle does affect the observed value of lost time.

Based on the saturation flow headway and lost time values just discussed, a series of adjusted critical lane volumes was developed. These values are given in Table 2. It can be observed as the cycle length increases, a slight increase in critical lane volume occurs. Although this may be a valid concept in considering intersection design, the increase in average vehicle delay that is encountered when cycle length is increased is ignored. If the volume on a critical approach is not of sufficient length to fully load a long cycle, the phase will not be used effectively and the flow rate will decrease. Short cycle lengths usually result in lower average vehicle delay for a given set of intersection conditions. Thus, in the intersection design procedure, the shortest cycle length that can accommodate the traffic demands should therefore be considered so that delay is minimized.

As indicated by the data in Table 2, a two-phase signal with a 120-sec cycle length can accommodate 5 percent more critical lane vehicles than would a two-phase signal with 60-sec cycle length. Likewise, increasing the number of phases for a given cycle length results in a reduction in critical lane volume. For example, at a 90-sec cycle length, a four-phase signal can process about 9 percent fewer critical lane vehicles than can a two-phase signal. The critical lane values given in Table 2 should not be extrapolated for cycle lengths of greater than 120 sec because of the increasing likelihood that vehicle platoons will disperse, thus reducing the achievable flow rate.

The adjusted critical lane volumes given in Table 2 were compared with those proposed in *Transportation Research Circular 212* for the signalized intersection operations and design application *(3)*. The values given in *Circular 212* have a threshold value of 1,800 passenger cars per hour for a two-phase signal. Given the 2-sec average vehicle headway assumed for saturation flow conditions in that procedure, the saturation flow values in *Circular 212* do not take into account the effect of lost time on saturation flow. The three- and four-phase values in *Circular 212* reflected 4 percent and 8 percent reductions in critical volume, respectively. These reductions are consistent with the reductions in critical lane volumes given in Table 2 in this paper for a 90-sec cycle length.

ADJUSTMENT FACTORS

The adjustment factors used to modify saturation flow rates vary significantly by source. Many of the factors from other sources have been estimated, without the benefit of research. Other factors have been based on a very small sample of data. Still other factors appear to rely on the 1965 HCM values *(8)*, which were derived from surveys conducted in the late 1950s. A major portion of the research effort documented in this paper involved the review of adjustment factors proposed in recent literature. The results from this new data collection effort were then used to verify the reliability of the adjustment factors contained in *Transportation Research Circular 212 (3)* and the 1985 HCM *(1)*.

The 12 factors that are most commonly considered to affect saturation flow rates were analyzed as a part of the research effort. These factors are as follows:

1. Lane width,
2. Heavy vehicles,
3. Vertical grade on approach,
4. Curb parking,
5. Local bus stop activity,
6. Area type,
7. Through-lane utilization,
8. Turning movements (single turn lanes),
9. Dual turn lanes,
10. Pedestrian conflict,
11. Metropolitan area size, and
12. One-way or two-way operation.

All of the surveys except for numbers 5, 7, 9, and 10 used the saturation flow technique as described in Appendix IV of the 1985 HCM *(1*, Chapter 9), and described in a previous section of this paper. The survey of Factor 5 and a portion of the survey of Factor 4 (Parking and Unparking Maneuvers) were conducted by observing moving lane blockage time experienced during green signal phases. The surveys of Factors 7 and 9 were conducted by recording approach vehicle volumes distributed by lane. Values for Factor 10 (and permissive left-turn values for Factor 8) were calculated based on surveys reported in other sources.

Surveys were conducted in 14 metropolitan areas throughout the United States at a total of 98 signalized intersection approaches. More than 20,000 vehicles operating in saturation flow conditions were surveyed to determine average vehicle headways for the various factors (Factors 1, 2, 3, 4, 6, 8, 11, and 12). To analyze each individual factor, intersections were

TABLE 2 CRITICAL LANE VOLUMES

Cycle Length (sec)	Maximum Sum of Critical Lane Volumes[a] (passenger cars/hr)		
	Two-Phase Signal	Three-Phase Signal	Four-Phase Signal
60	1,690	1,590	1,500
75	1,720	1,650	1,570
90	1,730	1,660	1,580
105	1,750	1,690	1,630
120	1,770	1,710	1,660

Source: Barton–Aschman Associates, Inc.

[a]Sum of critical lane volumes: $CV = (3{,}600/C)\ (C - nL/h) = (3{,}600/h)\ (C - nL/C) = (3{,}600/h)\ (1 - nL/C)$.

where C = cycle length; n = number of phases per cycle; L = lost time per phase—varies between 3.0 and 3.5 sec; h = saturation flow headway = 1.92 sec; and $CV = (1{,}875)\ (1 - nL/C)$.

selected that contained all of the baseline geometric conditions, except for the factor being tested. The results from the saturation flow surveys were then compared with the baseline saturation flow values to isolate the effect of the factor in question. More than 19,000 additional vehicles were counted to determine lane distribution rates, parking and unparking maneuvers, and bus stop maneuver influence (Factors 4, 5, 7, and 9).

Each of the 12 factors is discussed separately in the following subsections. Adjustment factors contained in *Transportation Research Circular 212 (3)* and the 1985 HCM *(1)* are reviewed and compared with the results of the data surveys.

Lane Width

The 1985 HCM adjustment factors for lane width are graduated in foot-by-foot increments between 8 and 15 ft *(1)*, as given in Table 3.

TABLE 3 ADJUSTMENT FACTORS FOR LANE WIDTH AS GIVEN IN THE 1985 HCM

Lane Width (ft)	f_{LW}
8	0.87
9	0.90
10	0.93
11	0.97
12	1.00
13	1.03
14	1.07
15	1.10
16	Use 2 lanes

The *Transportation Research Circular 212* review of lane width adjustment factors from various sources concludes that, "Lane widths in the 10- to 13-foot range have little effect on saturation flow or capacity" *(3)*. Thus, a step function is recommended whereby passenger car volumes are increased 10 percent for lane widths of between 8.0 and 9.9 ft and decreased by 10 percent for lane widths of between 13.0 and 15.9 ft (compared with a fixed saturation flow rate). In the Australian capacity procedures, capacity adjustments are made for lane widths outside the range of 10.0 to 12.0 ft, also using step functions.

As a part of the national surveys conducted for the research documented in this paper, saturation flow rates were measured on 11 approaches with lane widths varying between 8.5 and 9.5 ft. The sample size was 2,733 saturation flow vehicles. Four approaches with lane widths varying between 13.0 and 15.5 ft were surveyed, with a sample size of 1,568 saturation flow vehicles. All baseline conditions except for lane width were held constant at these locations. The survey results were then compared with those of the baseline condition surveys (with a sample size of 3,687 saturation flow vehicles). The narrower lane widths demonstrated saturation flow rates between 2 and 5 percent less than did those in the baseline surveys, while the wider lane widths demonstrated saturation flow rates 5 percent greater than did those in the baseline surveys. Thus, the following factors are proposed for lane-width adjustments based on the survey findings:

- 8 to 8.9 ft—0.95;
- 9 to 9.9 ft—0.98;
- 10 to 12.9 ft—1.00; and
- 13 to 15.9 ft—1.05.

Heavy Vehicles

A heavy vehicle is defined as any truck or bus having six or more tires on the pavement. All vans and light-duty trucks containing only four tires are excluded from this definition of heavy vehicles. Data used as input for the 1985 HCM indicated that the headway between preceding passenger vehicles and trucks averaged 2.06 sec (108 vehicle samples) *(6)*. The headway between trucks and following passenger vehicles averaged 2.61 sec (105 vehicle samples). Average vehicle headways between all vehicles preceding or following a bus were 3.10 sec (30 vehicle samples). This implies a heavy vehicle passenger car equivalent of between 1.3 and 1.6. In *Transportation Research Circular 212*, it is recommended that a passenger car equivalent value of 2.0 for all heavy vehicles (trucks or through buses not stopping at the intersection) be used to convert from vehicles to passenger cars. Surveys conducted by Carstens in 1971 for trucks in through lanes resulted in a heavy vehicle-passenger car equivalent of 1.6 *(9)*.

The surveys conducted for the research documented in this paper at intersections with significant volumes of heavy vehicles on the approaches indicated that heavy trucks had a significant influence on increasing vehicle headways. First, headways of automobiles following automobiles were surveyed to confirm that their saturation flow rates were comparable to baseline conditions (1.92-sec headways). Then, surveys were conducted to determine average headway values for three conditions: trucks following automobiles (68 samples)—5.16 sec; automobiles following trucks (64 samples)—2.22 sec; trucks following trucks (34 samples)—3.76 sec. These data suggest a passenger car equivalent of 1.92, which results in the proposed adjustment factors for heavy vehicles given in Table 4.

Vertical Grade on Approach

The headway data collected by intersection approach grade (percent) during the NCHRP study leading to the preparation of the 1985 HCM generally showed reductions in saturation flow rate for both upgrades and downgrades *(6)*. The reductions found in the NCHRP study for upgrades were as follows: 31 percent for 3 percent upgrades (55 vehicles sampled), 4.6 percent for upgrades of between 4 and 5 percent (38 vehicles sampled), and 23.9 percent for upgrades of between 6 and 7 percent (70 vehicles sampled). The surveys excluded heavy vehicles. The larger difference in saturation flow rate for 3 percent upgrades compared with that of steeper upgrades was noted. A reduction was also generally found to occur in saturation flow rates for downgrades: a 9.6 percent decrease in flow rates for downgrades of 3 percent (234 vehicles sampled), a slight (0.5 percent) increase in flow rates for downgrades of

TABLE 4 PROPOSED ADJUSTMENT FACTORS FOR HEAVY VEHICLES

Percent Heavy Vehicles	f_{HV}
0	1.00
2	0.98
4	0.96
6	0.95
8	0.93
10	0.92
20	0.84
30	0.78

between 4 and 5 percent (243 vehicles sampled), and a decrease of 10.2 percent in flow rates for downgrades of between 6 and 7 percent (24 vehicles sampled). An increase in flow rates for downgrades and a decrease in flow rates for upgrades are recommended in the 1985 HCM *(1)*. The suggested 1985 HCM factors are given in Table 5.

TABLE 5 SUGGESTED 1985 HCM FACTORS FOR VERTICAL APPROACH ON GRADE

Percent Grade	f_G
−6	1.03
−4	1.02
−2	1.01
0	1.00
+2	0.99
+4	0.98
+6	0.97

A sample of 1,592 saturation flow vehicles was surveyed on approaches with upgrades of between 4 and 6 percent as a part of the field surveys for the research documented in this paper. On downgrades of between 4 and 6 percent, a total of 1,697 saturation flow vehicles were surveyed. These surveys showed reductions in flow rates of 4 percent for downgrades and 10 percent for upgrades on intersection approaches. Observations at these intersections suggested that drivers on relatively steep downgrades proceeded through the intersections in a manner that was somewhat more tentative than that used on intersection approaches with no discernable grade. This resulted in an increase in headway between vehicles and a resulting decrease in saturation flow rates. On the basis of the results of these surveys, the following factors are proposed for intersection approach grades:

- 5 to 6 percent downgrade—0.96;
- 3 to 4 percent downgrade—0.98;
- 2 percent downgrade to 2 percent upgrade—1.00;
- 3 to 4 percent upgrade—0.95; and
- 5 to 6 percent upgrade—0.90.

Curb Parking

Research for the 1985 HCM used FHWA films taken in 1975 of vehicle parking movements *(6)*. It was indicated in this research that saturation flow rates were 9.2 percent lower in lanes adjacent to curb parking than in similar lanes not adjacent to curb parking (229 headways surveyed). In *Transportation Research Circular 212*, it is stated that, "Most North American techniques do not explicitly consider a reduction in capacity due to parking, if the parking ends 250 feet before the intersection stops. . ." *(3)*.

A sample of 1,811 saturation flow vehicles was collected as a part of the field survey for the research documented in this paper. The results of this survey indicated that saturation flow rates were reduced by 11 percent for vehicles traveling in lanes adjacent to curb parking (where parked vehicles exist between the stopline and 250 ft behind the stopline). A second series of surveys was conducted for parking and unparking maneuvers to determine the amount of time that the adjacent moving lane was blocked during these maneuvers. The average maneuver blocked the adjacent lane for about 7 sec (178 samples). Assuming that all parking or unparking maneuvers occur during the green signal phase on the approach being analyzed (because standing vehicles during the red phase would block the ability of a vehicle to enter or leave a parking space), the saturation flow time available for vehicles on an approach with one moving lane would be reduced by 3.9 percent for each 10 parking or unparking maneuvers per hour within 250 ft of the stopline. On this basis, the factors given in Table 6 are proposed for approaches with curb parking activity. These factors are similar to, but not exactly the same as, those contained in the 1985 HCM.

Local Bus Stop Activity

Research for the 1985 HCM included the review of 12 buses that stopped at traffic signals, blocking traffic from entering the intersection during the green phase *(6)*. The average length of

TABLE 6 FACTORS PROPOSED FOR CURB PARKING ACTIVITY

No. of Lanes on Approach	No Parking	No. of Parking Maneuvers/Hr				
		0	10	20	30	40
1	1.00	0.89	0.86	0.82	0.79	0.75
2	1.00	0.94	0.93	0.91	0.89	0.87
3	1.00	0.97	0.96	0.95	0.94	0.93

time for each bus to load and unload passengers was 20.7 sec. The average time that vehicles were blocked during a green phase was 14.0 sec. There were 2.4 vehicles blocked on average while a bus was entering or leaving the bus stop. Observations of the 12 buses were used to calibrate bus blockage adjustment factors for the 1985 HCM. An average passenger car equivalent value of 5.0 was suggested in *Transportation Research Circular 212* for local buses *(3)*. This is equal to an average headway of about 10 sec per bus (including those that stop at the intersection and those that do not stop).

The surveys for the research documented in this paper included observations of 262 buses stopping on intersection approaches. The average length of time that these buses blocked the adjacent lane during a green phase was 9.1 sec (compared with 14.0 sec from the 1985 HCM surveys). This resulted in a reduction of saturation flow of about 2.5 percent for each of 10 buses per hour stopping on a one-lane approach. The resulting bus blockage factors based on this data base, to be proposed, are given in Table 7.

TABLE 7 PROPOSED RESULTING BUS BLOCKAGE FACTORS

No. of Lanes on Approach	No. of Buses Stopping/Hr 0	10	20	30	40
1	1.00	0.97	0.95	0.92	0.90
2	1.00	0.99	0.97	0.96	0.95
3	1.00	0.99	0.99	0.98	0.97

Area Type

The influence of the surrounding environment on service flow rates is commonly taken into account by defining the intersection location within a metropolitan area as residential (RES), outlying commercial district (OCD), or central business district (CBD). Research for the 1985 HCM identified vehicle headway values of 2.11 and 2.09 sec for OCD and RES locations, respectively *(6)*. The sample sizes for these two area type surveys were 781 and 187 vehicles, respectively. Headway values in CBD locations were found to average 2.35 sec (500 vehicles sampled). As a result, an adjustment factor of 0.9 for intersections within CBDs is suggested in the 1985 HCM *(1)*.

The surveys for the research documented in this paper included sample sizes of 2,687, 709, and 883 saturation flow vehicles for OCD, RES, and CBD locations, respectively. An analysis of the survey results indicated that the saturation flow rates for vehicles in RES areas are 1 percent greater than the saturation flow rates in OCD areas, and that the saturation flow rates in CBD areas are 1 percent less than the saturation flow rates in OCD areas. Thus, no adjustment factors are proposed for area type.

Through-Lane Utilization

Lane utilization factors take into account that vehicles in exclusive through lanes do not distribute equally among the lanes available on an approach. A review of FHWA films by the NCHRP study team (conducting research for the 1985 HCM) identified a mean value for the high volume in two through lanes as 54 percent. The mean value for the high volume in three through lanes was 39 percent. These percentages result in lane utilization factors of 1.08 and 1.17 for two and three through lanes, respectively. In the 1985 HCM, lane utilization factors of 1.05 for two-lane approaches and 1.10 for three or more lane approaches are recommended (reductions of 5 percent and 9 percent of capacity). In *Transportation Research Circular 212*, a lane utilization factor of 1.05 for a two-lane approach (which represents 52.5 percent of the approach volume in the heavier lane) and a 1.10 factor for a three-lane approach (which represents 37 percent of the approach volume in the heaviest lane) are recommended.

Surveys of 5,600 vehicles on two-lane approaches and 6,900 vehicles on three-lane approaches were conducted throughout the county for the research documented in this paper. The results of these surveys indicated that 51.9 percent of the through volume occurred in the heavier lane on a two-lane approach and 36.5 percent of the through volume occurred in the heaviest lane on a three-lane approach. Based on the survey results, lane utilization factors of 0.96 and 0.91 are proposed for approaches with two lanes and three or more lanes, respectively. These values are consistent with those specified in the 1985 HCM *(1)*.

Turning Movements

Exclusive Turn Lanes—Protected Phase

Left-Turn Lanes A reduction of 15 percent in saturation flow rates for exclusive left-turn lanes with protected signal phases was found in the NCHRP study *(6)*. This was based on a sample size of 205 left-turning vehicles (in saturation flow after the fourth vehicle in queue). The adjustment factor contained in the 1985 HCM for this condition is 0.95, which compares with a 3 percent reduction recommended by Messer and Fambro in their paper titled "Critical Lane Analysis for Intersection Design" *(2)*. In *Transportation Research Circular 212*, the recommended adjustment factor for an exclusive left-turn lane with a protected turn phase is 1.05 (equivalent to a 5 percent reduction in capacity) *(3)*.

Results of surveys of 774 left-turning vehicles in saturation flow conducted for the research documented in this paper indicated an average 3 percent reduction in saturation flow rate when compared with through-lane headways at comparable locations. Thus, a 0.97 factor for single left-turn lanes with protected phases is proposed.

Right-Turn Lanes The 1985 HCM includes a right-turn adjustment factor of 0.85 for exclusive right-turn lanes controlled by protected signal phases *(1)*. Messer and Fambro recommend that "when a separate right-turn lane is provided, neither right-turning volume nor right-turn lane is analyzed" *(2)*. Surveys were conducted for the research documented in this paper for 723 right-turning vehicles in saturation flow. The locations surveyed had curb radii of between 10 and 30 ft. It was found that the average saturation flow headway for these right-turning vehicles was 19 percent longer than through-vehicle headways at comparable locations. Thus, a factor of 0.84 is proposed for exclusive right-turn lanes with short turn radii. When turn radii of between 25 and 44 ft were provided, the Kentucky research found a saturation flow rate that was 8 percent higher than that for narrow radii *(5)*. Thus, a factor of 0.91 is proposed for wider right-turn radii (30 to 44 ft).

Exclusive Turn Lanes—Permissive Phase

Left-Turn Lanes The 1985 HCM includes a procedure for exclusive left-turn lanes controlled by permissive signal phases that involves 11 calculations to determine an adjustment factor. In *Transportation Research Circular 212*, the recommended adjustment factor for an exclusive left-turn lane that operates on a permissive phase is based on the through and right-turn opposing volume *(3)*. The factors (expressed in passenger car equivalents) range between 1.0 for opposing volumes of 0 to 299 and 6.0 for opposing volumes greater than 1,000.

The field studies conducted in Kentucky for lost time included a determination of the relationship between lost time per cycle and opposing volume per cycle for opposed left turns *(7)*. This relationship was based on surveys conducted at locations where left-turning vehicles move across one lane of opposing traffic (a two-lane street) and across two lanes of opposing traffic (a four-lane street). The survey results from Kentucky were analyzed for this paper and a set of adjustment factors was derived by calculating the incremental lost time per cycle (at various opposing volumes) that would result from the additional delay experienced by the left-turning vehicles. This incremental value of lost time was then converted to adjustment factors for a range of opposing volumes. The proposed adjustment factors are given in Table 8.

Right-Turn Lanes The two primary factors that influence the saturation flow rates for right-turning vehicles operating in an exclusive lane without a protected phase are the right-turn cornering radius and the extent of pedestrian interference in the opposing crosswalk. Thus, the volume in this lane should be factored by the appropriate radius factor (either 0.84 or 0.91, as previously described) and a pedestrian blockage factor, as presented in a subsequent section. This proposed procedure is consistent with that included in the 1985 HCM *(1)*.

TABLE 8 ADJUSTMENT FACTORS FOR EXCLUSIVE LEFT-TURN WITH PERMISSIVE PHASE

	Adjustment Factor	
Opposing Volume	Two-Lane Street	Four-Lane Street
100	0.94	0.98
200	0.87	0.96
300	0.81	0.94
400	0.75	0.92
500	0.69	0.89
600	0.63	0.87
700	0.58	0.85
800	0.53	0.83
900	0.47	0.81
1,000	0.41	0.80
1,100	0.35	0.79
1,200	0.30	0.76

Optional Turn–Through Lanes—Protected Phase

The 1985 HCM does not distinguish between the adjustment factor for a left-turning or right-turning vehicle in an exclusive lane or in an optional turn–through lane as long as the turn movement operates during a protected phase. In both cases, the 1985 HCM adjustment factor is dependent on the base factor appropriate for an exclusive turn lane adjusted for the proportion of turning vehicles in the shared lane *(1)*. A passenger car equivalent value of 1.2 for optional turn–through lanes with protected signal phases is suggested in *Transportation Research Circular 212 (3)*. This is equivalent to a 17 percent reduction in saturation flow rate.

Consistent with the 1985 HCM procedure, it is proposed that either the left-turn saturation flow factor (0.97) or the appropriate right-turn saturation flow factor (0.84 or 0.91) be modified by the percent of turning vehicles in the lane. For example, the left-turn adjustment factor should vary as follows:

Percent of Left Turns in Shared Lane	*Adjustment Factor*
100	0.97
67	0.98
33	0.99
0	1.00

Optional Turn–Through Lanes—Permissive Phase

Left-Turning Vehicles For left-turning vehicles in an optional turn–through lane that do not have a protected signal phase, the 1985 HCM uses the same procedure as for the permissive phase—exclusive left-turn lane option—with a modification introduced to recognize the portion of left-turning vehicles in the shared lane *(1)*. The factors in *Transportation Research Circular 212* for left-turning vehicles in a shared lane are identical for both the protected and permissive phase options *(3)*.

The lost time surveys conducted in Kentucky allowed for a relationship to be determined between the change in lost time per cycle for a range of opposing volumes and the percent of left-turning vehicles in the shared lane *(7)*. These survey results were analyzed for this paper to derive a set of adjustment factors for this shared lane–permissive phase option, consistent with the factors developed for the exclusive lane–permissive phase option. Table 9 gives the proposed adjustment factors for a range of opposing volume values. Note that in this table the opposing flow is expressed in volume per lane rather than total opposing volume (as in Table 8).

Right-Turning Vehicles Factors for right-turning vehicles operating in a shared lane–permissive phase option should be

TABLE 9 LEFT-TURN ADJUSTMENT FACTORS FOR SHARED THROUGH–LEFT-TURN LANE WITH PERMISSIVE PHASE

Opposing Volume/Lane	Percent Left-Turn Volume in Shared Lane		
	10	30	50
100	0.99	0.98	0.97
200	0.99	0.96	0.95
300	0.98	0.94	0.92
400	0.97	0.92	0.91
500	0.96	0.90	0.88
600	0.95	0.88	0.86
700	0.93	0.86	0.82
800	0.91	0.84	0.80
900	0.90	0.80	0.77
1,000	0.87	0.78	0.75
1,100	0.85	0.76	0.71
1,200	0.83	0.72	0.70

based on the factors derived for right-turning vehicles operating in a shared lane–protected phase option. The appropriate right-turn saturation flow factor (0.84 or 0.91) should be modified by the percent of turning vehicles in the shared lane. For example, it is proposed that the right-turn adjustment factors for an optional lane with a 10- to 30-ft right-turn radius be as follows:

Percent of Right Turns in Shared Lane	*Adjustment Factor*
100	0.84
75	0.88
50	0.92
25	0.96
0	1.00

Both the right-turn and left-turn volumes in shared turn–through lanes should also be adjusted by the pedestrian conflict adjustment factor described in a subsequent section.

Dual Turn Lanes

A reduction factor for dual exclusive turn lanes reflects the potential for uneven distribution of vehicles in the two turn lanes. The 1985 HCM includes adjustment factors of 0.92 for dual left-turn lanes and 0.75 for dual right-turn lanes *(1)*. It is suggested in *Transportation Research Circular 212* that 55 percent of the total left-turn volume be assigned to one lane for a dual left-turn lane. This results in an adjustment factor of 0.91 for the dual left-turn lane volume. Stokes found in his surveys in Texas that 50.6 percent of vehicles in dual left-turn lanes (sample size of 3,458) used the outside lane *(4)*. This results in an adjustment factor of 0.97.

Surveys were also conducted for the research documented in this paper for dual exclusive turn lanes to determine volume distribution. The sample sizes for these surveys were 2,026 and 856 for dual left-turn and dual right-turn lanes, respectively. A total of 50.3 percent of the vehicles surveyed in dual left-turn lanes were in the curb (inside) lane and 55.3 percent of the vehicles surveyed in dual right-turn lanes were in the curb (inside) lane. This was a significantly high use of the curb lane for dual right-turn movements because the right-turning radii varied from 10 to 50 ft. The lane distributions suggest factors of 0.97 and 0.90 for the dual left-turn and dual right-turn lanes, respectively, when compared with single exclusive turn lanes. Combining the single and dual turn lane adjustment factors for protected turn phase operation yields an overall factor that can be applied to the unadjusted turn volumes in dual exclusive turn lanes (yielding one adjustment factor rather than two for each dual turn lane calculation). The recommended overall adjustment factors are as follows: dual left-turn lane—0.94, and dual right-turn lane—0.76.

Pedestrian Conflict

Pedestrians in the opposing street crosswalk conflict with both right-turning and left-turning vehicles when they must turn without a protected phase. It is suggested in the 1985 HCM that right-turn volumes be adjusted by a factor that takes into account the number of pedestrians per hour in the conflicting crosswalk but does not suggest an adjustment for permissive left turns. This adjustment reduces the basic right-turn factor (0.85) in proportion to the conflicting pedestrian volume up to a maximum of 1,700 pedestrians per hour. In *Transportation Research Circular 212*, a series of passenger car equivalent values for right-turning vehicles is proposed based on pedestrian volume ranges in the conflicting crosswalk. These values are 1.0 for pedestrian volumes under 100 per hour, 1.25 for pedestrian volumes between 100 and 600, 1.50 for pedestrian volumes between 600 and 1,200, and 2.0 for pedestrian volumes greater than 1,200.

A detailed survey was conducted for UMTA by others in Washington, D.C. *(9)*. As a result of these surveys, the relationship between additional vehicle delay (beyond that delay normally experienced when turning) and the number of pedestrians per cycle in the crosswalk was developed. These survey results were used for the research documented in this paper to derive an equivalent percent reduction in green time available for right-turning vehicles in relation to the number of pedestrians per hour for various signal cycle lengths. The resulting adjustment factors for pedestrian flows (expressed in terms of the number of pedestrians per hour) are given in Table 10.

TABLE 10 RESULTING ADJUSTMENT FACTORS FOR PEDESTRIAN FLOWS

No. of Pedestrians/Hr	Factor	No. of Pedestrians/Hr	Factor
100	0.93	1,100	0.75
200	0.91	1,200	0.73
300	0.89	1,300	0.71
400	0.87	1,400	0.68
500	0.86	1,500	0.66
600	0.84	1,600	0.65
700	0.82	1,700	0.63
800	0.80	1,800	0.61
900	0.78	1,900	0.59
1,000	0.76	2,000	0.57

The values in Table 10 provide a less significant adjustment factor at the low end of pedestrian volumes (100 to 600 pedestrians) than do the *Transportation Research Circular 212* factors *(3)*. A reduction of between 7 and 16 percent is proposed for low pedestrian volumes versus a 20 percent reduction for comparable volumes contained in the *Circular 212* technique. It is suggested that the adjustment factors given in Table 10 be applied to the portion of right- and left-turning vehicles turning from exclusive or optional turn lanes on permissive signal phases.

Metropolitan Area Size

The *Transportation Research Circular 212* analysis technique *(3)* and the 1985 HCM *(1)* do not take into account variations in driver characteristics based on city size. Data from two medium-sized cities (population of from 250,000 to 400,000) and from two large cities (population of more than 1,000,000) were reviewed as a part of the NCHRP study *(6)*. The results of this data review indicated that saturation flow rates in the two medium-sized cities were 11 percent higher than saturation flow rates in the two large cities.

Surveys in Kentucky were conducted in two communities with populations of more than 100,000 persons, three communities with populations of between 20,000 and 50,000 persons, and three communities with populations of less than 20,000 persons *(5)*. Saturation flow rates in the cities with 20,000 to 50,000 persons were 8 percent lower than in the largest city surveyed. Saturation flow rates in the cities with populations less than 20,000 persons were 17 percent lower than those in the largest city surveyed.

The surveys for the research documented in this paper for the metropolitan area size factor included an analysis of saturation flow rates in three communities with populations of between 50,000 and 100,000 persons and three communities with populations of between 300,000 and 800,000 persons. The sample sizes for these surveys were 671 saturation flow vehicles and 1,169 saturation flow vehicles, respectively. The surveys were compared with the baseline saturation flow surveys, collected at intersections in metropolitan areas with populations greater than 1,000,000 persons. The results of the data analysis indicated that saturation flow rates in communities with populations of between 300,000 and 800,000 persons were identical to saturation flow rates in larger communities. Saturation flow rates in communities with populations of between 50,000 and 100,000 persons were 9 percent lower than in the other communities surveyed (consistent with the Kentucky data for the population category of 20,000 to 50,000). Thus, the following adjustment factors are proposed to reflect the effect of metropolitan area size: population of more than 100,000—1.00; population of between 20,000 and 100,000—0.91; and population of less than 20,000—0.83.

One-Way or Two-Way Operation

The NCHRP research effort for the 1985 HCM found that saturation flow rates decreased by 7.5 percent on one-way streets compared with saturation flow rates on two-way streets. A total of 356 saturation flow vehicles were surveyed on one-way streets in this study. The *Transportation Research Circular 212* technique does not include an adjustment factor for one-way flow on urban streets *(3)*. Moreover, no such adjustment factor is included in the 1985 HCM procedure *(1)*.

The surveys on one-way streets for the research documented in this paper included 1,514 saturation flow vehicles. It was found that the one-way saturation flow rates were 9 percent lower on one-way streets than on comparable two-way streets that had baseline geometric conditions (consistent with the NCHRP surveys). The surveys conducted for the research documented in this paper were conducted on approaches with relatively congested peak-period conditions in major metropolitan areas. Thus, the lower saturation flow rates on one-way streets cannot be attributed to unpressured driving characteristics. Further investigation indicated that lower saturation flow rates could occur on one-way street approaches compared with those on two-way street approaches for two reasons:

- At most urban intersections, there is no difference in side friction between a one-way street approach and a two-way street approach. Many two-way street approaches have raised medians that separate the two directions of flow. Thus, the left lane on a one-way street approach has lateral characteristics comparable to those of the median approach lane on a two-way street approach.
- Most urban one-way streets (including the approaches surveyed) contain relatively frequent traffic signal spacing and good signal progression. Thus, many peak-period motorists leaving an intersection after a red signal phase will adjust their speeds to work into the signal progression band. This frequently results in a slightly reduced departure speed (below the speed that the motorist would otherwise select) and slightly longer headways between vehicles in saturation flow conditions on the approaches.

As a result of the operating conditions stated, an isolated intersection approach along a one-way street may experience lower saturation flow rates compared with those of a comparable two-way street approach; however, route and system efficiency along a one-way street is increased because traffic signal progression reduces the number of stops per vehicle and the overall average vehicle delay. Therefore, consistent with the field data collected during this study and the NCHRP surveys, it is proposed that an adjustment factor of 0.91 (9 percent reduction) be applied to one-way street approach lanes. It appears from observation that applying both the factor for a one-way street and the factor for two or three approach lanes (one-way factor and lane utilization factor) underestimates the critical volumes that can be accommodated. Thus, only the greater of the two factors should be applied in this instance.

Summary of Adjustment Factors

The proposed adjustment factors for each of the 12 characteristics described in this paper are given in the summary in Table 11. They reflect the results of extensive data collection efforts contained in *Transportation Research Circular 212*, the 1985

TABLE 11 SUMMARY OF ADJUSTMENT FACTORS

Adjustment	Range of Values	Factor
1. Lane width	8 to 8.9 ft	0.95
	9 to 9.9 ft	0.98
	13.1 to 15.9 ft	1.05
2. Heavy vehicles	2 percent heavy vehicles	0.98
	6 percent heavy vehicles	0.95
	10 percent heavy vehicles	0.92
3. grade on vertical approach	5 to 6 percent downgrade	0.96
	3 to 4 percent downgrade	0.98
	3 to 4 percent upgrade	0.95
	5 to 6 percent upgrade	0.90
4. Curb Parking		
Approach 1		
0 maneuvers/hr		0.89
20 maneuvers/hr		0.82
40 maneuvers/hr		0.75
Approach 2		
0 maneuvers/hr		0.94
20 maneuvers/hr		0.91
40 maneuvers/hr		0.87
Approach 3		
0 maneuvers/hr		0.97
20 maneuvers/hr		0.95
40 maneuvers/hr		0.93
5. Local bus stop activity		
Approach 1		
0 buses stopping/hr		1.00
20 buses stopping/hr		0.95
40 buses stopping/hr		0.90
Approach 2		
0 buses stopping/hr		1.00
20 buses stopping/hr		0.97
40 buses stopping/hr		0.95
Approach 3		
0 buses stopping/hr		1.00
20 buses stopping/hr		0.99
40 buses stopping/hr		0.97
6. Area type		No adjustment
7. Through-lane utilization		
Two lanes		0.96
Three or more lanes		0.91
8. Turning movements		
Exclusive left-turn lane/protected phase	–	0.97
Exclusive right-turn lane/protected phase	10- to 30-ft radius	0.84
Exclusive right-turn lane/protected phase	30- to 44-ft radius	0.91
Exclusive left-turn lane/permissive phase	–	See Table 3 (times pedestrian factor)
Exclusive right-turn lane/permissive phase	10- to 30-ft radius	0.84 (times pedestrian factor)
Exclusive right-turn lane/permissive phase	30- to 44-ft radius	0.91 (times pedestrian factor)
Optional left-turn through lane/protected phase	–	0.97 (varies by percent turns)
Optional right-turn through lane/protected phase	–	0.84 or 0.91 (varies by percent turns)
Optional left-turn through lane/permissive phase	–	See Table 4 (times pedestrian factor)
Optional right-turn through lane/permissive phase	–	0.84 or 0.91 (varies by percent turns and pedestrian factor)
9. Dual turn lanes (protected phase)/left turns	–	0.94
Dual turn lanes (protected phase)/right turns	–	0.76
10. Pedestrian conflict (permissive left- or right-turn phase)		
100 pedestrians/hr		0.93
200 pedestrians/hr		0.91
400 pedestrians/hr		0.87
800 pedestrians/hr		0.80
1,000 pedestrians/hr		0.76
1,400 pedestrians/hr		0.68
1,800 pedestrians/hr		0.61
2,000 pedestrians/hr		0.57
11. Metropolitan area size	Population of 0 to 20,000	0.83
	Population of 20,000 to 100,000	0.91
12. One-way or two-way operation	One-way operation	0.91

Source: Barton–Aschman Associates, Inc.

HCM, the surveys for the research documented in this paper, and other relevant sources. The proposed factors are similar to the factors contained in the 1985 HCM for curb parking, lane utilization, and exclusive right-turn lanes (protected phase). The factors given in Table 11 are proposed for use in the 1985 HCM technique for signalized intersections and in other signalized capacity analysis techniques in which demand volumes must be adjusted to account for geometric, vehicle characteristic, and environmental conditions.

CONCLUSION

An improved set of saturation flow rates and adjustment factors for signalized intersection capacity analyses has been presented. These values are based on an extensive data base collected throughout the United States. The factors have been structured to be incorporated into calculations of signalized intersection capacity when saturation flow rates must be considered. It is hoped that the results of this research effort will encourage the continuation of inquiry into the values incorporated into signalized intersection capacity analysis procedures.

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