REPRINT

Relating flow, density, speed and travel time models for uninterrupted and interrupted traffic

R. AKÇELIK

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NOTE:
This paper is related to the intersection analysis methodology used in the SIDRA INTERSECTION software. Since the publication of this paper, many related aspects of the traffic model have been further developed in later versions of SIDRA INTERSECTION. Though some aspects of this paper may be outdated, this reprint is provided as a record of important aspects of the SIDRA INTERSECTION software, and in order to promote software assessment and further research.
Relating flow, density, speed and travel time models for uninterrupted and interrupted traffic

by Rahmi Akçelik
Chief Research Scientist
ARRB Transport Research Ltd

Introduction. A vast amount of literature exists on the fundamental speed-density-flow models of traffic theory (e.g. May), TRB) and various travel time (cost)-flow models used for transport planning and demand management purposes (e.g. Akçelik). There is often a confusion about how these two groups of models are related, especially in terms of congested traffic conditions. These models have been discussed in some detail and the difficulty of relating the two groups of models has been emphasised in a recent Bureau of Transport and Communications Economics report on traffic congestion and road-user charges.

This paper offers an explanation of the relationships between speed, density, flow and travel time models for uninterrupted and interrupted traffic flows. The difference between arrival flows measured upstream of a queuing section and departure flows measured at a reference point along the road is emphasised. The former is related to demand while the latter is related to capacity. The difference is of particular importance in oversaturated (congested) conditions where demand exceeds capacity. The speed measured at a reference point along the road under congested conditions can be better understood as a queue discharge, queue formation or moving queue speed. This speed is associated with departure flow which cannot exceed the capacity flow. On the other hand, the average speed based on travel time through a road section including the travel distance upstream of the queuing section is associated with demand flow rate that can exceed the capacity.

Formulae and examples are given for the two types of speed-flow relationships as well as the basic relationships among speed, density, spacing and flow variables. Notations and definitions are given in the Appendix.

Headway, spacing, speed
As a starting-point, three basic variables describing the movement of a vehicle as observed at a reference point along the road are headway, spacing and speed. Headway (h) is the time between passage of the front ends of two successive vehicles. Spacing (L_h) is the distance corresponding to the headway time, i.e. the distance between the front end of the leading vehicle and the front end of the following vehicle. Speed (v) is the distance travelled per unit time.

As shown in Fig 1, the relationship between headway, spacing and speed is:

\[ v = \frac{L_h}{h} \]  \hspace{1cm} (1)

where \( h \) = headway (sec.)
\( L_h \) = spacing (m/veh)
\( v \) = vehicle speed (m/sec.)

Other variables shown in Fig 1 are the vehicle length, space (gap) length, and the corresponding vehicle passage time and gap time.

Vehicle passage time, \( t_p \), is the time between the passage of the front and back ends of a vehicle. Gap time, \( t_g \), is the time between the passage of the back end of the leading vehicle and the front end of the following vehicle, and is equivalent to headway time less vehicle passage time. Thus:

\[ t_p = \frac{L_h}{v} \]  \hspace{1cm} (2)
\[ t_g = h - t_p = h - \frac{L_h}{v} \]  \hspace{1cm} (3)
\[ t_p = \frac{L_h}{v} \]  \hspace{1cm} (4)

where \( h \) = headway (sec.)
\( t_p \) = vehicle passage time (sec.)
\( t_g \) = gap time (sec.)
\( L_h \) = vehicle length (m/veh)

In the calculations relating to average traffic conditions, the vehicle length should represent the actual traffic composition. Where the traffic stream is represented as a mixture of light vehicles (LVs) and heavy vehicles (HVs), the average vehicle length can be calculated as:

\[ L_v = (1 - p_{HV})L_{sv} + p_{HV}L_{svH} \]  \hspace{1cm} (5)

where \( p_{HV} \) = proportion of heavy vehicles in the traffic stream
\( L_{sv} \) = average vehicle length for light vehicles/or passenger car units (m/LV or m/pce)
\( L_{svH} \) = average vehicle length for heavy vehicles (m/HV)

Typical average vehicle lengths of \( L_{sv} = 4.0 \text{~m} \) and \( L_{svH} = 10 \text{~m} \) can be used where information is not available. For example, with 5 per cent heavy vehicles \( (p_{HV} = 0.05) \), the average vehicle length is found as:

\[ L_v = 0.95 \times 4 + 0.05 \times 10 = 4.3 \text{~m}. \]
Flow rate
Flow rate (veh/sec.) is the number of vehicles per unit passing (arriving or departing) a given reference point, and can be related to headway (sec.) through:

\[ h = \frac{1}{v} \quad \ldots \quad (6) \]

Thus, Equation (1) is equivalent to \( v = \frac{L}{h q} \), and vehicle spacing is given by \( L = \frac{v}{h q} \).

To understand the difference between congested and uncongested traffic operations, it is important to distinguish between the arrival (demand) flow rate and the departure flow rate for a given traffic facility. For example, at a signalised intersection approach lane, the departure flow rate measured at the stop-line is the queue discharge flow rate during the saturated portion of green period, \( q \) (departures from queue), and the arrival flow rate after queue has cleared, \( q \) (unqueued vehicles) as seen in Fig 2. The departure flow rate after queue clearance corresponds to the arrival (demand) flow rate measured under uninterrupted conditions at a point upstream of the back of queue, \( q_a \).

Density
Density (concentration) is the number of vehicles per unit distance, and is related to the average spacing through:

\[ k = \frac{1}{L_a} \quad \ldots \quad (7) \]

where \( L_a \) is in metres and \( k \) is in veh/m (or \( k = \frac{1000}{L_a} \) in veh/km).

Speed-density-flow relationships
As the vehicles speed up from a stationary queue, the space length between vehicles increases gradually, and therefore the spacing increases and the density decreases. The corresponding flow rate increases to a maximum flow \( (q_a) \) and then decreases as the speed increases towards the free-flow speed \( (v_f) \). The relationship between speed, density and flow is known as the fundamental relationship in traffic flow theory:

\[ q = kv \quad \ldots \quad (10) \]

where \( q \) is the flow rate (veh/h or veh/sec.), \( v \) is the speed (km/h or m/sec.) and \( k \) is the density (veh/km or veh/m).

This relationship is often discussed in the context of uninterrupted flows, e.g., as observed on freeways and rural roads. Recent ARRB Transport Research studies of vehicle spacing, headway and speed relationships for vehicles departing from a queue formed at traffic signals helped to understand the congested region of the speed-density relationship. Also refer to an earlier paper discussing application of speed-density-flow relationships to traffic observed at the signalised intersection stop-line (Fehe et al. 1973).

The speed-flow relationships for uninterrupted and uninterrupted conditions are depicted in Figs 3 and 4, respectively.

For uninterrupted traffic (Fig 3), the maximum flow rate, \( q_a \) is the capacity \( (Q = q_a) \). Region A represents undersaturated conditions with arrival flows below capacity \( q_a \leq Q \) which are associated with uninterrupted speeds \( (v_a \leq v_s, v_r) \), where \( v_s \) is the free-flow speed and \( v_r \) is the speed at maximum flow rate. Region B as observed at a reference point along the road represents oversaturated (congested) conditions with flow rates below the maximum flow rate \( (q_a < q_a \leq q_b) \) which are associated with reduced speeds \( (v_r \leq v_s) \).

Changes in conditions from Region A to Region B through the maximum flow point represents queue formation (e.g., due to two lanes of traffic merging into one lane, or traffic stopping at traffic signals). On the other hand, changes in conditions from Region B to Region A through the maximum flow point represents queue discharge (e.g., one lane of traffic diverging into two lanes, or traffic departing from a queue at traffic signals).
Region C for uninterrupted flow represents arrival flows above capacity \( q_a > q_s \) associated with average speeds based on travel time through the section. In this case, the flow represents the demand flow rate which can exceed the capacity value (as measured upstream of the reference point), and therefore Region C of the speed-flow curve is valid for degrees of saturation in excess of 1.0.

In Fig. 4 for interrupted traffic, capacity is given by \( Q = \frac{9s}{g+c} \) where \( s \) is the average queue discharge (saturation) flow rate, \( g \) is the effective green time and \( c \) is the cycle time. The average saturation flow rate is smaller than the maximum queue discharge rate \( (s < q_s) \) because of lower discharge rate at the start of the green period as seen in Fig. 2, and the capacity is the average saturation flow reduced by the available green time ratio, \( g/c \). The use of effective green and cycle time for unsignalised intersection capacity by traffic signal analysis has been described in Akcelik and Chung.

Region A for interrupted traffic represents approach (mid-block) travel conditions before the queuing section is entered.

The flow rate for the congested flow region (B), e.g., at a signalised intersection stop-line, is the rate of departure from the queue. This corresponds to the instantaneous queue discharge flow rate during the green period \( q_s \) that increases from zero to a steady maximum queue discharge flow rate \( q_s \) while the queue discharge speed increases from zero (stationary queue) to steady speed \( v_s \) corresponding to the maximum flow. This consideration brings an explanation to the speed-flow models for uninterrupted flows as well: the flow rate measured for forced flow (congested) conditions can be considered to be a reduced capacity flow (not actual flow). This is a result of vehicles (or moving queues) in the same stream interfering in the same way as the downstream queue interferes with the departure rate at the signal stop-line, resulting in reduced vehicle spacings, speeds and flow rates. Wardrop’s speed-flow surveys of traffic in a circular track are relevant in this respect.

Figure 4 shows the average speeds \( v_y \) based on section travel time (regions A’ and C) to be reduced from the approach (uninterrupted) speeds \( v_y = v_j \) as a result of the delays experienced due to traffic interruptions, e.g., red signal or give-way to major road traffic at a stop sign \( v_y = v_j \). Average speeds estimated by the SIDRA software package for interrupted movements correspond to the speed-flow relationship consisting of regions A’ and C in Fig. 4.

The free-flow speed for uninterrupted flow \( v_f \) is the average speed that occurs under zero-flow conditions. The corresponding zero-flow speed for interrupted flow \( v_{zr} \) includes the free-flow travel time for uninterrupted flow plus total minimum (zero-flow) delay \( d_{zr} \) at traffic interruptions.

There is a vast amount of literature on speed-density-flow relationships for uninterrupted traffic offering empirical as well as theoretical models based on car-following, hydrodynamic and kinetic theories of traffic flow. Based on theoretical approaches, various speed-density models (linear, parabolic, exponential) can be derived from \( dv/dk = -c \) \( v^2 = k^2 \) where parameters \( m \) and \( l \) are selected to satisfy boundary conditions such as \( v = v_j \) when \( k = 0 \) and \( k = k_s \) when \( v = v_f \). However, it has not been possible to obtain a satisfactory model based on these approaches, that gives realistically high values of \( v_s/v_j \sim 0.6 \rightarrow 0.9 \) (e.g. see TRB) associated with low values of \( k_s/k_f \) of 0.15-0.40 where \( v_s \) and \( k_s \) are the speed and density corresponding to the maximum flow rate. For example, \( q_s = 2000 \text{veh} h^{-1} \) and \( v_s = 55 \text{km} h^{-1} \) give \( k_s = q_s/v_s = 36.4 \text{veh} km^{-1} \), and using \( k_f = 10000 \text{veh} km^{-1} \) gives \( k_s/k_f \) of 0.23.

A model based on the use of vehicle spacing as a function of speed \((L_y = v)\) as a starting point (rather than a speed-density relationship) for the derivation of speed-density-flow models has been discussed in Akcelik and Akcelik and Besley.

Alternatively, a speed-flow model can be used as a starting point. For Region B, the following model form derived using exponential queue discharge flow and speed models can be used:

\[
v = v_f [1 - \frac{q_s}{v_s}] \left( \frac{k_s}{k_f} \right) \quad (\text{II})
\]

where \( v_s, q_s \) = speed (km/h) and flow rate (veh/h) in Region B of the speed-flow model

\( v_s \) = speed at maximum flow (km/h)

\( q_s \) = maximum flow rate (veh/h)

\( k_s \) = density at maximum flow (veh/km)

\( k_f \) = jam density (veh/km)

If the speed \( v_y \) is known, the flow rate in Region B \( q_y \) can be estimated from the following formula derived from Equation (11):

\[
q_y = q_s [1 - \frac{v_y}{v_s}] \left( \frac{k_s}{k_f} \right) \quad (\text{IIIA})
\]

For six signalised intersections sites in Melbourne (each representing a traffic lane with cars only), \( q_s = 1840 \text{ to } 2880 \text{veh} h^{-1}, v_s = 27 \text{ to } 56 \text{km} h^{-1}, k_s = 50 \text{ to } 68 \text{veh} km^{-1}, k_f = 0.30 \text{ to } 0.40 \text{ (km/h)} = 6.0 \text{m} v^{-1}, k_j = 167 \text{veh} km^{-1} \) were observed. For these sites, free-flow speeds were in the range \( v_f = 60 \text{ to } 80 \text{km/h} \) using the speed limit as the free-flow speed, and the ratios \( v/s \) to 0.4 to 0.8 were found.

The following model based on a time-dependent travel time function (Akcelik and Akcelik) can be used for Regions A and C (or A’ and C) of the speed-flow relationship for uninterrupted or interrupted flow conditions:

\[
v = \frac{v_f}{1 + 0.25 \frac{d_{zr}}{v_f}} \quad (\text{IIIB})
\]

where \( v \) = travel speed in km/h (\( v = v_f \) for uninterrupted flow, \( v = v_j \) for interrupted flow)

\[v_{of} = \text{zero-flow travel speed in km/h (} v_{of} = v_j \text{ for uninterrupted flow)}
\]

\( T_f \) = peak flow (analysis) period in hours (e.g. \( T_f = 0.25 \) to 2.0 h)

\( Q \) = capacity in veh/h (\( Q = q_s \) for uninterrupted flow)

\( z \) = \( x - 1 \)

\( x = q_s/Q \) (\( q_s \) is the demand flow rate; for uninterrupted flows, \( x = q_s/q_s \)).

\( m \) = a delay parameter

The slope of the speed-flow curve in Regions A and C (or A’ and C) is determined by the delay parameter \( m \). This slope indicates the rate of change of delay, i.e. the difference between the zero-flow travel time and the travel time at a given flow rate. For uninterrupted conditions, this delay is due to vehicle interactions within the traffic stream. For interrupted facilities, the delay also includes delays at traffic interruptions. The delay parameter \( m \) is related to parameter \( J_f \) used in Akcelik’s formula \( m = 8J_f \). Various values of \( J_f \) for different traffic facilities are given in Akcelik, indicating delay parameters in the range from \( m = 0.8 \) to \( 1.6 \) for a freeway to \( m = 1.3 \) to \( 1.6 \) for an interrupted facility with high friction. These values are not based on real-life data. They were constructed using various delay scenarios based on assumptions about the number and characteristics of delay-producing elements (intersections) with \( T_f = 1 \text{ h} \).

Where real-life data are not available, \( m \), can be calculated using estimates or measured values of capacity, zero-flow speed, speed at capacity, and duration of the analysis (peak demand) period:

\[
m = 16J_f \left( \frac{v_f}{v_0} - 1 \right) \quad (\text{IIIC})
\]

where \( v_f \) is the average travel speed at capacity (for uninterrupted traffic, \( v_0 = v_f \), \( v_0 = v_s \)).

It should also be noted that uninterrupted travel speeds in Region A estimated from Equation (12) are assumed to be the same as the speeds observed at a reference point along the road. The error involved in this assumption is negligible for low degrees of saturation, but may increase as arrival flows approach the capacity value.

A speed-flow function for interrupted traffic flow can be constructed from uninterrupted speed-flow function by calculating the zero-flow and speed at capacity \( v_{of}, v_0 \) from

\[
v_{of} = -\frac{v_j}{1 + 1.5 \frac{d_{zr}}{v_f}} \quad (\text{IIID})
\]

\[
v_0 = \frac{v_{of}}{1 + \frac{d_{zr}}{v_f}} \quad (\text{IIIE})
\]

where \( v_j \) is the interrupted zero-flow speed, \( v_{of} \) is the uninterrupted travel speed when the demand flow equals traffic capacity \( (q = Q) \), \( d_{zr} \) is the minimum (zero-flow) delay per unit distance, and \( d_{zr} \) is the delay per unit distance at capacity \( (q = Q) \). Minimum delay and capacity for various types of signalised and unsignalised intersections can be calculated using the delay models used in SIDRA.
Travel time-flow relationships

The travel time-flow relationships corresponding to Equations (11) and (12) can be obtained using the relationship:

\[ t = \frac{3600}{v} \quad \ldots (16) \]

where \( t \) is the travel time per unit distance (seconds/km), and the speed is \( v = v_s \) for Region B, \( v = v_c \) for Regions A and C (uninterrupted traffic), or \( v = v_p \) for Regions A and C (interrupted traffic). In fact, application of this to Region B observed in the literature is not relevant, but is used here to contrast it with the section travel time under congested conditions (Region C).

The travel time - flow model corresponding to Equation (12) is:

\[ t = t_{dr} + 0.25 T_r \left[ z + \sqrt{z^2 + \frac{m_x}{QT_r}} \right] \quad \ldots (17) \]

For uninterrupted flow, \( t_{dr} = t_r = 3600/v_s \), for interrupted flow, \( t_{dr} = 3600/v_p = t_r + d_u \) (see Equations (14) and (15), and other parameters are as in Equation (12). The travel time per unit distance for uninterrupted traffic under maximum flow conditions is \( t_u = 3600/v_s \).

The travel time corresponding to the congested flow speed in Region B (3600/v_s) and the section travel time in Region C for uninterrupted flow conditions (3600/v_s for \( x > 1 \)) cannot be equated.

The congested flow speed is measured at a particular reference point along the travel section. Different speed values would be measured at different reference points along the travel section due to the transient queueing characteristics (queue formation and queue discharge). This also explains the difficulty in determining the maximum flow value in real-life surveys.

The travel time predicted by the time-dependent model Equation (17) includes the average delay considering all vehicles that enter the travel section during the analysis period (\( T_r \)) including delays until the last vehicle that entered during \( T_r \) leaves the section. While the model implies time-dependence for undersaturated conditions, the effect of time-dependence is negligible for low degrees of saturation (the last vehicle is likely to leave the section during the same period). However, the effect of time-dependence increase as arrival flows approach the capacity value. For oversaturated conditions (\( x > 1 \)), time-dependent delays are the main component of travel time.

For uninterrupted traffic operating in undersaturated conditions (Region A), arrival (demand) and departure flows are equal. For demand flows above the maximum flow rate \( \left( q_s > q_o \right) \), traffic operates in Region B for the speed-flow curve, and the demand is not known at the detection point. This corresponds to oversaturated (congested) conditions where the demand can only be measured at the back of the queueing section. An approximate method for estimating the demand (derived using assumptions about density) can be expressed as follows:

\[ q'_o = \frac{v_c}{v_s} q_s \quad \ldots (18) \]

where \( q_o \) and \( v_c \) represent the conditions on the Region B curve, and \( v_s \) is the speed at maximum flow (this expression would probably underestimate the demand).
Example 1: uninterrupted traffic

Graphs of speed-flow (v-q), travel time-flow (t-q), speed-density (v-k), flow-density (q-k), and spacing-speed (L_s-v) relationships developed using the formulae given in this paper are shown in Figs 8 to 9 for the following example for an uninterrupted flow case:

\[ v_s = v_a = 100 \text{ km/h}, \quad v_e = 70 \text{ km/h} (v/v_e = 0.70), \quad Q = q_s = 2400 \text{ veh/h}, \]  
therefore \[ k_s = q_s/v_s = 2400/70 = 34.3 \text{ veh/km}, \]  
speeding at maximum flow \[ L_{sa} = 1000/34.3 = 29.2 \text{ m/veh}, \]  
jam spacing, \[ L_{ja} = 6.0 \text{ m/veh (cars only)}, \]  
jam density, \[ k_j = 1000/6.0 = 166.7 \text{ veh/km}, k_s/k_j = 34.3/166.7 = 0.206, \]  
and for the speed-flow model for Regions A and C, \[ m_s = 0.71 \]  
is calculated from Equation (13) using \[ T_s = 1 \text{ h}. \]

Example 2: interrupted traffic

Graphs of speed-flow (v-q) and travel time-flow (t-q) relationships are shown in Figs 10 and 11 for the following example for an interrupted flow case:

For mid-block (approach) section, uninterrupted flow parameters are \[ v_s = 80 \text{ km/h}, \quad v_e = 48 \text{ km/h} (v/v_e = 0.60), q_s = 2100 \text{ veh/h}, \]  
therefore \[ k_s = q_s/v_s = 2100/80 = 43.8 \text{ veh/km}, L_{sa} = 1000/43.8 = 22.9 \text{ m/veh}, \]  
jam spacing, \[ L_{ja} = 6.0 \text{ m/veh}, \]  
jam density, \[ k_j = 1000/6.0 = 166.7 \text{ veh/km}, k_s/k_j = 43.8/166.7 = 0.263, \]  
and parameter \[ m_s = 2.33 \]  
is calculated from Equation (13) using \[ T_s = 1 \text{ h}. \]

A lane of traffic controlled by fixed-time signals along a travel section of 1 km is considered. Signal parameters are \[ g = 54 \text{ sec}, \quad c = 90 \text{ sec}, \]  
saturated flow, \[ s = 2066 \text{ veh/h} (t < q_s), \]  
capacity is \[ Q = v_s/c = 1.239 \text{ veh/h}. \]  
From Akçelik and Chung,\(^a\) minimum delay is calculated as \[ d_{sa} = 7.2 \text{ sec/km}, \]  
and delay at capacity is calculated as \[ d_{ja} = 87.4 \text{ sec/km}. \]  
Using these in Equations (14) and (15), \[ v_s = 68.9 \text{ km/h}, \quad v_e = 27.0 \text{ km/h} \]  
are found, and using these in Equation (13), the delay parameter for the interrupted speed-flow model is found as \[ m_s = 10.02. \]

Concluding remarks

This paper presented an explanation of the relationships between fundamental traffic variables spacing, headway, speed, density, flow, and travel time for uninterrupted and interrupted traffic conditions. An Excel spreadsheet application has been developed to implement the formulae given in the paper.

Further work is recommended to calibrate the models presented here using real-life data from uninterrupted and interrupted traffic facilities with demand flows below and above capacity.

The speed-flow (or travel time-flow) model given in this paper is currently in use in the SIDRA software package\(^b\) for uninterrupted traffic facilities. It is an appropriate model for estimating travel times for such purposes as traffic assignment and congestion pricing as used in recent work by Luk and Hepburn\(^c\) and BTCE\(^d\).
APPENDIX

Notations and Definitions

- \( h \) (headway (seconds/veh)); the time between passage of the front ends of two successive vehicles
- \( h = \frac{1}{q} = \frac{L_o}{v} = t_s + t_d \) (in veh/sec., \( v \) in m/sec.), or \( h = 3,600q \) (in veh/h)
- \( k \) (density (concentration)); number of vehicles per unit distance
  \( k = \frac{q}{v} = \frac{1}{L_o} \) in veh/m., or \( k = \frac{1}{1,000L_o} \) in veh/km
- \( k_j \) (jam density): number of vehicles per unit distance at zero speed, i.e. for a stationary queue
  \( k_j = \frac{1}{L_{o_j}} \) in veh/m., or \( k = \frac{1}{1,000L_{o_j}} \) in veh/km
- \( k_m \) (density (veh/km) at maximum flow
  \( k_m = \frac{q}{v} = \frac{q}{v_{f_m}} \) (in veh/h and \( v \) in km/h)
- \( L_s \) (Spacing (m/veh); the headway distance as measured between the front ends of two successive vehicles (sum of space length and vehicle length)
  \( L_s = v + \frac{q}{v} = L_o + L_s \) (in veh/sec., \( v \) in m/sec.)
- \( L_{o_j} \) (Average jam spacing (m/veh), which is the sum of vehicle length and average space length for vehicles in a stationary queue
  \( L_{o_j} = L_s + L_{o_j} \)
- \( L_{o_m} \) (Average spacing (m/veh) at the maximum queue discharge flow rate of the actual traffic mix, \( q \)
- \( L_s \) (Space (gap) length (m/veh)); the following distance between two successive vehicles as measured between the back end of the leading vehicle and the front end of the following vehicle
  \( L_s = L_o - L_s = t_d \)
- \( L_{o_j} \) (Average space length for vehicles in a stationary queue (m)
- \( L_v \) (Average vehicle length (m/veh) for the actual traffic mix
  \( L_v = (1 - \rho_m) L_{o_m} + \rho_m L_{o_m} \)
- \( L_{o_m} \) (Average vehicle length for heavy vehicles (m/HV)
- \( L_{o_m} \) (Average vehicle length for light vehicles, or passenger car units (m/LV or m/PCU)
- \( P_{HV} \) (Proportion of heavy vehicles (HV/s) in the stream, e.g. \( p_{HV} = 0.05 \) means 5 percent HVs and 95 percent LVS (light vehicles, or pcus)
- \( q \) (Flow rate (veh/sec. or veh/h); the number of vehicles per unit time passing (arriving or departing) a reference point along the road
- \( a_d \) (Arrival (demand) flow rate (veh/h), i.e. the number of vehicles per unit time as measured at a point upstream of the queueing section
- \( q_s \) (Maximum flow rate for uninterrupted traffic (veh/h); maximum queue discharge flow rate at a signal stop-line
- \( q_f \) (Flow rate under congested flow conditions, observed at a reference point along the road (veh/h) or veh/sec.) flow rate after the saturated part of the green period at a signal stop-line
- \( q_a \) (Flow rate for uninterrupted traffic (veh/h); departure flow rate during the unsaturated part of the green period at a signal stop-line (\( q_a = q_s; \) departure flow rate = arrival flow rate)
- \( Q \) (Capacity (veh/h); maximum arrival flow rate that can be serviced under prevailing flow conditions at a traffic facility; for uninterrupted traffic, \( Q = q_a \)
- \( t_s \) (Average saturation (queue discharge) flow rate during the effective green period at a signal stop-line (veh/h)
- \( t_e \) (Travel time per unit distance (sec./km)
  \( t_e = \frac{3,600}{v} \) (in km/h)
- \( t_r \) (Free-flow travel time per unit distance of uninterrupted traffic; average travel time under zero-flow conditions (can be approximated by the speed limit) (sec./km)
  \( t_r = \frac{3,600}{v_f} \) (in km/h)
- \( v \) (Gap time (sec./veh); the time between the passage of the back end of leading vehicle and the front end of the following vehicle as measured at a reference point along the road; this is the time taken to travel the space (gap) length, \( L_s \)
- \( t_s = L_s/v = h - t_d \) (in m/sec.)
- \( t_e \) (Travel time per unit distance under maximum flow conditions for uninterrupted traffic (\( q = q_s \)) (sec./km)
  \( t_e = \frac{3,600}{v_f} \) (in km/h)
- \( v_{f_m} \) (Zero-flow travel time per unit distance for interrupted traffic; average travel time under zero-flow conditions including the approach free-flow travel time, \( t_f \) plus total minimum delay at traffic interruptions (sec./km)
  \( v_{f_m} = \frac{3,600}{v_f} \) (in km/h)
- \( v_t, v_{f_m} \) (Vehicle passage time (sec./veh); time it takes for the vehicle length (from front end to back end) to pass a reference point at speed \( v \) (m/sec.)
  \( t_s = L_s/v \)
- \( v, v_{f_m} \) (Speed (m/sec. or km/h); distance travelled per unit time
  \( v = h/q \)
- \( v_{f_m} \) (Average approach speed (km/h) measured at a point upstream of the queueing section; average cruise speed at arrival flow rate \( q_a \)
- \( v_{f_m} \) (Average speed for uninterrupted traffic including the effect of delays at traffic interruptions (km/h)
- \( v_f \) (Free-flow speed for uninterrupted traffic (km/h)
  \( v_f = \frac{3,600}{t_s(t_s)_{max}} \) (in sec./km)
- \( v_f \) (Speed at maximum flow (km/h); speed corresponding to the maximum queue discharge flow rate, \( q_s \) at a signal stop-line
- \( v_f \) (Zero-flow speed for interrupted traffic (km/h)
  \( v_f = \frac{3,600}{t_s(t_s)_{in sec.}} \) (in sec./km)
- \( v_f \) (Average travel speed when the demand flow rate equals the capacity for interrupted traffic (km/h)
- \( v_f \) (Speed under congested (forced-flow) conditions observed at a reference point along the road (km/h); departure speed during the saturated part of the green period at a signal stop-line
- \( v_f \) (Average travel speed (km/h or m/sec.) for uninterrupted traffic (\( v_f = v_{f_m} \))
- \( v_f \) (Degree of saturation, i.e. the ratio of arrival (demand) flow rate to capacity; for uninterrupted traffic, \( v_f = q_a/q_s \)

REFERENCES