Akcelik & Associates Pty Ltd

TECHNICAL REPORT

Speed-Flow Models for Uninterrupted Traffic Facilities

Author:

Rahmi Akçelik

December 2003

Speed-Flow Models for Uninterrupted Traffic Facilities

CONTENTS

1st version (August 2002)

2nd version (November 2002): Three new references to the use of Akçelik's function included.

3rd version (December 2003): Figures (2.1) and (2.2) updated. Equations (5.7) to (5.9) fixed.

Correspondence: Rahmi Akçelik
Director, Akcelik & Associates Pty Ltd,
P O Box 1075 G, Greythorn Victoria, Australia 3104
Tel: +61 3 9857 9351, Fax: + 61 3 9857 5397
rahmi@akcelik.com.au

www.aatraffic.com

1 INTRODUCTION

Recently, the aaSIDRA (Akcelik and Associates 2002) speed-flow relationships for uninterrupted movements were calibrated against the US Highway Capacity Manual (TRB 2000) speed-flow models for *basic freeway segments* (Chapters 13 and 23) and *multilane highways* (Chapters 12 and 21). The purpose of this exercise was to adopt the HCM 2000 Level of Service method in aaSIDRA. This effort indicated various unexpected characteristics of the HCM 2000 models. This report presents the results of an investigation of related issues.

The HCM speed-flow models are given for undersaturated conditions on these uninterrupted-flow facilities. Multilane highway analysis is qualified as relevant to "rural and suburban" highways. Urban roads with signalised intersections spaced at 3 km or more fall into this category, otherwise they are classified as *urban streets* (Chapters 10 and 15).

The uninterrupted travel speed concept is explained in *Section 2*, the HCM speed-flow models are described in *Section 3,* issues of concern are discussed in *Section 4*, and a proposed solution based on the use of "Akcelik's" speed-flow function (Akçelik 1991, 1996; Akçelik, Roper and Besley 1999; Akcelik and Associates 2002) is presented in *Section 6* after the Akçelik function is described in *Section 5*.

Section 7 presents the speed-flow models for the running time component of the HCM travel speed model for urban streets derived using Akçelik's speed-flow function.

2 UNINTERRUPTED TRAVEL SPEED CONCEPT

The *average uninterrupted travel speed* can be expressed as:

$$v_u = 3600 / t_u = 3600 / (t_f + d_{tu}) \tag{2.1}$$

where

v_u = uninterrupted travel speed at a given flow rate (km/h),

t_u = uninterrupted travel time per unit distance, $t_u = t_f + d_{tu}$ (seconds/km),

d_{tu} = traffic delay (uninterrupted travel delay) per unit distance (seconds/km),

t_f = free-flow travel time per unit distance (seconds/km):

$$t_f = 3600 / v_f \tag{2.2}$$

v_f = free-flow speed (km/h).

Definitions of free-flow and uninterrupted travel speed are shown in a time-distance diagram in *Figure 2.1* where L_t is the travel distance (km).

Speed-flow relationships for uninterrupted movements can be explained with the help of *Figure 2.2* which also shows the associated travel time - flow and traffic delay - flow relationships.

In *Figure 2.2*, Region A represents undersaturated conditions with arrival flows below capacity ($q_a \le Q$) which are associated with uninterrupted travel speeds, v_u between v_f and v_n ($v_f \ge v_u \ge v_n$) where v_f is the free-flow speed and v_n is the speed at capacity. With increasing flow rate in Region A, speeds are reduced below the free-flow speed due to traffic delays resulting from interactions between vehicles.

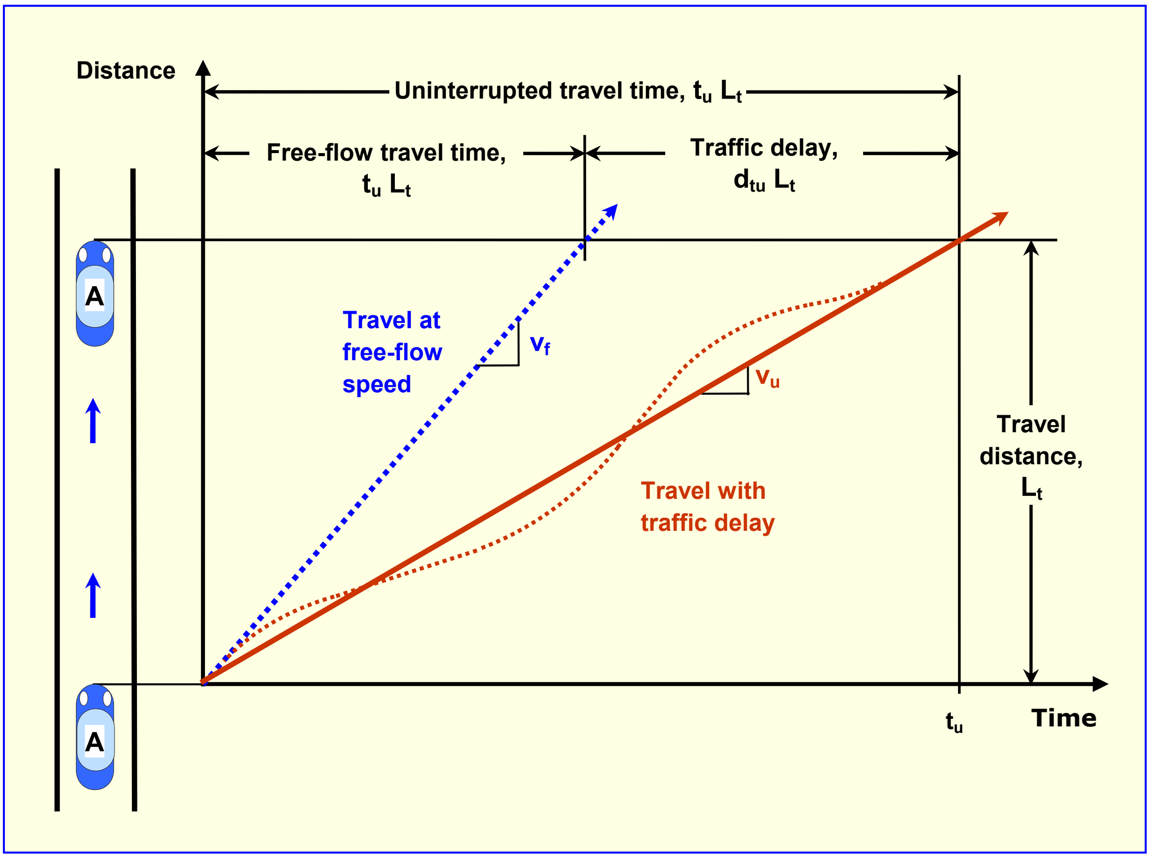

Figure 2.1 - Definition of free-flow and uninterrupted travel speed

Region B in *Figure 2.2* represents the forced (congested) flow conditions with flow rates reduced below capacity ($q < Q$) which are associated with further reduced speeds ($v < v_n$) as observed at a reference point along the road. In this region, flow rates (q) are reduced flow rates due to forced flow conditions, not demand flow rates (q_a).

Region C represents oversaturated conditions, i.e. arrival (demand) flow rates above capacity ($q_a > Q$) which are associated with reduced travel speeds ($v < v_n$) observed by travel through the total section (along distance L_t), e.g. by an instrumented car. In this case, the flow represents the demand flow rate which can exceed the capacity value as measured at a point upstream of the queuing section.

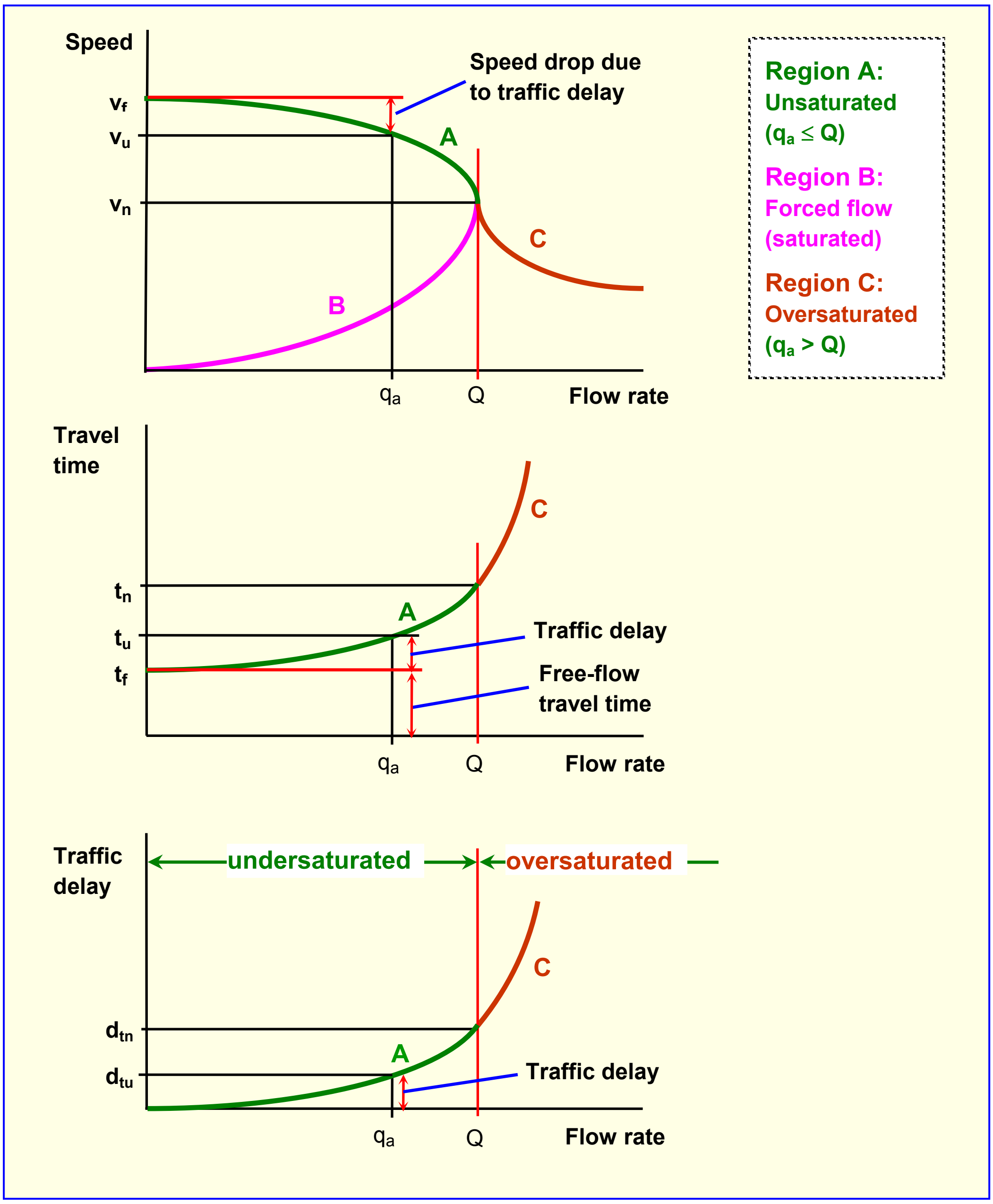

Figure 2.2 - Speed, travel time and delay as a function of flow rate for uninterrupted traffic streams

3 HCM SPEED-FLOW MODELS

The HCM describes speed-flow models for four classes of basic freeway segments and four classes of multilane highways as summarised in *Table 3.1* and shown in *Figures 3.1 and 3.2*. The model parameters summarised in *Table 3.1* represent conditions for a single-lane traffic stream consisting of passenger cars only. *Figures 3.1 and 3.2* show the speed-flow curves for flow rates up to capacity as given in the HCM. These models are based on NCHRP Projects 3-33 (Reilly, Harwood, Schoen and Holling 1990) and 3-45 (Schoen, May, Reilly and Urbanik 1995).

The main parameters describing the HCM speed-flow models are:

free-flow speed, v_f (km/h),
capacity, Q (veh/h), and
density at capacity, k_n (veh/km).

Other important parameters that can be determined once the above parameters are known are:

speed at capacity, $v_n = Q / k_n$ (km/h),
speed ratio, v_n / v_f,
free-flow travel time, $t_f = 3600 / v_f$ (s/km),
travel time at capacity, $t_n = 3600 / v_n$ (s/km),
traffic delay at capacity, $d_{tn} = t_n - t_f$ (s/km),
average headway at capacity, $h_n = 3600 / Q$ (s/veh), and
average spacing at capacity, $L_{hn} = 1000 / k_n$ (m/veh).

Also of interest are:

flow limit for free-flow speed, q_o ($v = v_f$ for flow rate $q_a \le q_o$) (veh/h), and
degree of saturation below which speed equals free-flow speed, $x_o = q_o / Q$.

Table 3.1

Free-flow speed, capacity, density and other parameters for speed-flow relationships for four classes of BASIC FREEWAY SEGMENTS and four classes of MULTILANE HIGHWAYS given in HCM 2000

	Basic Freeway Segments				**Multilane Highways**			
Facility class	**1**	**2**	**3**	**4**	**1**	**2**	**3**	**4**
Free-flow speed, v_f (km/h)	**120**	**110**	**100**	**90**	**100**	**90**	**80**	**70**
Capacity, Q (veh/h)	**2400**	**2350**	**2300**	**2250**	**2200**	**2100**	**2000**	**1900**
Density at capacity, k_n (veh/km)	**28.0**	**28.0**	**28.0**	**28.0**	**25.0**	**26.0**	**27.0**	**28.0**
Speed at capacity, v_n (km/h)	**85.7**	**83.9**	**82.1**	**80.4**	**88.0**	**80.8**	**74.1**	**67.9**
Speed ratio, v_n / v_f	**0.71**	**0.76**	**0.82**	**0.89**	**0.88**	**0.90**	**0.93**	**0.97**
Free-flow travel time, t_f (s/km)	30.0	32.7	36.0	40.0	36.0	40.0	45.0	51.4
Travel time at capacity, t_n (s/km)	42.0	42.9	43.8	44.8	40.9	44.6	48.6	53.1
Traffic delay at capacity, $d_{tn} = t_n - t_f$ (s/km)	12.0	10.2	7.8	4.8	4.9	4.6	3.6	1.6
Average headway at capacity, h_n (s/veh)	1.500	1.532	1.565	1.600	1.636	1.714	1.800	1.895
Average spacing at capacity, L_{hn} (m/veh)	35.7	35.7	35.7	35.7	40.0	38.5	37.0	35.7
Flow limit for free-flow speed, q_o (veh/h)	1300	1450	1600	1750	1400	1400	1400	1400
Degree of saturation at q_o ($x_o = q_o$ / Q)	0.54	0.62	0.70	0.78	0.64	0.67	0.70	0.74

Data given in this table represent conditions for passenger cars only in a single lane.

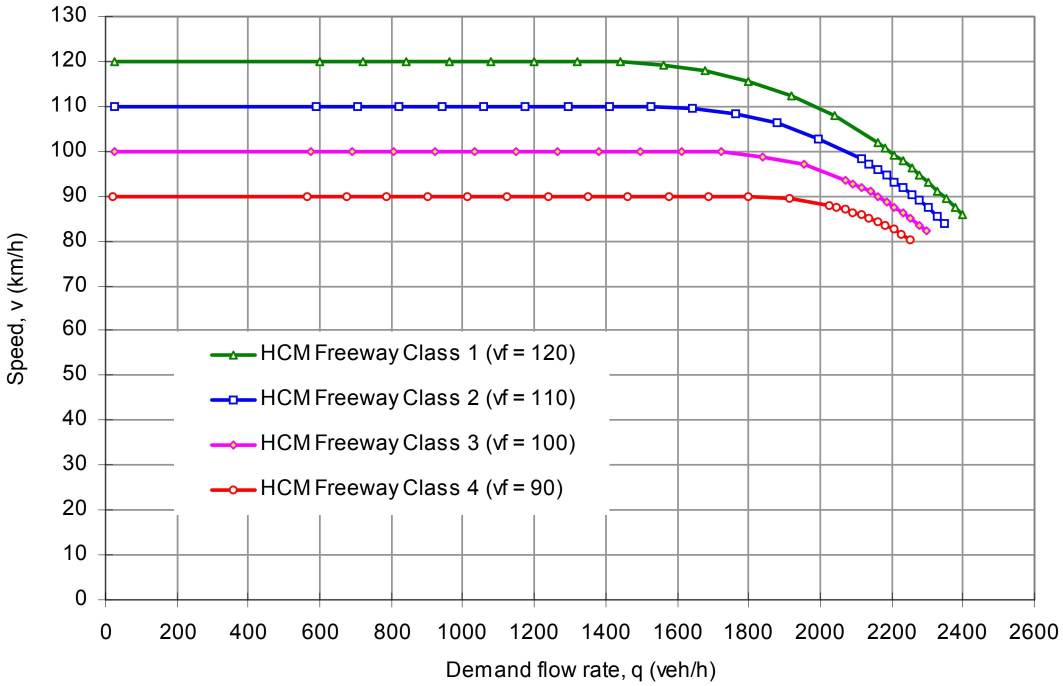

Figure 3.1 - HCM 2000 speed-flow models for four classes of BASIC FREEWAY SEGMENTS (based on functions given in HCM Chapter 23, Exhibit 23-3)

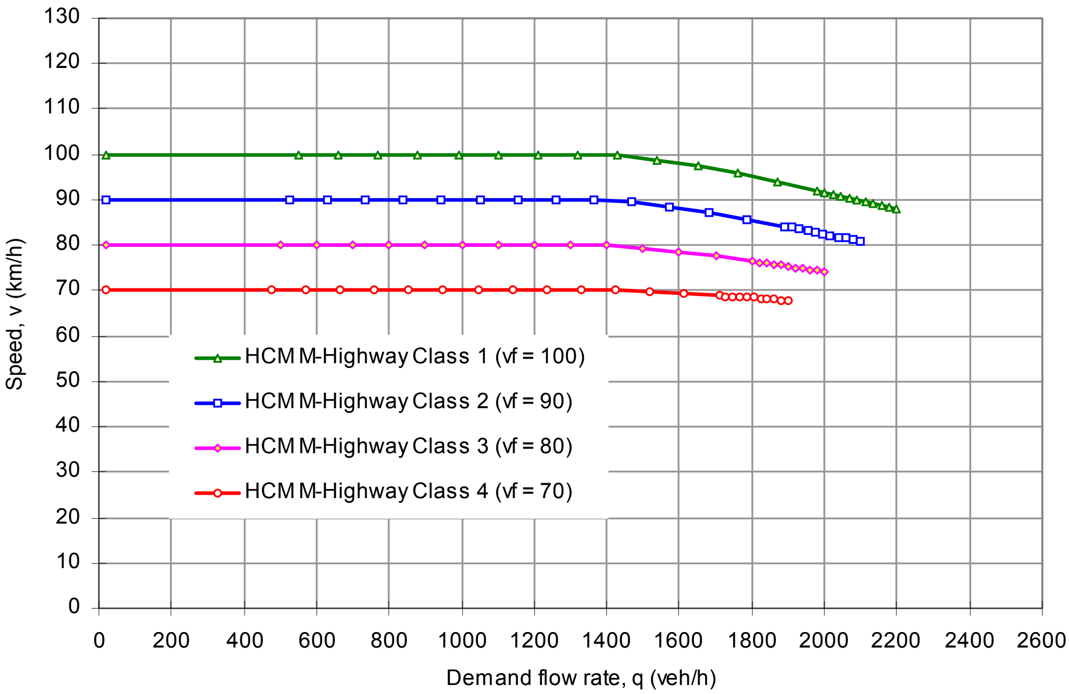

Figure 3.2 - HCM 2000 speed-flow models for four classes of MULTILANE HIGHWAYS (based on functions given in HCM Chapter 21, Exhibit 21-3)

The HCM defines the free-flow speed as the mean speed of passenger cars measured under low to moderate flow conditions. These flow conditions are qualified as up to 1400 veh/h per lane for multilane highways (Chapter 21), and up to 1300 veh/h per lane for basic freeway segments (Chapter 23).

The HCM provides methods to estimate the free-flow speed as a function of the physical (geometric) characteristics of the facility. According to the method for basic freeway segments, the free-flow speed decreases with decreasing lane width, decreasing number of lanes, decreasing lateral clearance, and increasing interchange density. According to the method for multilane highways, the free-flow speed decreases with decreasing lane width, decreasing lateral clearance and increasing access point (intersection and driveway) density, and is reduced for undivided highways. Thus, a higher free-flow speed represents a higher-quality facility generally. This is consistent with higher capacities specified for facilities with higher free-flow speeds, and higher capacities for freeways compared with multilane highways with the same free-flow speed (e.g. Freeway Class 3 vs Highway Class 1).

After the flow rate exceeds a limit flow rate (q_o), speeds decrease below the free-flow speed with increasing flow rate towards the speed at capacity due to an increasing level of interactions between vehicles as seen in *Figures 3.1 and 3.2*. From *Equation 2.1*, this speed drop corresponds to the *traffic delay*:

$$\begin{aligned} d_{tu} &= t_u - t_f \\ &= (3600 / v_u) - (3600 / v_f) \end{aligned} \tag{3.1}$$

where

d_{tu} = traffic delay per unit distance (seconds/km),

t_u = uninterrupted travel time per unit distance at a given flow rate (seconds/km),

t_f = free-flow travel time per unit distance (seconds/km),

v_u = uninterrupted travel speed at a given flow rate (km/h), and

v_f = free-flow speed (km/h).

Figures 3.3 and 3.4 show traffic delay as a function of degree of saturation ($x = q_a / Q$ where q_a is the demand flow rate, Q is the capacity) for basic freeway segments and multilane highways. These graphs help to identify some characteristics of the HCM speed-flow models which are of concern as discussed in the following section.

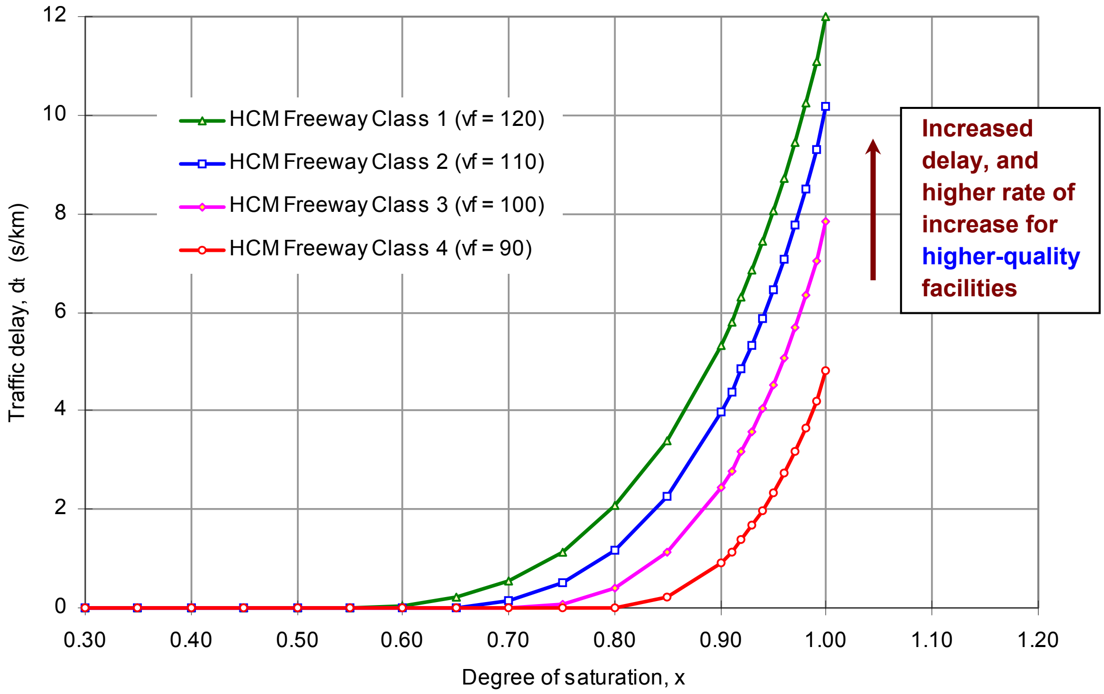

Figure 3.3 - Traffic delay graphs corresponding to HCM 2000 speed-flow models for BASIC FREEWAY SEGMENTS

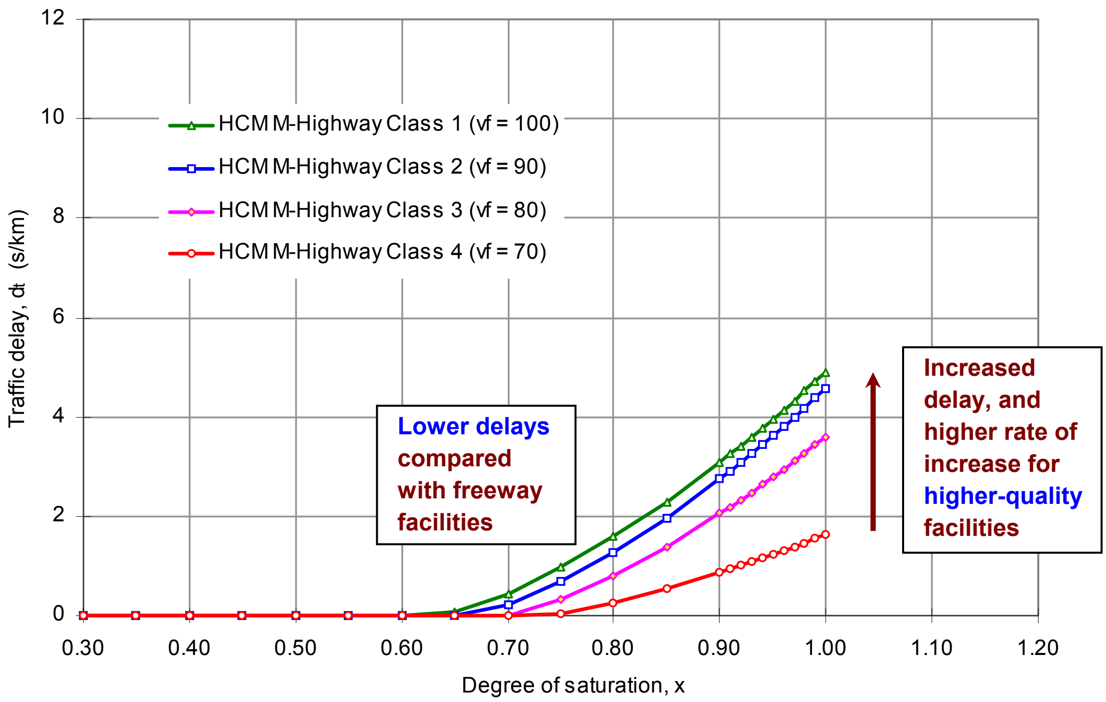

Figure 3.4 - Traffic delay graphs corresponding to HCM 2000 speed-flow models for MULTILANE HIGHWAYS

4 ISSUES

The HCM speed-flow models for both basic freeway segments and multilane highways indicate some features that do not appear to be consistent with expected traffic flow characteristics related to in-stream vehicle interaction and queuing considerations.

The HCM models suggest that the speed ratio (v_n / v_f) decreases with increasing free-flow speed, i.e. it is lower for higher-quality facilities. This ratio determines the sharpness of the speed-flow function, i.e. how quickly the speed drops with increased flow rate. As stated in HCM Chapter 13, "the higher the free-flow speed, the greater the drop in speed as flow rates move towards capacity". Thus, the HCM speed-flow models suggest that the rate of reduction in speed with increased flow is greater, and therefore traffic delays increase at a faster rate, and in fact the traffic delays are larger, for higher-quality facilities (see *Table 3.1* for traffic delays at capacity, and *Figures 3.3 and 3.4* for traffic delay - flow graphs for freeways and multilane highways).

This characteristic of the HCM speed-flow models is in contrast with travel-time - flow models for different road classes used for transport planning purposes (Akçelik 1991, 1996). Higher traffic delays for higher-quality facilities do not appear to be consistent with "queuing" mechanisms inherent to in-stream vehicle interactions (Blunden 1971, 1978; Davidson 1966). It is expected that such features as wider lanes, a larger number of lanes, more lateral clearance and lower interchange or access point density represent higher-quality facilities with lower frequency (intensity) of delay-producing elements and situations.

On this basis, the following features of the HCM speed-flow models could also be questioned:

(i) that the non-zero traffic delay starts, i.e. the speed starts dropping below the free-flow speed, at a lower degree of saturation (x_o) for higher-quality facilities both in the case of freeways and multilane highways (see *Figures 3.1 to 3.4*);

(ii) that traffic delays are generally lower for multilane highways compared with basic freeway segments (*Figure 3.4 vs Figure 3.3*);

(iii) that the speeds at capacity (v_n) for freeways with v_f = 100 and 90 km/h are smaller than the corresponding speeds at capacity for multilane highways; this means that the traffic delay at capacity is larger for freeways with the same free flow speed while freeway lanes have higher capacities.

Furthermore, the HCM recommends the same value of *density* at capacity (k_n = 28 veh/km) for all classes of basic freeway segments while the value of density at capacity increases with decreasing free-flow speed for multilane highways (k_n = 25 veh/km for v_f = 100 km/h to k_n = 28 veh/km for v_f = 70 km/h) as seen in *Table 3.1*. The implication of this can be understood better by considering the relationships between corresponding average vehicle spacing, average headway and speed values at capacity. Vehicle spacing, L_h (m/veh) is the distance between the front ends of two successive vehicles in the same traffic lane. Headway, h (s/veh) is the time between passage of the front ends of two successive vehicles. The average vehicle spacing and headway at capacity, L_{hn} and h_n, can be determined from:

$$L_{hn} = 1000 / k_n \tag{4.1}$$

$$h_n = 3600 / Q \tag{4.2}$$

where k_n is the density at capacity (veh/km) and Q is the capacity (veh/h).

The average vehicle spacing and headway at capacity, L_{hn} (m/veh) and h_n (s/veh), are related through:

$$v_n = 3.6\, L_{hn} / h_n \tag{4.3}$$

where v_n is the speed at capacity (km/h).

The values of L_{hn} and h_n for all freeway and multilane highway classes are given in *Table 3.1*. The relationships between average vehicle spacing, average headway and speed values at capacity are shown in *Figures 4.1 and 4.2*.

The use of the same density at capacity for all freeway classes implies the same vehicle spacing at different speeds. On the other hand, the multilane highway models imply increasing vehicle spacings with increasing speeds at capacity, which is more consistent with expected safe driver behaviour. Relationships between spacing and speed for freeway and multilane highway classes implied by the HCM speed-flow models are given in *Figure 4.3*.

For the purpose of comparison with the HCM relationships, *Figures 4.4 and 4.5* give the speed-flow and spacing-speed relationships observed on Eastern Freeway in Melbourne, Australia (Akçelik, Roper and Besley 1999). Estimated speed-flow and spacing-speed relationships are based on "Model 4+5" described in detail in Akçelik, Roper and Besley (1999). In *Figures 4.4 and 4.5,* broken lines are for saturated conditions when the estimated lower-bound jam spacing of 10 m/veh is used, and the solid lines for saturated conditions are derived using estimated mean jam spacing of 15 m/veh. The model parameters estimated for this site (based on data aggregated for 5-minute intervals) were as follows:

Free-flow speed, v_f	= 101 km/h
Capacity, Q	= 2500 veh/h
Density at capacity, k_n	= 27.8 veh/km
Speed at capacity, v_n	= 90 km/h
Speed ratio, v_n / v_f	= 0.89
Free-flow travel time, t_f	= 35.6 s/km
Travel time at capacity, t_n	= 40.0 s/km
Traffic delay at capacity, d_{tn}	= 4.4 s/km
Average headway at capacity, h_n	= 1.440 s/veh
Average spacing at capacity, L_{hn}	= 36.0 m/veh

Various methods to revise the HCM speed-flow models to achieve expected characteristics discussed in this section have been investigated. The results based on a simple solution using "Akçelik's" speed-flow function are given in *Section 6* after the Akçelik function is described in *Section 5*.

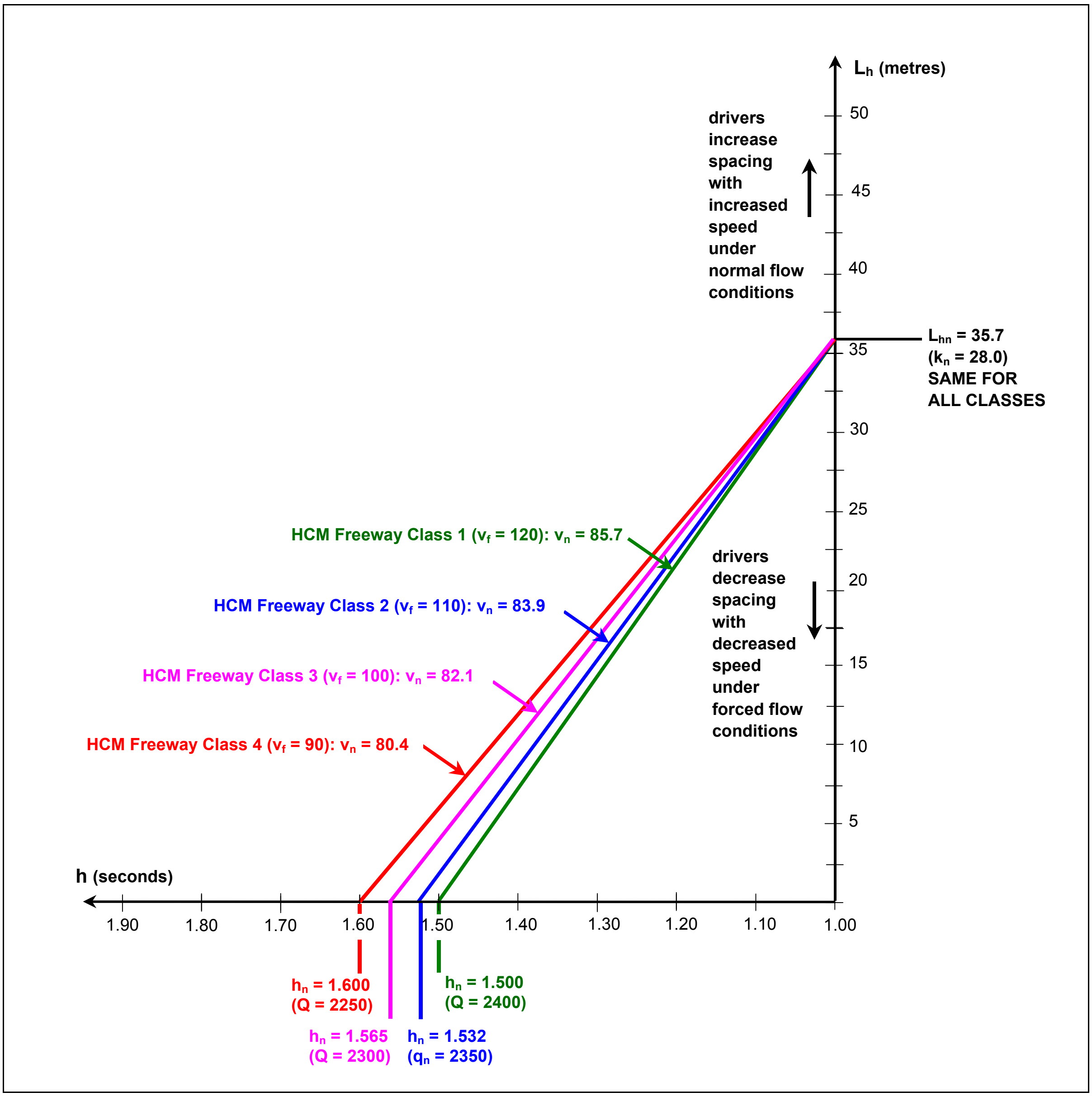

Figure 4.1 - Relationships between spacing, headway and speed at capacity corresponding to HCM 2000 speed-flow models for BASIC FREEWAY SEGMENTS

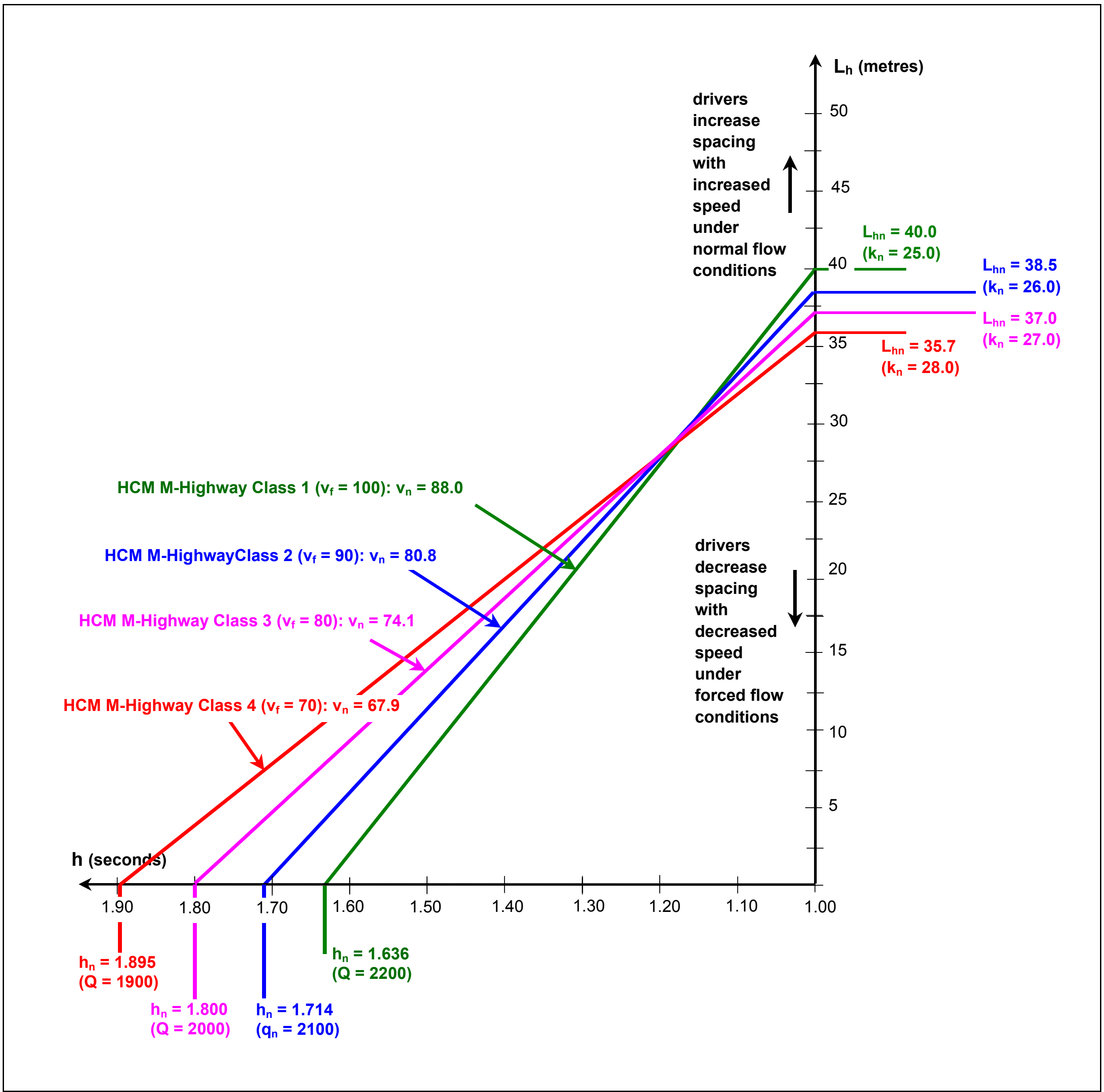

Figure 4.2 - Relationships between spacing, headway and speed at capacity corresponding to HCM 2000 speed-flow models for MULTILANE HIGHWAYS

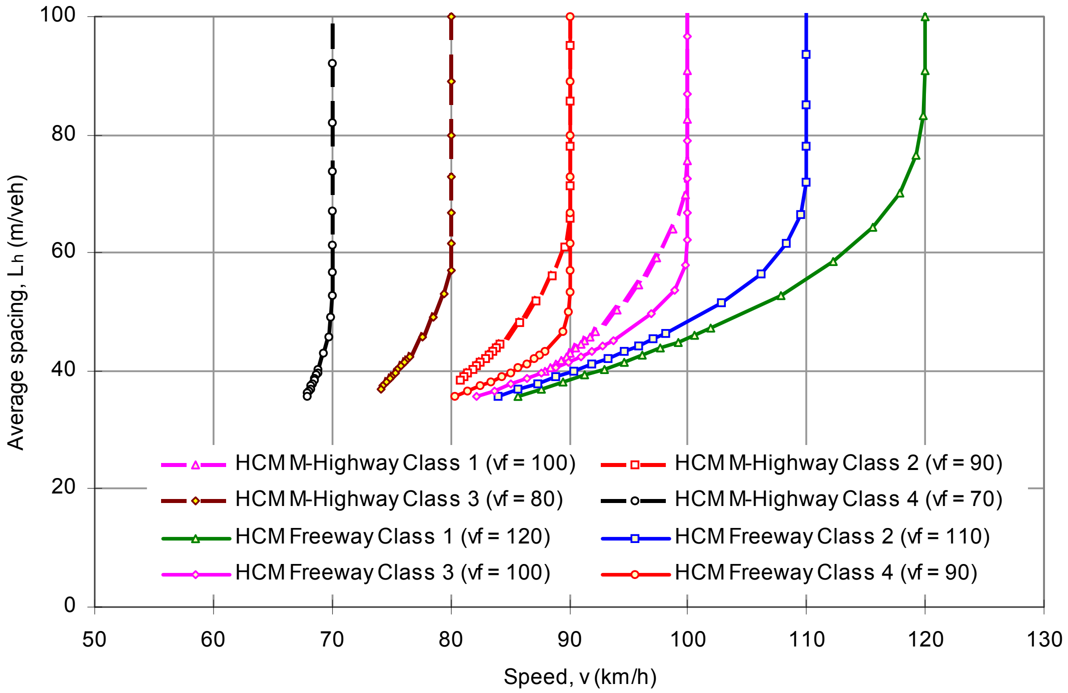

Figure 4.3 - Relationship between average vehicle spacing and speed corresponding to HCM 2000 speed-flow models for BASIC FREEWAY SEGMENTS and MULTILANE HIGHWAYS

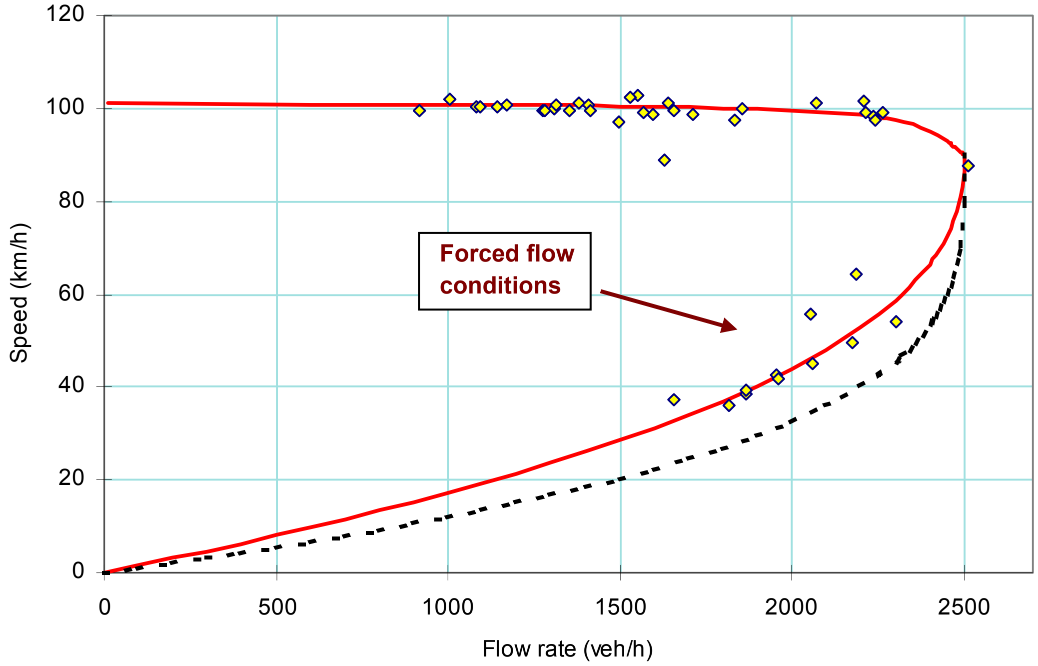

Figure 4.4 - Measured and estimated speed as a function of flow rate for the Eastern Freeway in Melbourne, Australia (including forced flow conditions)

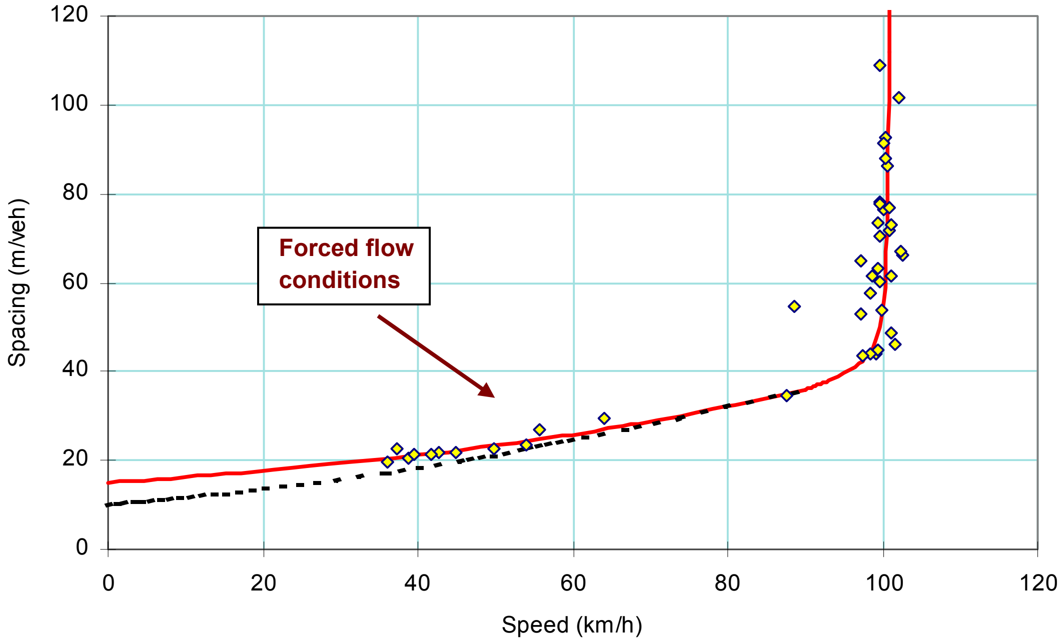

Figure 4.5 - Measured and estimated spacing as a function of speed for the Eastern Freeway in Melbourne, Australia (including forced flow conditions)

5 A TIME-DEPENDENT SPEED-FLOW FUNCTION

A time-dependent speed-flow model developed previously has been used in various applications successfully, and has been referred to as *Akcelik's function* in the literature (Akçelik 1991, 1996; Akçelik, Roper and Besley 1999; Akcelik and Associates 2002; Dowling and Alexiadis (1997), Dowling, Singh and Cheng 1998, Nakamura and Kockelman 2000, Singh (1999), Sinclair Knight Merz 1998). This function estimates average speeds corresponding to Regions A and C in *Figure 2.2*. It is based on queuing theory concepts, providing a smooth transition between a steady-state queuing delay function for undersaturated conditions and a deterministic delay function for oversaturated conditions. Thus, it allows for estimation of "travel speed" for both undersaturated and oversaturated conditions.

A general formulation of the model is given below to apply to both uninterrupted and interrupted facilities. In the case of uninterrupted facilities, use speed $v = v_u$ and delay $d_t = d_{tu}$ (traffic delay). In the case of interrupted facilities, replace the free-flow speed (v_f) by zero-flow speed (v_o) which includes minimum delay at zero flow conditions.

The most general form of the function using the x_o parameter and allowing for initial queued demand (N_i) can be expressed as follows:

$$v = v_f / \{1 + 0.25\, v_f\, T_f\, [z + (z^2 + 8\, k_d\, (x - x_o) / (Q\, T_f) + 16\, k_d\, N_i / (Q\, T_f)^2)^{0.5}]\} \qquad \textit{for } x' > x_o \qquad (5.1)$$

$$= v_f \qquad \textit{for } x' \leq x_o$$

where

v = speed at a given degree of saturation x (km/h),

v_f = free-flow speed (speed at $x = 0$) (km/h),

T_f = duration of the analysis period (h) ($T_f = 0.25$ for HCM models),

k_d = delay parameter, or $1 - k_d$ = quality of service parameter (symbol J_D was used in Akçelik (1991) instead of k_d used here),

x = degree of saturation:

$$x = q_a / Q \qquad (5.2a)$$

x' = degree of saturation adjusted to take into account the initial queued demand effects:

$$x' = x + [N_i / (Q\, T_f)] \qquad (5.2b)$$

x_o = degree of saturation below which the traffic delay is zero (speed equals free-flow speed),

z = a parameter calculated as:

$$z = x - 1 + 2\, N_i / (Q\, T_f) \qquad (5.3)$$

where q_a is the arrival (demand) flow rate (veh/h) and Q is the capacity (maximum flow rate), and N_i is the initial queued demand observed at the start of the analysis period (vehicles).

The delay parameter, k_d is a constant and is related to other parameters through:

$$k_d = 2\, Q\, ((v_f / v_n) - 1)^2 / (v_f^2\, T_f\, (1 - x_o)) \qquad (5.4)$$

where v_n (km/h) is the speed at maximum flow rate.

The following simpler function form is obtained for the condition when there is no initial queued demand (i.e. the previous analysis interval is undersaturated, $x \le 1.0$), $N_i = 0$ in *Equation (5.1)*:

$$v = v_f / \{1 + 0.25\, v_f\, T_f\, [z + (z^2 + 8\, k_d\, (x - x_o) / (Q\, T_f)\,)^{0.5}]\} \qquad \text{for } x > x_o \qquad (5.5)$$

$$= v_f \qquad \text{for } x \le x_o$$

where $z = x - 1$.

The simplest function form is obtained using $x_o = 0$ in *Equation (5.5)*:

$$v = v_f / \{1 + 0.25\, v_f\, T_f\, [z + (z^2 + 8\, k_d\, x / (Q\, T_f)\,)^{0.5}]\} \qquad (5.6)$$

The travel time - flow functions corresponding to *Equations 5.1, 5.5 and 5.6* are expressed as:

$$t = t_f + 900\, T_f\, \{z + [z^2 + 8\, k_d\, (x - x_o) / (Q\, T_f) + 16\, k_d\, N_i / (Q\, T_f)^2]^{0.5}\} \qquad \text{for } x' > x_o \qquad (5.7)$$

$$= t_f \qquad \text{for } x' \le x_o$$

With no initial queued demand, $N_i = 0$:

$$t = t_f + 900\, T_f\, \{z + [z^2 + 8\, k_d\, (x - x_o) / (Q\, T_f)]^{0.5}\} \qquad \text{for } x' > x_o \qquad (5.8)$$

$$= t_f \qquad \text{for } x' \le x_o$$

Using, $x_o = 0$:

$$t = t_f + 900\, T_f\, \{z + [z^2 + 8\, k_d\, x / (Q\, T_f)]^{0.5}\} \qquad (5.9)$$

where

t = travel time per unit distance at a given degree of saturation x (seconds/km),

t_f = free-flow travel time per unit distance (travel time at $x = 0$) (seconds/km),

and other parameters are as in *Equations 5.1, 5.5 and 5.6.*

For interrupted facilities, replace the free-flow travel time (t_f) by zero-flow travel time (t_o):

$$t_o = t_f + d_m \qquad (5.10)$$

where d_m = minimum delay per unit distance at zero flow conditions (use $d_m = 0$ for uninterrupted facilities).

Zero-flow speed, v_o (km/h) can be calculated from:

$$v_o = 3600 / t_o \qquad (5.11)$$

For through movements at signalised intersections:

$$d_m = 0.5\, r\, (1 - u) \qquad (5.12a)$$

where r = effective red time (s), u = green time ratio, u = g / c where g = effective green time (s), c = cycle time (s).

For through movements at roundabouts and sign-controlled intersections:

$$d_m = 3600 / Q + d_{ig} \qquad (5.12b)$$

where Q = capacity (veh/h) and d_{ig} = intersection geometric delay.

The basis of the above equations is explained below considering undersaturated and oversaturated conditions.

The function is based on the following *steady-state* queuing delay expression for undersaturated conditions at a traffic interruption point (e.g. an intersection):

$$d_q = k_{di} (x - x_o) / [Q (1 - x)] \quad (5.13)$$

where k_{di} is the *delay parameter* for a single delay-producing element, and Q is the capacity. The delay parameter depends on the level of randomness (or regularity) of the arrival and service processes. The special cases of parameter k_{di} for the case of random arrivals are $k_{di} = 1$ for exponential service times and $k_{di} = 0.5$ for regular (constant) service times. The former has been used for unsignalised intersections, and the latter for signalised intersections.

The travel time along a road section can be expressed explicitly as the sum of zero-flow (minimum) travel time and total queuing delay ($d_t = \Sigma\, d_q$) along the road section:

$$t = t_o + d_t \quad (5.14)$$

As an approximation to the real value of $\Sigma\, d_q$, the delay parameter k in *Equation (5.13)* can be replaced by $k_d = p\, k_{di}$ where p is the intensity of delay-producing elements along the travel section (e.g. the number of intersections per unit distance):

$$p = n / L_t \quad (5.15)$$

where n = number of delay-producing elements along the travel section, and L_t = travel distance (km).

If there are different types of delay elements on the road section, then k_d could be obtained as some form of weighted average of delay parameters for individual delay elements (k_{di}).

Thus, the steady-state expression for travel delay per unit distance (without the minimum delay component which is included in the zero-flow travel time) is:

$$d_t = k_d\ (x - x_o) / [Q (1 - x)] \quad (5.16)$$

where the delay parameter is $k_d = n\, k_{di} / L_t$.

The time-dependent form of the travel delay, as seen from the second term of *Equations 5.7 to 5.9*, is obtained from the steady-state expression (*Equation 5.16*) and the deterministic delay expression for oversaturated conditions (explained below) by using the well-known coordinate transformation method.

Equations 5.1 and 5.8 include the effect of an *initial queued demand* (N_i) which may exist if the previous flow period is oversaturated as shown in *Figure 5.1*. The *initial queued demand* is the *residual queued demand* at the end of the previous oversaturated flow period.

The oversaturation delay is defined as the average delay to vehicles *arriving* during the current flow period. As seen in *Figure 5.1*, some of these vehicles may depart after the end of the flow period. It is also seen in *Figure 5.1* that the deterministic delay component of this period associated with delay to vehicles arriving during the previous period is excluded in the derivation of the formula.

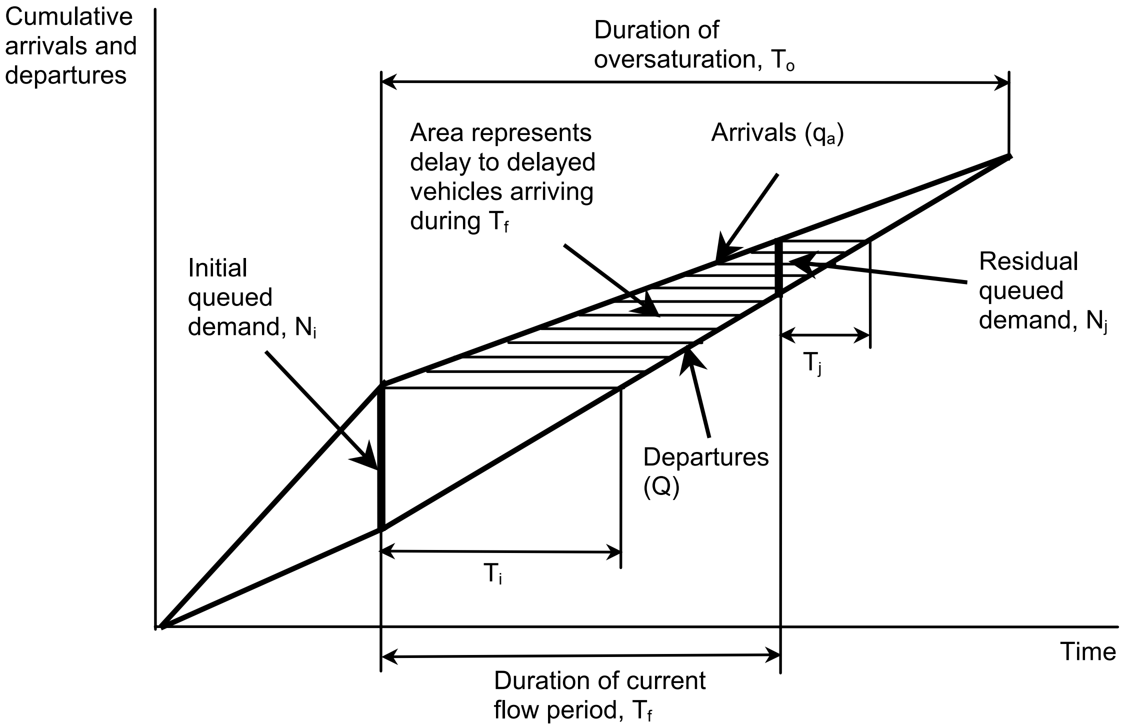

Figure 5.1 - Parameters in the derivation of delay and back of queue formulae for the case with initial queued demand

A deterministic expression for oversaturation delay based on this definition is derived assuming constant demand flow and capacity flow rates:

$$d_d = 3600\ [(N_i / Q) + 0.5\ T_f\ (x - 1)\ ] \tag{5.17}$$

where d_d is in seconds, N_i is in vehicles, T_f is in hours, Q is in veh/h.

This expression is used together with the steady-state delay expression (*Equation 5.16*) in applying the coordinate transformation method to obtain the time-dependent delay expression.

Other useful relationships are given below (see *Figure 5.1*).

Residual queued demand at the end of the flow period (vehicles):

$$N_j = \mathit{min}\ [0, N_i + (q_a - Q)\ T_f] \tag{5.18}$$

Time for the initial queued demand to clear (hours):

$$T_i = N_i / Q \tag{5.19}$$

Time for the residual queued demand to clear (hours):

$$T_j = N_j / Q \tag{5.20}$$

Duration of oversaturation, i.e. the time for the total demand during the current flow period to clear (hours):

$$
\begin{aligned}
T_o &= N_j / (Q - q_a) && \textit{for } q_a < Q \\
&= \textit{indefinite} && \textit{for } q_a \geq Q
\end{aligned}
\qquad (5.21)
$$

Equations (5.20) and (5.21) assume that capacity of the current flow period is valid after the current flow period until the residual queued demand clears. Therefore, T_o does not necessarily represent the actual duration of oversaturation as it needs to be revised during the calculations for the next flow period using the capacity calculated for that flow period.

6 PROPOSED SOLUTION

Various methods to revise the HCM speed-flow models to achieve expected characteristics discussed in *Section 4* have been investigated. The following method provides a simple solution for this purpose:

(i) use the same value of speed ratio for all four classes of basic freeway segments and for all four classes of multilane highways (v_n / v_f = 0.85 for freeways and 0.82 for multilane highways have been selected); and

(ii) use the same value of degree of saturation to determine the flow limit for free-flow speed (or zero traffic delay) for all four classes of basic freeway segments and for all four classes of multilane highways (x_o = 0.70 for freeways and 0.65 for multilane highways have been selected).

The resulting models are presented here using "Akcelik's" speed-flow function described in *Section 5*.

The results for the revised HCM models based on the use of v_n / v_f = 0.85 and x_o = 0.70 for freeways, and v_n / v_f = 0.82 and x_o = 0.65 for multilane highways are summarised in *Table 6.1* and shown in *Figures 6.1 to 6.7* (including speed and traffic delay estimates for oversaturated conditions). These results satisfy all the desirable characteristics stated in *Section 4*.

Table 6.1 should be compared with *Table 3.1*, and *Figures 6.1 to 6.7* should be compared with figures in Sections 3 and 4 as indicated in figure captions.

The simpler function form using x_o = 0 (therefore different k_d values) also gives satisfactory results.

Better model calibration would be achieved by selecting the v_n / v_f and x_o values using real-life data. Given the v_n / v_f and x_o values, all other parameters (k_n, d_{tn}, etc) can be determined including the delay parameter (k_d) for Akçelik's function form.

Table 6.1

Parameters for revised HCM speed-flow relationships for four classes of BASIC FREEWAY SEGMENTS and four classes of MULTILANE HIGHWAYS

	Basic Freeway Segments				**Multilane Highways**			
Facility class	**1**	**2**	**3**	**4**	**1**	**2**	**3**	**4**
Free-flow speed, v_f (km/h)	**120**	**110**	**100**	**90**	**100**	**90**	**80**	**70**
Capacity, Q (veh/h)	**2400**	**2350**	**2300**	**2250**	**2200**	**2100**	**2000**	**1900**
Density at capacity, k_n (veh/km)	**23.5**	**25.1**	**27.1**	**29.4**	**26.8**	**28.5**	**30.5**	**33.1**
Speed at capacity, v_n (km/h)	**102.0**	**93.5**	**85.0**	**76.5**	**82.0**	**73.8**	**65.6**	**57.4**
Speed ratio, v_n / v_f	**0.85**	**0.85**	**0.85**	**0.85**	**0.82**	**0.82**	**0.82**	**0.82**
Free-flow travel time, t_f (s/km)	30.0	32.7	36.0	40.0	36.0	40.0	45.0	51.4
Travel time at capacity, t_n (s/km)	35.3	38.5	42.4	47.1	43.9	48.8	54.9	62.7
Traffic delay at capacity, $d_{tn} = t_n - t_f$ (s/km)	5.3	5.8	6.4	7.1	7.9	8.8	9.9	11.3
Average headway at capacity, h_n (s/veh)	1.500	1.532	1.565	1.600	1.636	1.714	1.800	1.895
Average spacing at capacity, L_{hn} (m/veh)	42.5	39.8	37.0	34.0	37.3	35.1	32.8	30.2
Flow limit for free-flow speed, q_o (veh/h)	1680	1645	1610	1575	1430	1365	1300	1235
Degree of saturation at q_o ($x_o = q_o / Q$)	**0.70**	**0.70**	**0.70**	**0.70**	**0.65**	**0.65**	**0.65**	**0.65**
Delay parameter k_d for Akcelik's function (for $x_o > 0$)	0.14	0.16	0.19	0.23	0.24	0.29	0.34	0.43
Delay parameter k_d for Akcelik's function (for $x_o = 0$)	0.04	0.05	0.06	0.07	0.08	0.10	0.12	0.15

Data given in this table represent conditions for passenger cars only in a single lane.

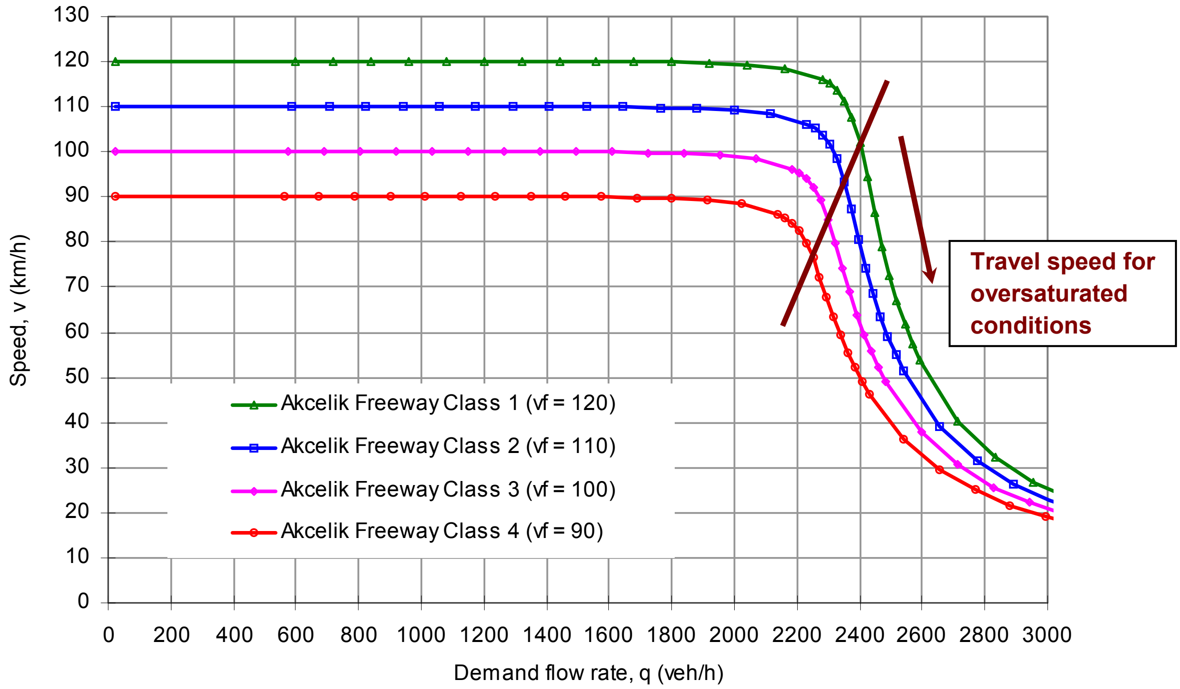

Figure 6.1 - Revised HCM 2000 speed-flow models using Akçelik's function (v_n / v_f = 0.85, x_o = 0.70) for four classes of BASIC FREEWAY SEGMENTS (compare with Figure 3.1)

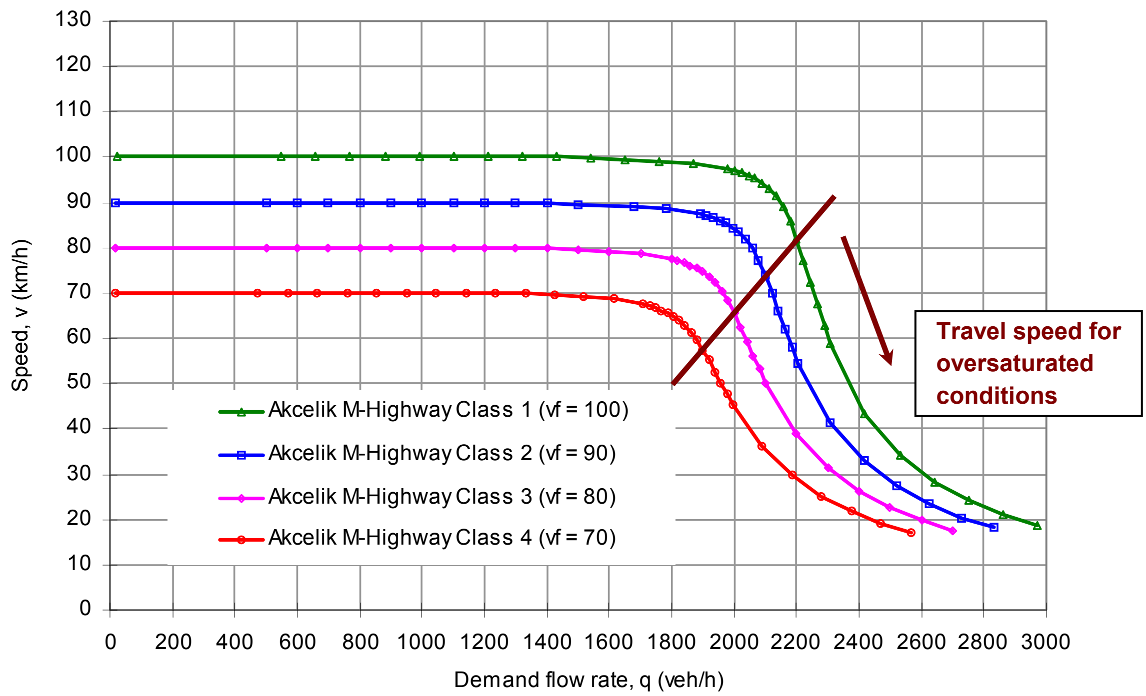

Figure 6.2 - Revised HCM 2000 speed-flow models using Akçelik's function (v_n / v_f = 0.83, x_o = 0.65) for four classes of MULTILANE HIGHWAYS (compare with Figure 3.2)

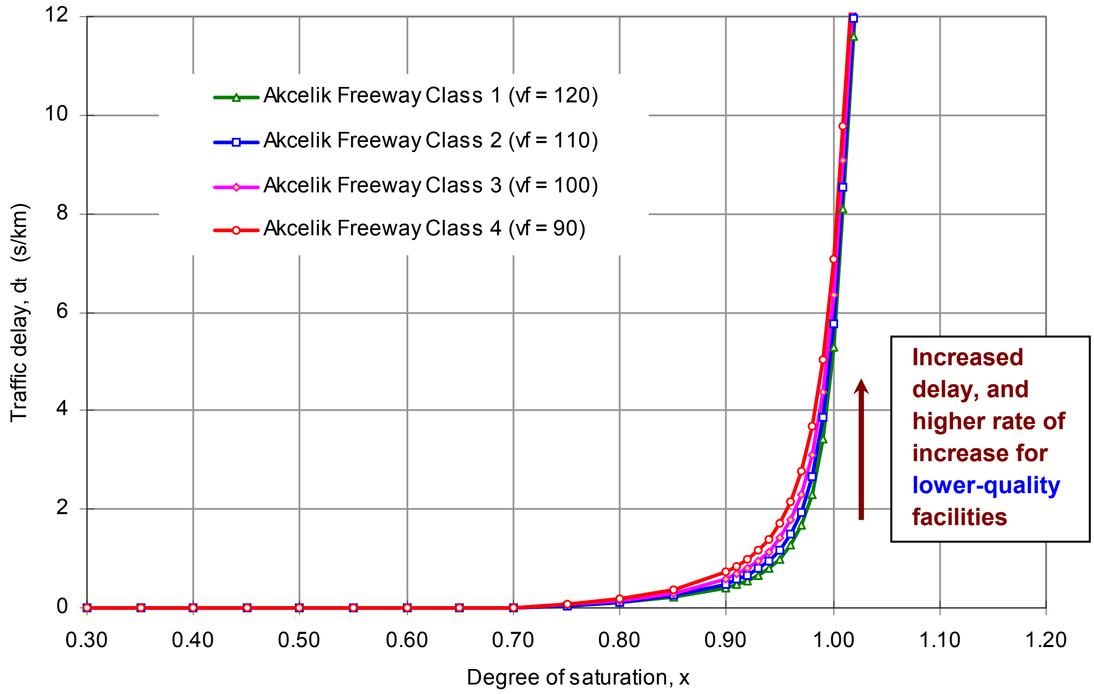

Figure 6.3 - Traffic delay graphs corresponding to the revised HCM speed-flow models using Akçelik's function for BASIC FREEWAY SEGMENTS (compare with Figure 3.3)

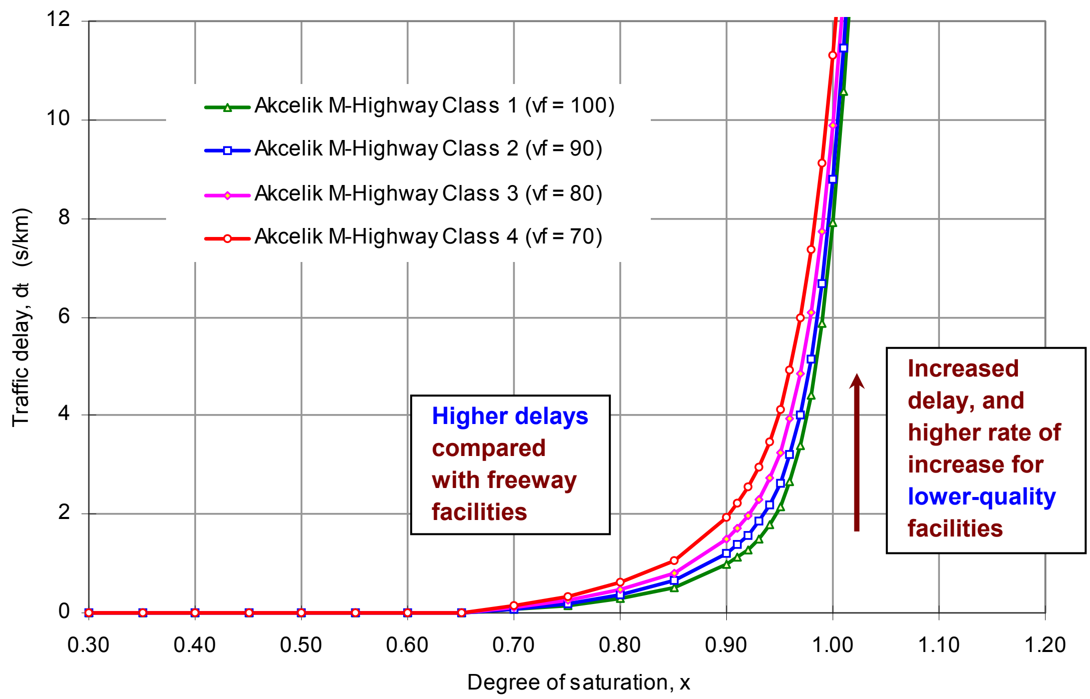

Figure 6.4 - Traffic delay graphs corresponding to the revised HCM speed-flow models using Akçelik's function for MULTILANE HIGHWAYS (compare with Figure 3.4)

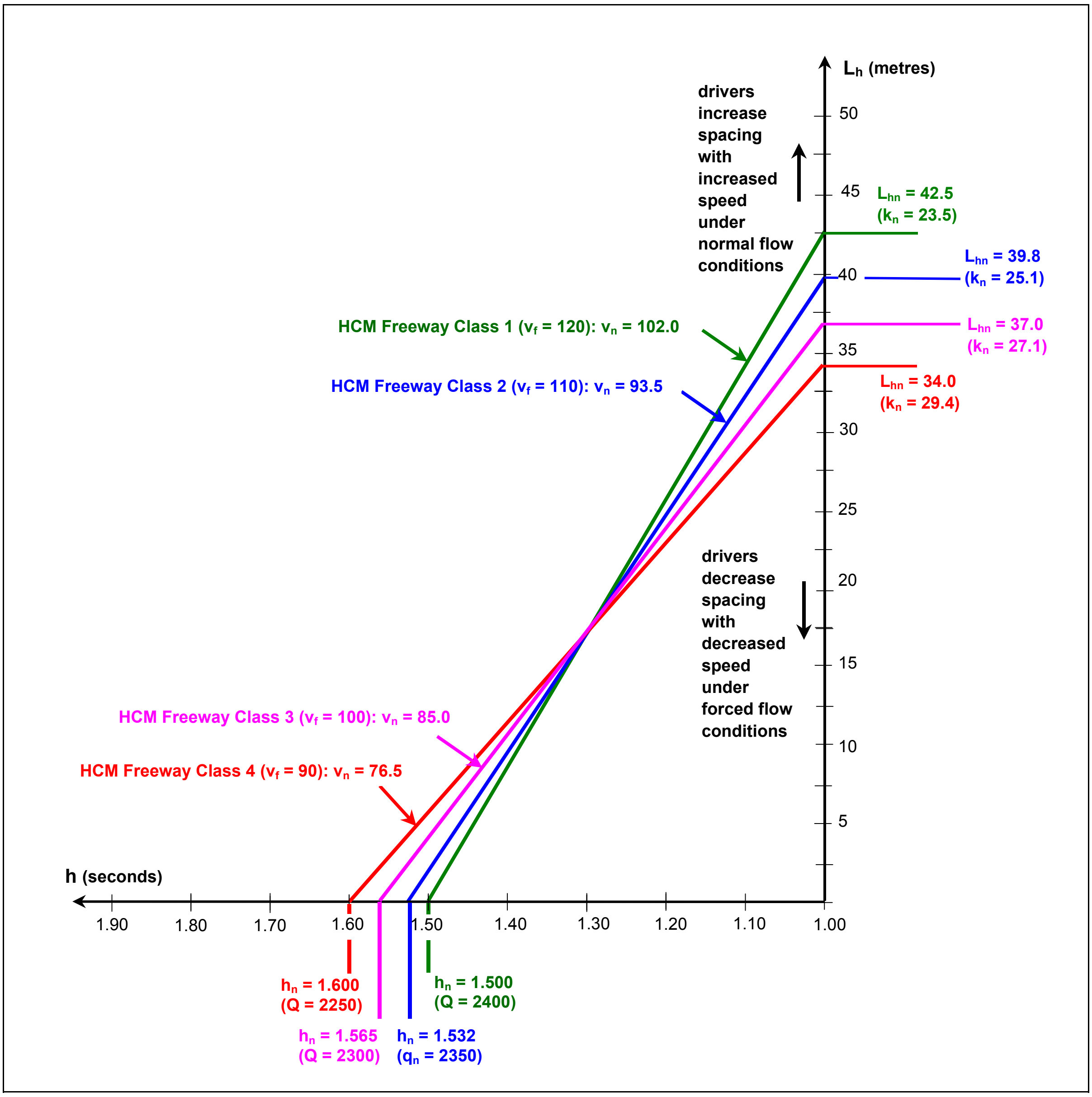

Figure 6.5 - Relationships between spacing, headway and speed at capacity corresponding to the revised HCM speed-flow models using Akçelik's function for BASIC FREEWAY SEGMENTS (compare with Figure 4.2)

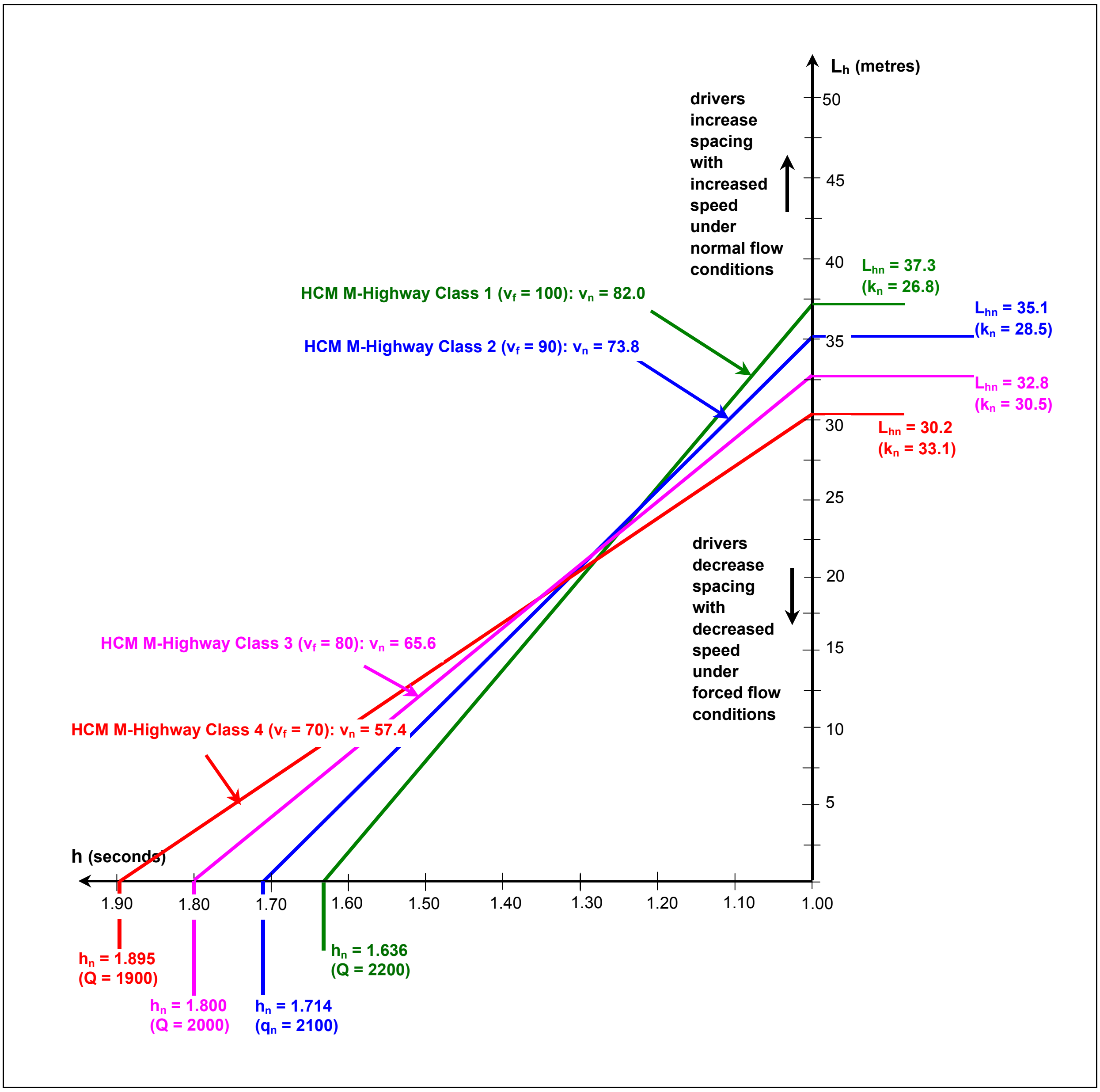

Figure 6.6 - Relationships between spacing, headway and speed at capacity corresponding to the revised HCM speed-flow models using Akçelik's function for MULTILANE HIGHWAYS (compare with Figure 4.3)

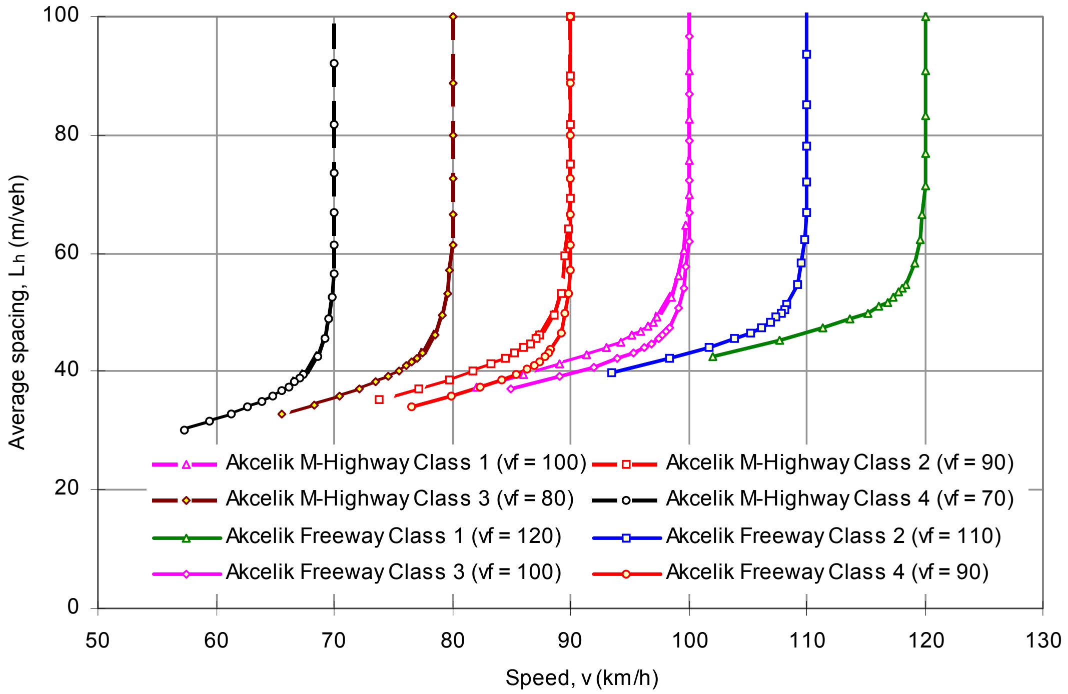

Figure 6.7 - Spacing-speed graphs corresponding to the revised HCM speed-flow models using Akçelik's function for BASIC FREEWAY SEGMENTS and MULTILANE HIGHWAYS (compare with Figure 4.3)

7 SPEED-FLOW MODELS FOR URBAN STREETS

The HCM classifies urban and suburban roads with signalised intersections spaced at less than 3 km as *urban streets* (Chapters 10 and 15). This section presents the speed-flow models for the *running time* component of the of the HCM travel speed model for urban streets derived using Akçelik speed-flow function described in *Section 5*. These are uninterrupted flow models applicable to midblock travel conditions, i.e. they do not include the intersection *control delay* component of travel.

The HCM describes four classes of urban streets. Exhibits 10-3 and 10-4 provide help with establishing urban street class. Exhibits 10-5 and 10-6 give default values of free-flow speed and signalised intersection intensity (signals per km). Exhibit 10-7 gives an example of further parameters chosen for four urban street classes.

Default free-flow speeds for urban street classes 1 to 4 are 80, 65, 55 and 45 km/h, respectively (Exhibit 10-5). HCM defines the *free-flow speed* on an urban street as "the average speed of the traffic stream when traffic volumes are sufficiently low that drivers are not influenced by the presence of other vehicles and when intersection traffic control (signal or sign) is not present or is sufficiently distant as to have no effect on speed choice", and states that "the best location to measure urban street free-flow speed is midblock and as far as possible from the nearest signalised or stop-controlled intersection".

In line with the definition of free-flow speed, *running speed* is based on running time measured as "the time taken to traverse the street segment, less any stop-line delay". This is interpreted here as the midblock "cruise" speed that is not affected by any intersection traffic control but influenced by the presence of other vehicles. Exhibit 15-3 gives running times per km recommended as a function of the segment length. In the method presented here, speeds are not adjusted for segment length.

Default signalised intersection intensity values for urban street classes 1 to 4 are 0.5, 2.0, 4.0 and 6.0 signals/km, respectively (Exhibit 10-6).

Exhibit 10-7 gives adjusted saturation flow values of 1850, 1800, 1750 and 1700 veh/h for four urban street classes for urban street classes 1 to 4. These are used as midblock capacities for the speed-flow models given here.

For the purpose of midblock speed-flow models for urban streets, the following key parameter values were selected for use in "Akcelik's" speed-flow function with a view to consistency with multilane highway values to reflect the premise that urban streets are lower-quality facilities relative to multilane highways (see *Section 6*):

(i) use the same value of speed ratio for all four classes of urban streets (v_n / v_f = 0.80 was selected); and

(ii) use the same value of degree of saturation to determine flow limit for free-flow speed (or zero traffic delay) for all four classes of urban streets (x_o = 0.50 was selected).

The resulting models are summarised in *Table 7.1* and shown in *Figures 7.1 to 7.4* (including speed and traffic delay estimates for oversaturated conditions). Note that the intersection control component reduces the capacity substantially compared with uninterrupted flow capacity, resulting in low degrees of saturation at midblock location. Therefore the uninterrupted flow component of urban street travel remains close to the free-flow speed.

Table 7.1

Parameters for revised HCM speed-flow relationships for four classes of URBAN STREETS

	Urban Streets			
Facility class	**1**	**2**	**3**	**4**
Free-flow speed, v_f (km/h)	**80**	**65**	**55**	**45**
Capacity, Q (veh/h)	**1850**	**1800**	**1750**	**1700**
Density at capacity, k_n (veh/km)	**28.9**	**34.6**	**39.8**	**47.2**
Speed at capacity, v_n (km/h)	**64.0**	**52.0**	**44.0**	**36.0**
Speed ratio, v_n / v_f	**0.80**	**0.80**	**0.80**	**0.80**
Free-flow travel time, t_f (s/km)	45.0	55.4	65.5	80.0
Travel time at capacity, t_n (s/km)	56.3	69.2	81.8	100.0
Traffic delay at capacity, $d_{tn} = t_n - t_f$ (s/km)	11.3	13.8	16.4	20.0
Average headway at capacity, h_n (s/veh)	1.946	2.000	2.057	2.118
Average spacing at capacity, L_{hn} (m/veh)	34.6	28.9	25.1	21.2
Flow limit for free-flow speed, q_o (veh/h)	925	900	875	850
Degree of saturation at q_o ($x_o = q_o / Q$)	**0.50**	**0.50**	**0.50**	**0.50**
Delay parameter k_d for Akcelik's function (for $x_o > 0$)	2.31	3.41	4.63	6.72
Delay parameter k_d for Akcelik's function (for $x_o = 0$)	0.14	0.21	0.29	0.42

Data given in this table represent conditions for passenger cars only in a single lane.

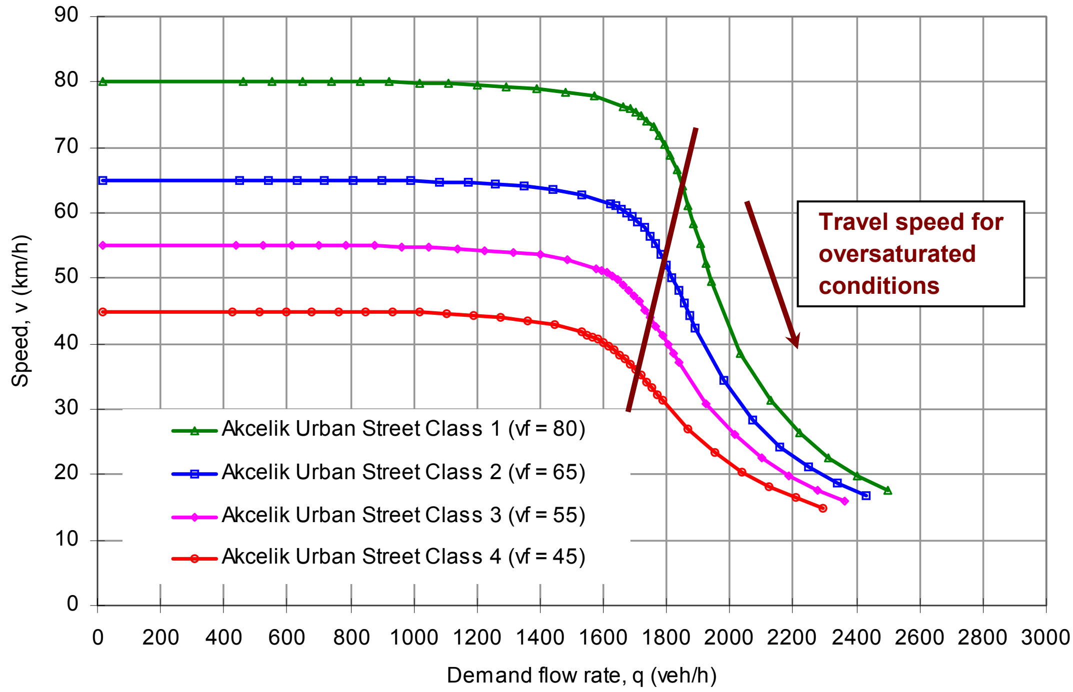

Figure 7.1 - Speed-flow models derived using Akçelik's function ($v_n / v_f = 0.80$, $x_o = 0.50$) for four classes of URBAN STREETS as defined in HCM 2000

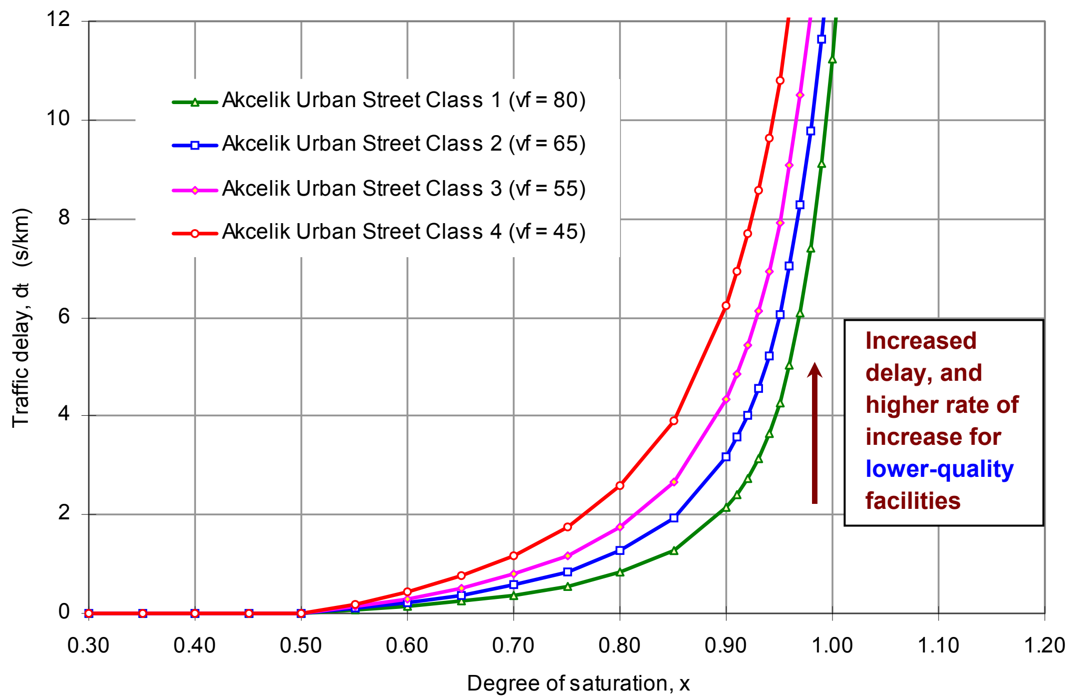

Figure 7.2 - Traffic delay graphs corresponding to the speed-flow models for four classes of URBAN STREETS as defined in HCM 2000

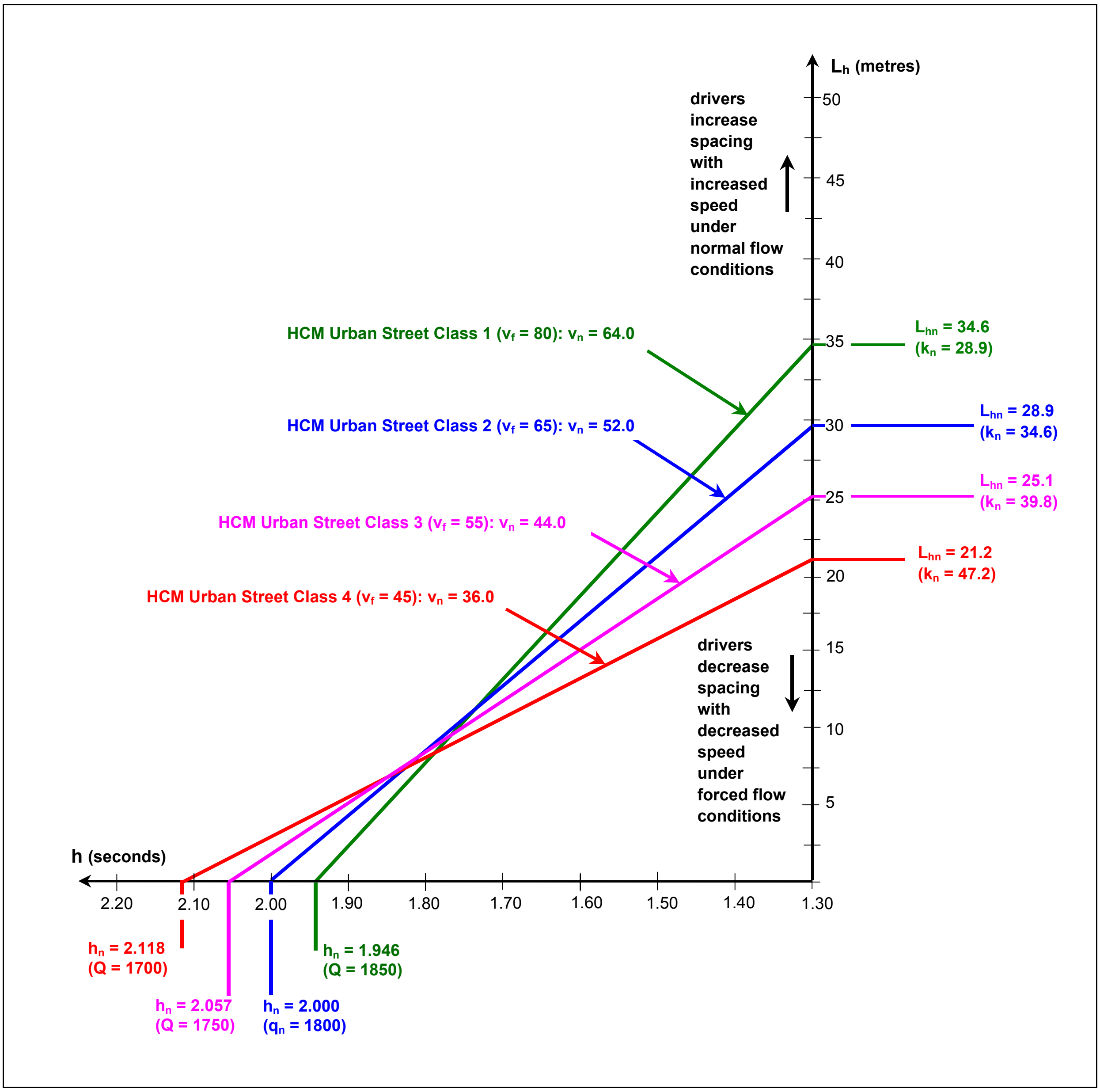

Figure 7.3 - Relationships between spacing, headway and speed at capacity corresponding to the speed-flow models for four classes of URBAN STREETS as defined in HCM 2000

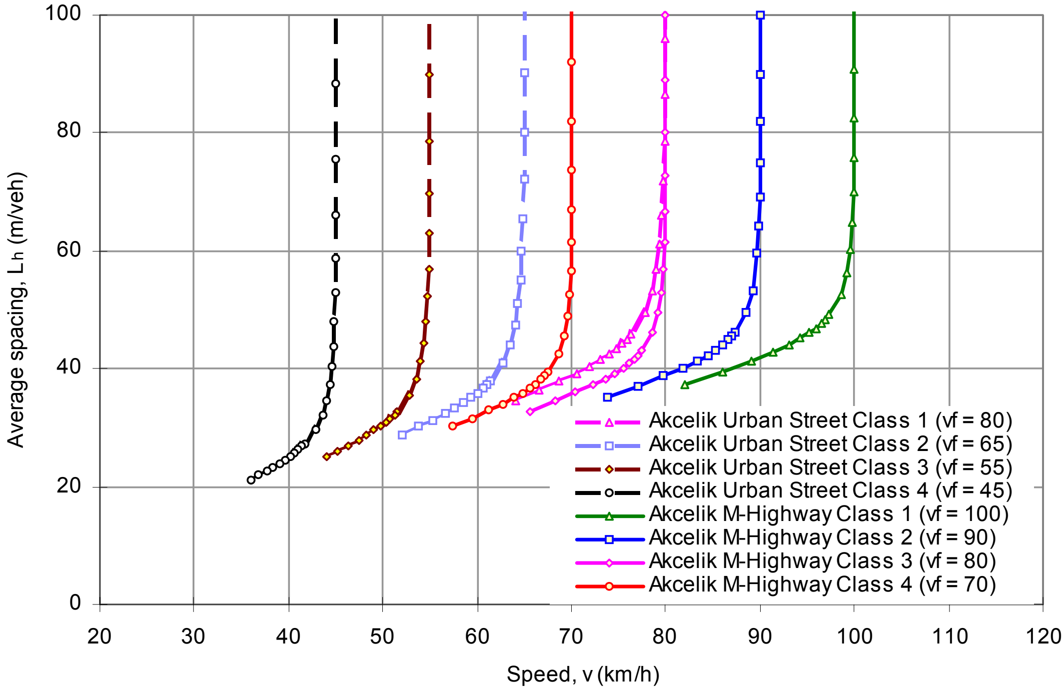

Figure 7.4 - Spacing-speed graphs corresponding to the speed-flow models for HCM 2000 URBAN STREETS and MULTILANE HIGHWAYS

8 DISCUSSION

Various counter-intuitive characteristics of the HCM speed-flow models for basic freeway segments and multilane highways have been highlighted in *Section 4* and a proposed revision has been described in *Section 6*.

The problem is related to the well-known difficulty in determining capacity and the corresponding speed from real-life data using regression methods. The results depend on the model chosen to some extent (Akçelik, Roper and Besley 1999).

HCM freeway speed-flow models are based on the NCHRP 3-45 research (Schoen, May, Reilly and Urbanik 1995). However, an inspection of the 5-minute speed-flow scatter plots given in Appendix B of the NCHRP 3-45 Report does not appear to support the HCM model characteristics discussed in this paper (most v_n / v_f ratios appear to be in the range 0.83 to 0.93). In fact, the NCHRP 3-45 Report, Section 4 concluded that "*Although only a limited validation of the analysis procedures was performed, the results with respect to estimation of free-flow speed and level of service are reasonable. However, the procedures tend to fall short in estimating travel speed at higher flow rates.*". A validation exercise in the same section found that, for a site chosen from the data set randomly, the observed speeds were found to be "*much less sensitive to higher flow rates than predicted by the (speed-flow) curves*".

HCM uses density as a level of service (LOS) measure for basic freeway segments and multilane highways. Different treatment of density at capacity suggested in *Section 4* of this report has LOS implications that:

(i) for freeways, maximum density that defines the upper limit of LOS E does not vary with different free-flow speeds, i.e. is the same for all freeway classes (k_n = 28 veh/km), but

(ii) for multilane highways, maximum density that defines the upper limit of LOS E varies with different free-flow speeds, i.e. is different for each multilane highway class (k_n = 25 veh/km for v_f = 100 km/h to k_n = 28 veh/km for v_f = 70 km/h).

Any revision of the speed-flow models needs to consider such LOS implications as well. Interestingly, unlike the latest 2000 edition, HCM 1994 edition used constant capacity values but different densities (therefore spacings) at capacity for freeways with different free-flow speeds.

The concerns raised in *Section 3* in terms of *traffic delay* functions for uninterrupted facilities can also be explained in relation to the *overflow delay* functions used for interrupted facilities in the aaSIDRA package (Akcelik and Associates 2002). These functions use a degree of saturation (x_o) below which the overflow queues and the corresponding delays are zero. This degree of saturation is higher for higher-quality facilities (highest for signalised intersections, followed by roundabouts, and lowest for sign-controlled intersections). Similarly, the overflow delay functions increase at the fastest rate for sign-controlled intersections and at the lowest rate for signalised intersections. It is for this reason that the practical (target) degrees of saturation for intersection design (e.g. to determine spare capacity values) are chosen as 0.80 for sign-controlled intersections, 0.85 for roundabouts, 0.90 for signalised intersections and 0.98 for uninterrupted movements.

Speed-flow models for the running time component of the HCM urban street classes derived using Akçelik's speed-flow function have also been provided. These allow for comparison with the speed-flow models for multilane highways.

Further analyses are recommended to determine fundamental characteristics of speed-flow models for freeways, multilane highways and urban streets using real-life data.

Comparisons of the HCM models as well as the models developed in Australia (Akçelik, Roper and Besley 1999) with the speed-flow relationships implied by various microsimulation models (Akçelik and Besley 2001) are also recommended.

REFERENCES

AKCELIK AND ASSOCIATES (2002). *aaSIDRA User Guide.* Akcelik and Associates Pty Ltd, Melbourne, Australia.

AKÇELIK, R. (1991). Travel time functions for transport planning purposes: Davidson's function, its time-dependent form and an alternative travel time function. *Australian Road Research* 21 (3), pp. 49–59.

AKÇELIK, R. (1996). Relating flow, density, speed and travel time models for uninterrupted and interrupted traffic. *Traffic Engineering and Control* 37(9), pp. 511-516.

AKÇELIK, R. and BESLEY, M. (2001). Microsimulation and analytical methods for modelling urban traffic. Paper presented at the *Conference on Advance Modeling Techniques and Quality of Service in Highway Capacity Analysis*, Truckee, California, USA.

AKÇELIK, R., ROPER, R. and BESLEY, M. (1999). *Fundamental Relationships for Freeway Traffic Flows.* Research Report ARR 341. ARRB Transport Research Ltd, Vermont South, Australia.

BLUNDEN, W. R. (1971). *The Land-Use / Transport System: Analysis and Synthesis*. Pergamon Press, Oxford.

BLUNDEN, W. R. (1978). On Davidson's flow/travel time relationship. *Australian Road Research*, 8 (2), pp. 50-51.

DAVIDSON, K. B. (1966). A flow–travel time relationship for use in transportation planning. *Proc. 3rd ARRB Conf.* 3 (1), pp. 183-194.

DOWLING, R.G., SINGH, R. and CHENG, W.W.K. (1998). The accuracy and performance of improved speed-flow curves. Technical Note. *Road and Transport Research* 7 (2), pp. 36-51.

DOWLING, R.G. and ALEXIADIS, V. (1997). A blueprint for applying EMME2 to ramp metering analyses. Paper submitted for presentation at *12th Annual International User's Group Conference, San Francisco, October 22-24, 1997.*

NAKAMURA, K. and KOCKELMAN, K.M. (2000). Congestion pricing and roadspace rationing: an application to the San Francisco Bay Bridge corridor. Paper submitted to *Transportation Research Board*.

REILLY, W., HARWOOD, D., SCHOEN, J. and HOLLING, M. (1990). *Capacity and LOS Procedures for Rural and Urban Multilane Highways*. Final Report, National Cooperative Highway Research Program, NCHRP Project 3-33. JHK & Associates, Tucson, Arizona.

SCHOEN, J., MAY, A., REILLY, W. and URBANIK, T. (1995). *Speed-Flow Relationships for Basic Freeway Segments*. Final Report, National Cooperative Highway Research Program, NCHRP Project 3-45. JHK & Associates, Tucson, Arizona.

SINCLAIR KNIGHT MERZ (1998). *Review and Update of the Speed/Flow Curves and Road Link Types in the Melbourne Strategic Highway Model*. Final report to Department of Infrastructure. Melbourne, Victoria, Australia.

SINGH, R. (1999). Improved speed-flow relationships: application to transportation planning models. Paper presented at the *7th TRB Conference on Application of Transportation Planning Methods, Boston, MA, March 1999*.

TRB (2000). *Highway Capacity Manual.* Transportation Research Board, National Research Council, Washington, D.C., U.S.A. (*"HCM 2000"*).