**CHAPTER 7**

# TRAFFIC FLOW PARAMETERS

## CONTENTS



## EXHIBITS

# I. INTRODUCTION

Three basic variables—volume or flow rate, speed, and density—can be used to describe traffic on any roadway. In this manual, volume or traffic flow is a parameter common to both uninterrupted- and interrupted-flow facilities, but speed and density apply primarily to uninterrupted flow. Some parameters related to flow rate, such as spacing and headway, also are used for both types of facilities; other parameters, such as saturation flow or gap, are specific to interrupted flow.

# II. UNINTERRUPTED FLOW

## VOLUME AND FLOW RATE

Basic concepts for uninterrupted-flow facilities: volume, flow rate, speed, density, headway, and capacity

Volume and flow rate are two measures that quantify the amount of traffic passing a point on a lane or roadway during a given time interval. These terms are defined as follows:

- Volume—the total number of vehicles that pass over a given point or section of a lane or roadway during a given time interval; volumes can be expressed in terms of annual, daily, hourly, or subhourly periods.
- Flow rate—the equivalent hourly rate at which vehicles pass over a given point or section of a lane or roadway during a given time interval of less than 1 h, usually 15 min.

Volume and flow are variables that quantify demand, that is, the number of vehicle occupants or drivers (usually expressed as the number of vehicles) who desire to use a given facility during a specific time period. Congestion can influence demand, and observed volumes sometimes reflect capacity constraints rather than true demand.

The distinction between volume and flow rate is important. Volume is the number of vehicles observed or predicted to pass a point during a time interval. Flow rate represents the number of vehicles passing a point during a time interval less than 1 h, but expressed as an equivalent hourly rate. A flow rate is the number of vehicles observed in a subhourly period, divided by the time (in hours) of the observation. For example, a volume of 100 vehicles observed in a 15-min period implies a flow rate of 100 veh/0.25 h or 400 veh/h.

Volume and flow rate can be illustrated by the volumes observed for four consecutive 15-min periods. The four counts are 1,000, 1,200, 1,100, and 1,000. The total volume for the hour is the sum of these counts, or 4,300 veh. The flow rate, however, varies for each 15-min period. During the 15-min period of maximum flow, the flow rate is 1,200 veh/0.25 h, or 4,800 veh/h. Note that 4,800 vehicles do not pass the observation point during the study hour, but they do pass at that rate for 15 min.

Consideration of peak flow rates is important in capacity analysis. If the capacity of the segment of highway studied is 4,500 veh/h, capacity would be exceeded during the peak 15-min period of flow, when vehicles arrive at a rate of 4,800 veh/h, even though volume is less than capacity during the full hour. This is a serious problem, because dissipating a breakdown of capacity can extend congestion for up to several hours.

Calculating a peak-hour factor

Peak flow rates and hourly volumes produce the peak-hour factor (PHF), the ratio of total hourly volume to the peak flow rate within the hour, computed by Equation 7-1:

$$PHF = \frac{\text{Hourly volume}}{\text{Peak flow rate (within the hour)}} \tag{7-1}$$

If 15-min periods are used, the PHF may be computed by Equation 7-2:

$$PHF = \frac{V}{4 \times V_{15}} \tag{7-2}$$

where

$PHF$ = peak-hour factor,
$V$ = hourly volume (veh/h), and
$V_{15}$ = volume during the peak 15 min of the peak hour (veh/15 min).

When the PHF is known, it can convert a peak-hour volume to a peak flow rate, as in Equation 7-3:

$$v = \frac{V}{PHF} \tag{7-3}$$

where

$v$ = flow rate for a peak 15-min period (veh/h),
$V$ = peak-hour volume (veh/h), and
$PHF$ = peak-hour factor.

Equation 7-3 does not need to be used to estimate peak flow rates if traffic counts are available; however, the chosen count interval must identify the maximum 15-min flow period. The rate then can be computed directly as 4 times the maximum 15-min count. When flow rates in terms of vehicles are known, a conversion to a flow rate in terms of passenger car equivalents (pce) can be computed using the PHF and the heavy vehicle factor.

## SPEED

Although traffic volumes provide a method of quantifying capacity values, speed (or its reciprocal of travel time) is an important measure of the quality of the traffic service provided to the motorist. It is an important measure of effectiveness defining levels of service for many types of facilities, such as rural two-lane highways, urban streets, freeway weaving segments, and others.

Speed parameters

Speed is defined as a rate of motion expressed as distance per unit of time, generally as kilometers per hour (km/h). In characterizing the speed of a traffic stream, a representative value must be used, because a broad distribution of individual speeds is observable in the traffic stream. In this manual, average travel speed is used as the speed measure because it is easily computed from observation of individual vehicles within the traffic stream and is the most statistically relevant measure in relationships with other variables. Average travel speed is computed by dividing the length of the highway, street section, or segment under consideration by the average travel time of the vehicles traversing it. If travel times $t_1$, $t_2$, $t_3$,..., $t_n$ (in hours) are measured for n vehicles traversing a segment of length L, the average travel speed is computed using Equation 7-4.

$$S = \frac{nL}{\sum_{i=1}^{n} t_i} = \frac{L}{\frac{1}{n}\sum_{i=1}^{n} t_i} = \frac{L}{t_a} \tag{7-4}$$

where

$S$ = average travel speed (km/h),
$L$ = length of the highway segment (km),
$t_i$ = travel time of the ith vehicle to traverse the section (h),
$n$ = number of travel times observed, and
$t_a$ = $\frac{1}{n}\sum_{i=1}^{n} t_i$ = average travel time over L (h).

The travel times in this computation include stopped delays due to fixed interruptions or traffic congestion. They are total travel times to traverse the defined roadway length.

Several different speed parameters can be applied to a traffic stream. These include the following:

Average running speed—A traffic stream measure based on the observation of vehicle travel times traversing a section of highway of known length. It is the length of the segment divided by the average running time of vehicles to traverse the segment. Running time includes only time that vehicles are in motion.

Average travel speed—A traffic stream measure based on travel time observed on a known length of highway. It is the length of the segment divided by the average travel time of vehicles traversing the segment, including all stopped delay times. It is also a space mean speed.

Space mean speed—A statistical term denoting an average speed based on the average travel time of vehicles to traverse a segment of roadway. It is called a space mean speed because the average travel time weights the average to the time each vehicle spends in the defined roadway segment or space.

Time mean speed—The arithmetic average of speeds of vehicles observed passing a point on a highway; also referred to as the average spot speed. The individual speeds of vehicles passing a point are recorded and averaged arithmetically.

Free-flow speed—The average speed of vehicles on a given facility, measured under low-volume conditions, when drivers tend to drive at their desired speed and are not constrained by control delay.

For most of the procedures using speed as a measure of effectiveness in this manual, average travel speed is the defining parameter. For uninterrupted-flow facilities not operating at level of service (LOS) F, the average travel speed is equal to the average running speed.

Exhibit 7-1 shows a typical relationship between time mean and space mean speeds. Space mean speed is always less than time mean speed, but the difference decreases as the absolute value of speed increases. Based on the statistical analysis of observed data, this relationship is useful because time mean speeds often are easier to measure in the field than space mean speeds.

Average running speed

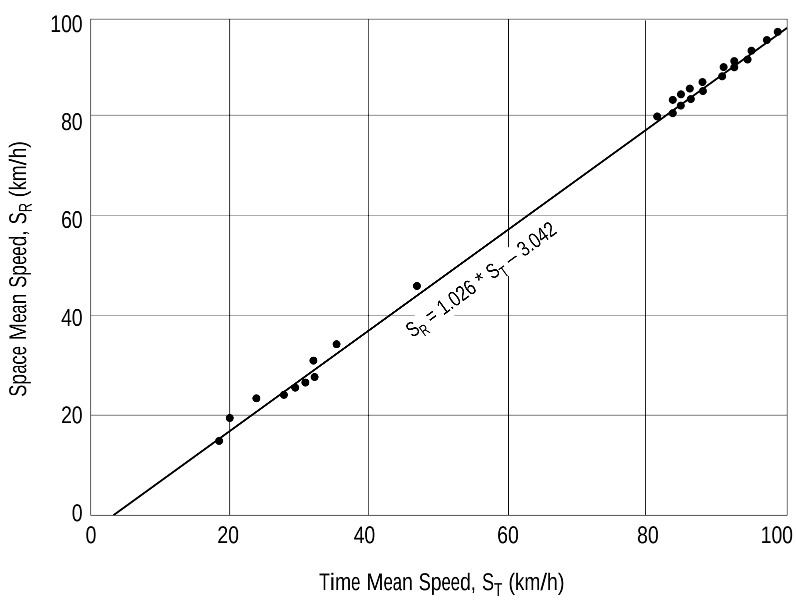
Average travel speed

Space mean speed

Time mean speed

Free-flow speed

EXHIBIT 7-1. TYPICAL RELATIONSHIP BETWEEN TIME MEAN AND SPACE MEAN SPEED




Source: Drake et al. (*1*).

It is possible to calculate both time mean and space mean speeds from a sample of individual vehicle speeds. For example, three vehicles are recorded with speeds of 40, 60, and 80 km/h. The time to traverse 1 km is 1.5 min, 1.0 min, and 0.75 min, respectively. The time mean speed is 60 km/h, calculated as (40 + 60 + 80)/3. The space mean speed is 55.4 km/h, calculated as (60)[3 ÷ (1.5 + 1.0 + 0.75)].

For capacity analysis, speeds are best measured by observing travel times over a known length of highway. For uninterrupted-flow facilities operating in the range of stable flow, the length may be as short as 50 to 100 m for ease of observation.

As measures of effectiveness, speed criteria must recognize driver expectations and roadway function. For example, a driver expects a higher speed on a freeway than on an urban street. Lower free-flow speeds are tolerable on a roadway with more severe horizontal and vertical alignment, since drivers are not comfortable driving at high speeds. LOS criteria reflect these expectations.

## DENSITY

Density is the number of vehicles (or pedestrians) occupying a given length of a lane or roadway at a particular instant. For the computations in this manual, density is averaged over time and is usually expressed as vehicles per kilometer (veh/km) or passenger cars per kilometer (pc/km).

Direct measurement of density in the field is difficult, requiring a vantage point for photographing, videotaping, or observing significant lengths of highway. Density can be computed, however, from the average travel speed and flow rate, which are measured more easily. Equation 7-5 is used for undersaturated traffic conditions.

Computing density

$$D = \frac{v}{S} \tag{7-5}$$

where

$v$ = flow rate (veh/h),
$S$ = average travel speed (km/h), and
$D$ = density (veh/km).

A highway segment with a rate of flow of 1,000 veh/h and an average travel speed of 50 km/h would have a density of

$$D = \frac{1000\ veh/h}{50\ km/h} = 20\ veh/km$$

Density is a critical parameter for uninterrupted flow facilities because it characterizes the quality of traffic operations. It describes the proximity of vehicles to one another and reflects the freedom to maneuver within the traffic stream.

Roadway occupancy is frequently used as a surrogate for density in control systems because it is easier to measure. Occupancy in space is the proportion of roadway length covered by vehicles, and occupancy in time identifies the proportion of time a roadway cross section is occupied by vehicles.

## HEADWAY AND SPACING

Spacing is the distance between successive vehicles in a traffic stream, measured from the same point on each vehicle (e.g., front bumper, rear axle, etc.). Headway is the time between successive vehicles as they pass a point on a lane or roadway, also measured from the same point on each vehicle.

These characteristics are microscopic, since they relate to individual pairs of vehicles within the traffic stream. Within any traffic stream, both the spacing and the headway of individual vehicles are distributed over a range of values, generally related to the speed of the traffic stream and prevailing conditions. In the aggregate, these microscopic parameters relate to the macroscopic flow parameters of density and flow rate.

Spacing is a distance, measured in meters. It can be determined directly by measuring the distance between common points on successive vehicles at a particular instant. This generally requires complex aerial photographic techniques, so that spacing usually derives from other direct measurements. Headway, in contrast, can be easily measured with stopwatch observations as vehicles pass a point on the roadway.

Relationships among density, speed and flow rate, and headway and spacing

The average vehicle spacing in a traffic stream is directly related to the density of the traffic stream, as determined by Equation 7-6.

$$Density\ (veh/km) = \frac{1000}{spacing\ (m/veh)} \qquad (7\text{-}6)$$

The relationship between average spacing and average headway in a traffic stream depends on speed, as indicated in Equation 7-7.

$$Headway\ (s/veh) = \frac{spacing\ (m/veh)}{speed\ (m/s)} \qquad (7\text{-}7)$$

This relationship also holds for individual headways and spacings between pairs of vehicles. The speed is that of the second vehicle in a pair of vehicles. Flow rate is related to the average headway of the traffic stream with Equation 7-8.

$$Flow\ rate\ (veh/h) = \frac{3600}{headway\ (s/veh)} \qquad (7\text{-}8)$$

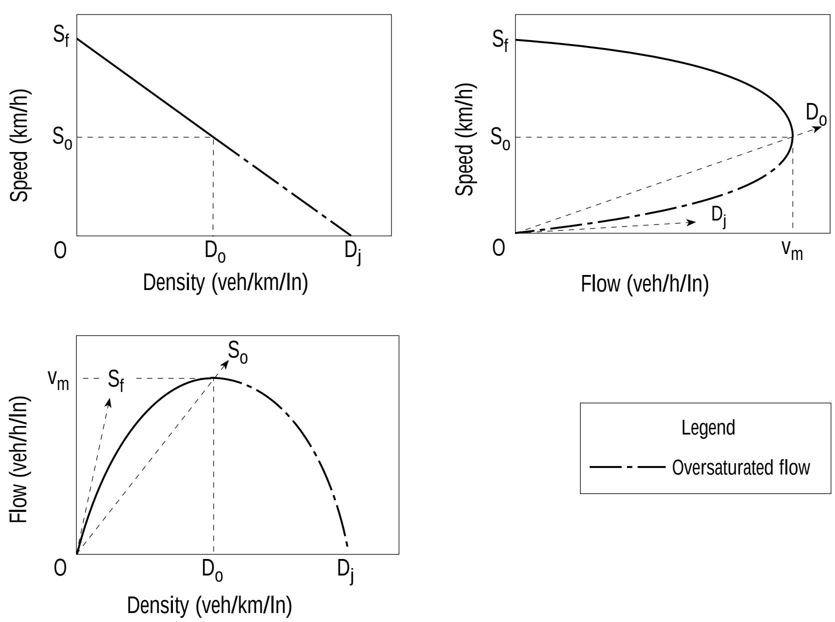
## RELATIONSHIPS AMONG BASIC PARAMETERS

Equation 7-5 cites the basic relationship among the three parameters, describing an uninterrupted traffic stream. Although the equation $v = S * D$ algebraically allows for a given flow rate to occur in an infinite number of combinations of speed and density, there are additional relationships restricting the variety of flow conditions at a location.

Exhibit 7-2 shows a generalized representation of these relationships, which are the basis for the capacity analysis of uninterrupted-flow facilities. The flow-density function is placed directly below the speed-density relationship because of their common horizontal scales, and the speed-flow function is placed next to the speed-density relationship because of their common vertical scales. Speed is space mean speed.

Illustration of speed-density, flow-density, and speed-flow relationships

EXHIBIT 7-2. GENERALIZED RELATIONSHIPS AMONG SPEED, DENSITY, AND FLOW RATE ON UNINTERRUPTED-FLOW FACILITIES




Source: Adapted from May (*2*).

The form of these functions depends on the prevailing traffic and roadway conditions on the segment under study and on its length in determining density. Although the diagrams in Exhibit 7-2 show continuous curves, it is unlikely that the full range of the functions would appear at any particular location. Survey data usually show discontinuities, with part of these curves not present (*2*).

The curves of Exhibit 7-2 illustrate several significant points. First, a zero flow rate occurs under two different conditions. One is when there are no vehicles on the facility—density is zero, and flow rate is zero. Speed is theoretical for this condition and would be selected by the first driver (presumably at a high value). This speed is represented by $S_f$ in the graphs.

The second is when density becomes so high that all vehicles must stop—the speed is zero, and the flow rate is zero, because there is no movement and vehicles cannot pass a point on the roadway. The density at which all movement stops is called jam density, denoted by $D_j$ in the diagrams.

Between these two extreme points, the dynamics of traffic flow produce a maximizing effect. As flow increases from zero, density also increases, since more vehicles are on the roadway. When this happens, speed declines because of the interaction of vehicles. This decline is negligible at low and medium densities and flow rates. As density increases, these generalized curves suggest that speed decreases significantly before capacity is achieved. Capacity is reached when the product of density and speed results in the maximum flow rate. This condition is shown as optimum speed $S_o$ (often called critical speed), optimum density $D_o$ (sometimes referred to as critical density), and maximum flow $v_m$.

The slope of any ray line drawn from the origin of the speed-flow curve to any point on the curve represents density, based on Equation 7-5. Similarly, a ray line in the flow-density graph represents speed. As examples, Exhibit 7-2 shows the average free-flow speed and speed at capacity, as well as optimum and jam densities. The three diagrams are redundant, since if any one relationship is known, the other two are uniquely defined. The speed-density function is used mostly for theoretical work; the other two are used in this manual to define LOS.

As shown in Exhibit 7-2, any flow rate other than capacity can occur under two different conditions, one with a high speed and low density and the other with high density and low speed. The high-density, low-speed side of the curves represents oversaturated flow. Sudden changes can occur in the state of traffic (i.e., in speed, density, and flow rate). LOS A though E are defined on the low-density, high-speed side of the curves, with the maximum-flow boundary of LOS E placed at capacity; by contrast, LOS F, which describes oversaturated and queue discharge traffic, is represented by the high-density, low-speed part of the functions.

## III. INTERRUPTED FLOW

Basic concepts for interrupted-flow facilities: intersection control, saturation flow rate, lost time, and queuing

Interrupted flow is more complex than uninterrupted flow because of the time dimension involved in allocating space to conflicting traffic streams. On an interrupted-flow facility, flow usually is dominated by points of fixed operation, such as traffic signals and stop signs. These controls have different impacts on overall flow.

The operational state of traffic at an interrupted traffic-flow facility is defined by the following measures:

- Volume and flow rate,
- Saturation flow and departure headways,
- Control variables (stop or signal control),
- Gaps available in the conflicting traffic streams, and
- Delay.

The discussion of volume and flow rate in the first part of this chapter also is applicable to interrupted-flow facilities. An important additional point is the screenline at which the traffic volume or flow rate is surveyed. Traditional intersection traffic counts yield only the number of vehicles that have departed the intersection. The maximum flow is therefore limited to the capacity of the facility. When demand exceeds capacity and a queue is growing, it is advisable to survey traffic upstream, before the congestion.

## SIGNAL CONTROL

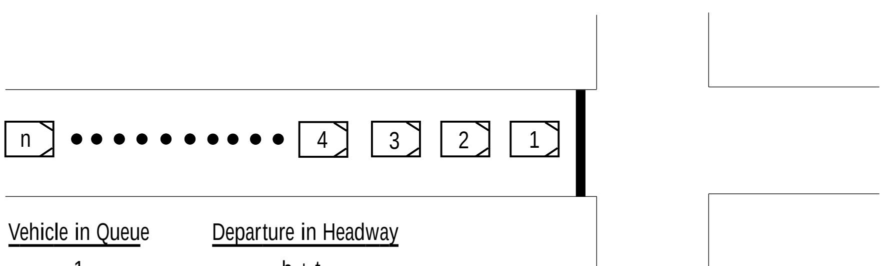
The most significant source of fixed interruptions on an interrupted-flow facility is the traffic signal. Traffic signals periodically halt flow in each movement or set of movements. Movement on a given set of lanes is possible only for a portion of the total time, because the signal prohibits movement during some periods. Only the time during which the signal is effectively green is available for movement. For example, if one set of lanes at a signalized intersection receives a 30-s effective green time out of a 90-s total cycle, only 30/90 or 1/3 of total time is available for movement on the subject lanes. Thus, only 20 minutes of each hour are available for flow on the lanes. If the lanes can accommodate a maximum flow rate of 1,500 veh/h with the signal green for a full hour, they could accommodate a total rate of flow of only 500 veh/h, since only one-third of each hour is available as green.

Impact of signal control on maximum flow rate

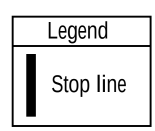
Because signal timings are subject to change, it is convenient to express capacities and service flow rates for signalized intersections in terms of vehicles per hour (veh/h). In the previous example, the maximum flow rate is 1,500 veh/h. This can be converted to a real-time value by multiplying it by the ratio of effective green time to the cycle length for the signal.

When the signal turns green, the dynamics of starting a stopped queue of vehicles must be considered. Exhibit 7-3 shows a queue of vehicles stopped at a signal. When the signal turns green, the queue begins to move. The headway between vehicles can be observed as the vehicles cross the stop line of the intersection. The first headway would be the elapsed time, in seconds, between the initiation of the green and the crossing of the front wheels of the first vehicle over the stop line. The second headway would be the elapsed time between the crossing of front wheels of the first and of the second vehicles over the stop line. Subsequent headways are measured similarly.

EXHIBIT 7-3. CONDITIONS AT TRAFFIC INTERRUPTION IN AN APPROACH LANE OF A SIGNALIZED INTERSECTION




| Vehicle in Queue | Departure in Headway |
|---|---|
| 1 | $h + t_1$ |
| 2 | $h + t_2$ |
| 3 | $h + t_3$ |
| 4 | $h + t_4$ |
| . | h |
| . | h |
| . | . |
| . | . |
| . | . |
| n | h |



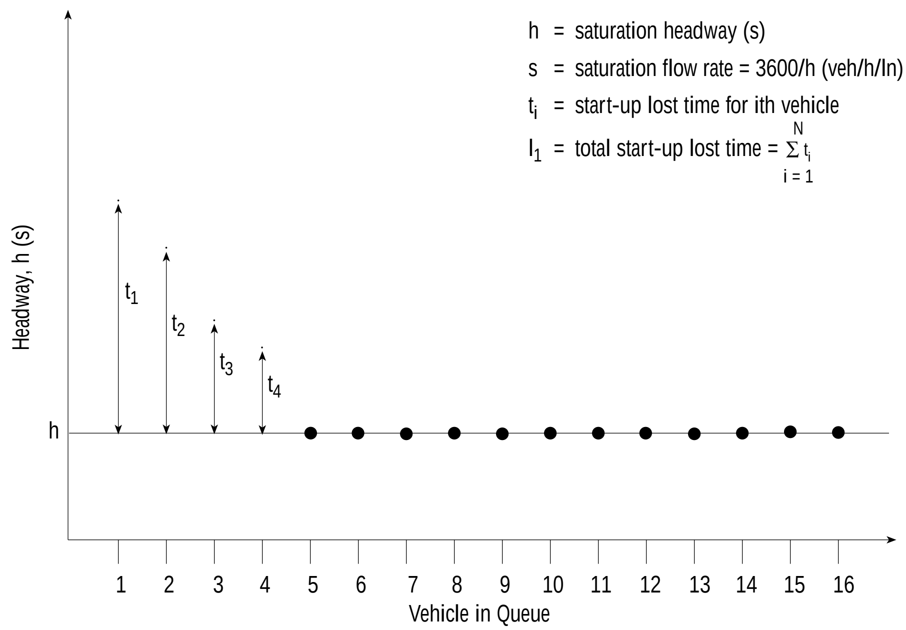

Determining average headway

The driver of the first vehicle in the queue must observe the signal change to green and react to the change by releasing the brake and accelerating through the intersection. The first headway will be comparatively long, as a result. The second vehicle in the queue follows a similar process, except that the reaction and acceleration period can occur while the first vehicle is beginning to move. The second vehicle will be moving faster than the first as it crosses the stop line, because it has length in which to accelerate. Its headway will generally be less than that of the first vehicle. The third and fourth vehicles follow a similar procedure, each achieving a slightly lower headway than the preceding vehicle. After four vehicles, the effect of the start-up reaction and acceleration has dissipated. Successive vehicles then move past the stop line at a steady speed until the last vehicle in the original queue has passed. The headway for these vehicles will be relatively constant.

In Exhibit 7-3, this constant average headway, denoted as h, is achieved after four vehicles. The headways for the first four vehicles are, on the average, greater than h and are expressed as $h + t_i$, where $t_i$ is the incremental headway for the ith vehicle due to the start-up reaction and acceleration. As i increases from 1 to 4, $t_i$ decreases.

Exhibit 7-4 shows a conceptual plot of headways. In this manual, for practical reasons, the fifth vehicle following the beginning of a green is used as the starting point for saturation flow measurements.

EXHIBIT 7-4. CONCEPT OF SATURATION FLOW RATE AND LOST TIME




The value h represents the saturation headway, estimated as the constant average headway between vehicles after the fourth vehicle in the queue and continuing until the last vehicle that was in the queue at the beginning of the green has cleared the intersection. The saturation headway is the amount of time that a vehicle in the stopped queue takes to pass through a signalized intersection on the green signal, assuming that there is a continuous queue of vehicles moving through the intersection.

In this manual, the definition of saturation headway differs for interrupted-flow and uninterrupted-flow facilities. For interrupted flow, headway represents the time between the passage of the front axle of one vehicle and of the front axle of the next vehicle over a given cross section of the roadway; for uninterrupted-flow facilities, the vehicle reference points usually are the front bumpers of the vehicles.

## STOP- OR YIELD-CONTROLLED INTERSECTIONS

The driver on a minor street or turning left from the major street of a two-way stop-controlled intersection faces a specific task: selecting a gap in the priority flow through which to execute the desired movement. The term gap refers to the space between the vehicles on the roadway that has the right-of-way at an unsignalized intersection. Gap acceptance describes the completion of a vehicle's movement into a gap.

The capacity of a minor street approach depends on two factors:

- The distribution of available gaps in the major-street traffic stream, and
- The gap sizes required by minor-street drivers to execute their desired movements.

Critical gap and gap acceptance

The distribution of available gaps in the major-street traffic stream depends on the total volume on the street, its directional distribution, the number of lanes on the major street, and the degree and type of platooning in the traffic stream. The gap sizes required by the minor-street drivers depend on the type of maneuver (left, through, right), the number of lanes on the major street, the speed of major-street traffic, sight distances, the length of time the minor-street vehicle has been waiting, and driver characteristics (eyesight, reaction time, age, etc.). The critical gap is the minimum time interval between the front bumpers of two successive vehicles in the major traffic stream that will allow the entry of one minor-street vehicle. When more than one minor-street vehicle uses one major-street gap, the time headway between the two minor-street vehicles is called follow-up time. In general, the follow-up time is shorter than the critical gap. Roundabouts operate similarly to two-way stop-controlled intersections. In roundabouts, however, entering drivers scan only one stream of traffic—the circulating stream—for an acceptable gap.

At an all-way stop-controlled intersection, all drivers must come to a complete stop. The decision to proceed is based in part on the rules of the road, which suggest that the driver on the right has the right-of-way; it also is a function of the traffic condition on the other approaches. The departure headway for the subject approach is defined as the time between the departure of one vehicle and that of the next behind it. A departure headway is considered a saturation headway if the second vehicle stops behind the first at the stop line. If there is traffic on one approach only, vehicles can depart as rapidly as the drivers can safely accelerate into and clear the intersection. If traffic is present on other approaches, the saturation headway on the subject approach will increase, depending on the degree of conflict between vehicles.

As at signalized intersections, the front axles of two consecutive vehicles are the reference points for determining the saturation headways of the vehicles departing from the stop line of two-way and all-way stop-controlled intersection approaches. In measuring the unobstructed flow of vehicles on the major roadway at a two-way stop-controlled intersection, the reference points normally are the front bumpers.

## SPEED

For interrupted-flow conditions, delay rather than speed is the primary measure of operations. However, speed measures similar to those for uninterrupted flow are helpful in determining the added travel time due to deceleration, movement in queues, and acceleration of vehicles passing through an intersection.

## DELAY

Control delay

Delay is a critical performance measure on interrupted-flow facilities. There are several types of delay, but in this manual, control delay is the principal service measure for evaluating LOS at signalized and unsignalized intersections. Although the definition of control delay is the same for signalized and unsignalized intersections, its application, including LOS threshold values, differs.

Control delay involves movements at slower speeds and stops on intersection approaches, as vehicles move up in the queue or slow down upstream of an intersection. Drivers frequently reduce speed when a downstream signal is red or there is a queue at the downstream intersection approach. Control delay requires the determination of a realistic average speed for each roadway segment. Any estimate of the average travel speed on urban streets implies the effects of control delay.

At two-way stop-controlled and all-way stop-controlled intersections, control delay is the total elapsed time from a vehicle joining the queue until its departure from the stopped position at the head of the queue. The control delay also includes the time required to decelerate to a stop and to accelerate to the free-flow speed.

## SATURATION FLOW RATE AND LOST TIME

Saturation flow rate is defined as the flow rate per lane at which vehicles can pass through a signalized intersection. By definition, it is computed by Equation 7-9:

Computing saturation flow rate and lost time

$$s = \frac{3600}{h} \quad (7\text{-}9)$$

where

$s$ = saturation flow rate (veh/h), and
$h$ = saturation headway (s).

The saturation flow rate represents the number of vehicles per hour per lane that can pass through a signalized intersection if the green signal was available for the full hour, the flow of vehicles was never halted, and there were no large headways.

Each time a flow is stopped, it must start again, with the first four vehicles experiencing the start-up reaction and acceleration headways shown in Exhibit 7-3. In this exhibit, the first four vehicles in the queue encounter headways longer than the saturation headway, h. The increments, $t_i$, are called start-up lost times. The total start-up lost time for the vehicles is the sum of the increments, computed using Equation 7-10.

$$l_1 = \sum_{i=1}^{N} t_i \quad (7\text{-}10)$$

where

$l_1$ = total start-up lost time (s),
$t_i$ = lost time for ith vehicle in queue (s), and
$N$ = last vehicle in queue.

Each stop of a stream of vehicles is another source of lost time. When one stream of vehicles stops, safety requires some clearance time before a conflicting stream of traffic is allowed to enter the intersection. This interval when no vehicles use the intersection is called clearance lost time, $l_2$.

In practice, signal cycles provide for this clearance through change intervals, which can include yellow or all-red indications or both. Drivers generally cannot observe this entire interval but can use the intersection during some portion of it. The clearance lost time, $l_2$, is the portion of this change interval not used by drivers.

The relationship between saturation flow rate and lost times is a critical one. For any given lane or movement, vehicles use the intersection at the saturation flow rate for a period equal to the available green time plus the change interval minus the start-up and clearance lost times. Because lost time is experienced with each start and stop of a movement, the total amount of time lost over an hour is related to the signal timing. For instance, if a signal has a 60-s cycle length, it will start and stop each movement 60 times per hour, and the total lost time per movement will be $60(l_1 + l_2)$.

Lost time affects capacity and delay. It might appear that the capacity of an intersection would increase with increased cycle length. But this is offset somewhat by the observation that the saturation headway, h, can increase if the length of a continuous green indication increases. Other intersection features, such as turning lanes, also can offset the reduction in capacity due to short cycles. Longer cycle lengths increase the number of vehicles in the queues and can cause the left-turn lane to overflow, reducing capacity by blocking the through-lanes.

As cycle length is increased, the control delay per vehicle also tends to increase, assuming that capacity is adequate. Delay, however, is a complex variable affected by many other variables besides cycle length.

## QUEUING

When demand exceeds capacity at an approach to a signalized intersection at the start of an effective green period, a queue forms (*2*). Because of the arrival of vehicles during the red phases, some vehicles might not clear the intersection during the given green phase. A queue also forms when arrivals wait at a service area. The service can be the arrival of an accepted gap in a major-street traffic stream, the payment of tolls at a toll booth or of parking fees at a parking garage, and so forth. Back of queue refers to the number of vehicles queued at an approach to a signalized intersection due to the arrival patterns of vehicles and to vehicles unable to clear the intersection during a given green phase (i.e., overflow). Most queuing theory relates to undersaturated conditions.

To predict the characteristics of a queuing system mathematically, it is necessary to specify the following system characteristics and parameters (*3*):

- Arrival pattern characteristics, including the average rate of arrival and the statistical distribution of time between arrivals;
- Service facility characteristics, including service-time average rates and the distribution and number of customers that can be served simultaneously or the number of channels available; and
- Queue discipline characteristics, such as the means of selecting which customer is next.

In oversaturated queues, the arrival rate is higher than the service rate; in undersaturated queues, the arrival rate is less than the service rate. The length of an undersaturated queue can vary but will reach a steady state with the arrival of vehicles. By contrast, the length of an oversaturated queue never will reach a steady state but will increase with the arrival of vehicles.

An undersaturated queue at a signalized intersection is shown in Exhibit 7-5 (*2*). The exhibit assumes queuing on one approach with two signal phases. In each cycle, the arrival demand is less than the capacity of the approach; no vehicles wait longer than one cycle; and there is no overflow from one cycle to the next. Exhibit 7-5(a) specifies the arrival rate, v, in vehicles per hour and is constant for the study period. The service rate, s, has two states: zero when the signal is effectively red and up to saturation flow rate when the signal is effectively green. Note that the service rate is equal to the saturation flow rate only when there is a queue.

Exhibit 7-5(b) diagrams cumulative vehicles over time. The horizontal line, v, in Exhibit 7-5(a) becomes a solid sloping line in Exhibit 7-5(b), with the slope equal to the flow rate. Thus the arrival rate goes through the origin and slopes up to the right with a slope equal to the arrival rate. Transferring the service rate from Exhibit 7-5(a) to Exhibit 7-5(b) creates a different graph. During the red period, the service rate is zero, so the service is shown as a horizontal line in the lower diagram. At the start of the green period, a queue is present, and the service rate is equal to the saturation flow rate. This forms a series of triangles, with the cumulative arrival line as the top side of each triangle and the cumulative service line forming the other two sides.

Each triangle represents one cycle length and can be analyzed to calculate the time duration of the queue. It starts at the beginning of the red period and continues until the queue dissipates. Its value varies between the effective red time and the cycle length, and it is computed using Equation 7-11.

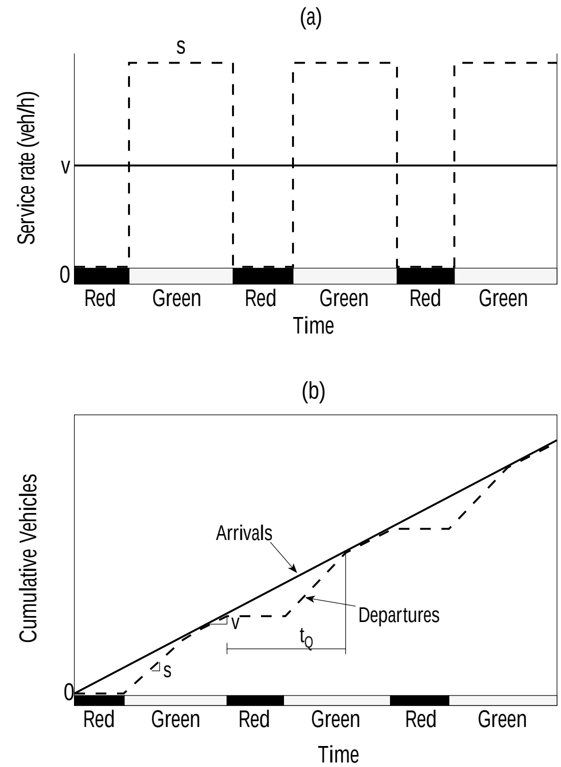
$$vt_Q = s(t_Q - r) \text{ or } t_Q = \frac{sr}{(s - v)} \qquad (7\text{-}11)$$

where

$t_Q$ = time duration of queue (s),

$v$ = mean arrival rate (veh/h),
$s$ = mean service rate (veh/h), and
$r$ = effective red time (s).

EXHIBIT 7-5. QUEUING DIAGRAM FOR SIGNALIZED INTERSECTION




Assumptions of queue length

The queue length is represented by the vertical distance through the triangle. At the beginning of red, the queue length is zero and increases to its maximum value at the end of the red period. Then the queue length decreases until the arrival line intersects the service line, when the queue length equals zero. Three queue lengths can be derived using the relationship shown in Exhibit 7-5: the maximum queue length, the average queue length while queue is present, and the average queue length; these are shown in Equations 7-12, 7-13, and 7-14, respectively.

$$Q_M = \frac{vr}{3600} \quad (7\text{-}12)$$

$$Q_Q = \frac{vr}{7200} \quad (7\text{-}13)$$

$$Q = \frac{Q_M t_Q}{2C} \quad (7\text{-}14)$$

where

$Q_M$ = maximum queue length (veh),
$Q_Q$ = average queue length while queue is present (veh),
$Q$ = average queue length (veh),
$v$ = mean arrival rate (veh/h),
$r$ = effective red time (s),
$C$ = cycle length (s), and
$t_Q$ = time duration of queue (s).

The queuing characteristics can be modeled by varying the arrival rate, service rate, and timing plan. In real-life situations, arrival rates and service rates are constantly changing. These variations complicate the model, but the basic relationships do not change. Queue length can be estimated for planning purposes by assuming a storage density (the average density of vehicles in the queue) and then using the relationship shown in Equation 7-15 (*4*). Note that demand must be greater than capacity to use this relationship.

$$QL = \frac{T * (v - c)}{N * d_s} \quad (7\text{-}15)$$

where

$QL$ = queue length (km),
$T$ = duration of analysis period (h),
$v$ = demand (veh/h),
$c$ = capacity (veh/h),
$N$ = number of lanes, and
$d_s$ = storage density (veh/km/ln).

## IV. REFERENCES

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2. May, A. D., Jr. *Traffic Flow Fundamentals*. Prentice-Hall, Englewood Cliffs, N.J., 1990.
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4. Dowling, R. G., W. Kittelson, J. Zegeer, and A. Skabardonis. *NCHRP Report 387: Planning Techniques to Estimate Speeds and Service Volumes for Planning Applications*. TRB, National Research Council, Washington, D.C., 1997.