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NCHRP 3-55(2)A
PLANNING APPLICATIONS FOR THE
YEAR 2000 HIGHWAY CAPACITY MANUAL

APPENDIX A

LITERATURE REVIEW

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Literature Review of Speed and LOS Estimation Techniques

This appendix presents a review of various techniques for estimating speed and level of service that have been recommended by various authors in the literature. It is an update and expansion of the literature review originally conducted and included in the Final Report for the NCHRP 3-55(2) Project, Planning Techniques to Estimate Speeds and Service Volumes[**1**].

The review is divided into four major topic areas:

1. Techniques for estimating free-flow speeds.
2. Simple equations for quickly estimating link or segment speeds given volume and capacity.
3. More elaborate multi-step procedures for estimating facility speeds.
4. Methods for estimating facility and system-wide levels of service.

1. Free-Flow Speed Estimation Techniques

1.1 1994 Highway Capacity Manual - Multi Lane Highways

The 1994 Highway Capacity Manual (**2**) contains a procedure for estimating free-flow speeds for multilane highways where signalized intersections are spaced at least 3 kilometers (about 2 miles) apart. The equation requires as input the median type, the lane width, lateral clearance, and number of access points per mile (see Equation 7-1, Table 7-2, Table 7-3, Table 7-4, Table 7-5 of the HCM).

$$FFS = FFS_I - F_M - F_{LW} - F_{LC} - F_A \quad \text{Equation 1}$$

where:

FFS = Computed Free-Flow Speed (mph),

FFS_I = Ideal free-flow speed (mph),

F_M = Adjustment factor for median type,

F_{LW} = Lane width adjustment,

F_{LC} = Lateral clearance adjustment,

F_A = Access points density adjustment.

The user must also provide the “ideal” free-flow speed before these adjustments can be applied to arrive at the “actual” free-flow speed. Page 7-10 of the HCM cites unreferenced recent research that found that ideal free flow speed is 5 to 7 mph higher than the posted speed limit.

1.2 NCHRP 3-45 - Freeways

Schoen [**3**] developed the following equation for predicting the free-flow speed on freeways:

$FFS = 70 - F_n - F_{lw} - F_{lc} - F_{id}$ **Equation 2**

where:

FFS = free flow speed for basic freeway segment (mph)
F_n = adjustment factor for effect of number of lanes
F_{lw} = adjustment factor for effect of lane width
F_{lc} = adjustment factor for effect of lateral clearance
F_{id} = adjustment factor for effect of interchange density

The 70 mph ideal speed assumed in the equation should be replaced with a higher value if the posted speed limit exceeds 70 mph. This equation has been adopted for the 1997 update of the Highway Capacity Manual.

1.3 NCHRP 3-55(2) Method - Uninterrupted Flow Facilities

Dowling [**4**] suggests a set of linear equations for estimating free-flow speed based upon data gathered on mean speed, and the posted speed limit. Figure 1 shows the relationship of 85th percentile speed, and mean speed to posted speed limit from data developed by Tignor and Warren. Two regression equations are recommended, one for high speed facilities (speed greater than 50 mph), the other for lower speed facilities. These equations were derived from field measurements of free-flow speed.

Equation for Posted Speed Limits Over 80 km/h (about 50 mph):

Mean Speed (kph) = 0.88 * (Posted Speed Limit)(km/h) + 22 **Equation 3**

Equation for Posted Speed Limits of 80 km/h or less:

Mean Speed (kph) = 0.79 * (Posted Speed Limit)(km/h) + 19 **Equation 4**

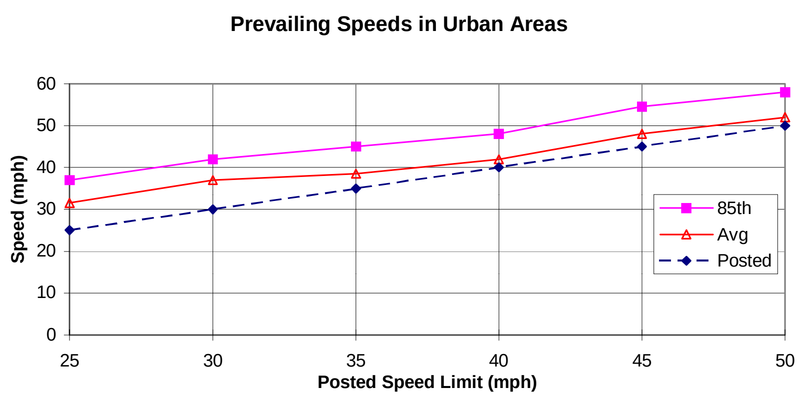


Figure 1. Speed Limit, 85 Percentile, and Mean Speeds in Urban Areas. (Tignor & Warren) [5]

1.4 NCHRP 3-55(2) Method - Interrupted Flow Facilities

Dowling [**6**] suggests the following equation from the 1994 Highway Capacity Manual for estimating the free-flow speed for signalized facilities. This equation is nearly identical to one recommended by Horowitz [**7**].

The equation takes into account both the posted speed limit and the signal delays along the street (which occur even at low volumes). The mean free flow speed (including signal delay) is computed using the following equation which adds together the free-flow travel time between signals and the delay time at signals (under free-flow conditions).

$$S_f = \frac{L}{L/S_{mb} + N*\left(D/3600\right)} \qquad \textbf{Equation 5}$$

Where:

S_f = Free flow speed for urban interrupted facility (km/h)
L = Length of facility (km)
S_{mb} = 0.79 (Posted Speed Limit in km/h) + 19
N = number of signalized intersections on length "L" of facility
D = average delay per signal per equation 4 below (sec).

The average delay per signal is computed using the following equation:

$$D = DF * 0.5 * C(1-g/C)^2 \qquad \textbf{Equation 6}$$

where:

D = The total signal delay per vehicle (sec)
g = The effective green time (sec)
C = The cycle length (sec)
DF = (1-P) / (1-g/C) where: P = The proportion of vehicles arriving on green

Horowitz suggests the following signal delay values (D) based upon the cycle length and g/C, in lieu of the above equation for computing signal delay.

g/C	60" Cycle	70" Cycle	90" Cycle
Low (0.33)	21	26	31
Medium (0.50)	17	20	24
High (0.67)	12	14	17

Horowitz then suggests the following delay adjustment factors (DF) based on the quality of the signal coordination on the facility:

Quality of Signal Coordination	DF
Poor coordination	1.85
No Coordination	1.00
Excellent coordination	0.53

2. Speed Estimation Equations

2.1 Bureau of Public Roads Equations

The Bureau of Public Roads (BPR) equation and its variations are used by transportation demand modelers to predict speed as a simple function of volume/capacity ratio (Comsis **[8])**. The standard BPR equation is as follows:

$$s = \frac{s_f}{1 + a(v/c)^b} \qquad \text{Equation 7}$$

where:

- s = predicted mean speed
- s_f = free flow speed
- v = volume
- c = practical capacity,
- a = 0.15
- b = 4

Practical capacity is defined in this equation as 80% of the capacity. Free-flow speed is defined as 1.15 times the speed at the practical capacity.

The parameter “a” determines the ratio of free-flow speed to the speed at capacity. The parameter “b” determines how abruptly the curve drops from the free-flow speed. A high value of “b” causes speed to be insensitive to v/c until the v/c gets close to 1.0, then the speed drops abruptly.

One basic problem with the BPR formula is that it has delay as a function of the free flow travel time and therefore the length of the link. This is illogical for conditions where demand exceeds capacity. Classical deterministic queuing theory demonstrates that delay (for over congested conditions) is strictly a function of the number of vehicles in the queue and the discharge rate (capacity) of the facility. Length of the facility has no impact on queue delay. According to the BPR formula over congested short links will experience less delay than similarly over congested long links.

2.2 Horowitz Variation of BPR Curves

Horowitz [**9**] used a least squares fitting technique to derive the “a” and “b” parameters for the BPR curve that best fit the 1985 HCM curves for freeways and multi-lane highways (see Table 1).

Table 1. Parameters for Best Fitting BPR Curve to 1985 HCM Curves

Facility Type	Free-Flow Speed	“a”	“b”
6-Lane Freeway	70 mph	0.88	9.8
	60 mph	0.83	5.5
	50 mph	0.56	3.6
4-Lane Multilane Highway	70 mph	1.00	5.4
	60 mph	0.83	2.7
	50 mph	0.71	2.1

He suggests in a later paper **[10]** that these parameters can be easily updated for the 1994 HCM, but does not provide the specific results.

Horowitz suggests that for two lane rural highways 40% of the opposing direction flow should be added to the subject direction flow in order to use the BPR curve to obtain speed estimates consistent with the speed-flow curve shown in the 1994 Highway Capacity Manual for two-lane rural roads. Specific “a” and “b” parameters are not suggested for two-lane rural highways. It is presumed that the multi-lane highway parameters could be used.

2.3 Conical Delay Functions

H. Spiess [**11**] developed a revised speed-flow equation designed to enable computers to compute equilibrium traffic flows much more rapidly than with the standard BPR curve. The BPR curve is highly volatile at high v/c ratios (a slight change in the forecasted volume results in large changes in the estimated speed) and is too insensitive at low v/c ratios (large changes in volumes result in minor changes in speed). The BPR curve also uses exponentials which slows computer computations. All of these characteristics of the BPR curve tend to slow down computer travel model computations of equilibrium traffic volumes. Spiess suggested a “conical delay function” as a more computationally efficient speed-flow curve that still is very similar to the BPR curve. The conical delay function drops off fairly constantly over lower ranges of v/c ratios and does not increase as rapidly as the BPR curve at higher v/c ratio ranges. The equation is as follows:

$$t = t_0 * \left[2 + \sqrt{a^2 * (1-x)^2 + b^2} - a * (1-x) - b \right] \qquad \textbf{Equation 8}$$

where:

t = travel time (sec)

t_o = the travel time under free-flow conditions (sec)

a = a calibration parameter that must be greater than 1.

b= (2a -1) / (2a -2)

x = v/c ratio

Note that: at capacity (x=1), t = 2 t_o ; and at zero volume (x=0), t = t_o .

2.4 Akçelik / Davidson Formula

Akçelik **[12][13]** proposed the following modification to Davidson’s equation **[14]** for predicting the travel time on any road facility. The equation applies to v/c ratios above and below 1.00. The equation predicts the inverse of speed, the travel time per unit distance.

$$t = t_0 \left\{ 1 + 0.25 \frac{T}{t_0} \left[(x-1) + \sqrt{(x-1)^2 + \frac{8J_A}{QT} x} \right] \right\} \qquad \text{Equation 9}$$

where:

t = average travel time per unit distance (hours/mile)
t_0 = free-flow travel time per unit distance (hours/mile)

T	= the flow period, (typically one hour) (hours)
x	= the degree of saturation = volume/capacity
Q	= capacity (vph)
J_A	= the delay parameter

The delay parameter J_A is a function of the number of delay causing elements in the section of road and the variability of the demand. Akçelik suggests lower values of J_A for freeways and coordinated signal systems. Higher values apply to secondary roads and isolated intersections.

The value of J_A can be computed if the difference in the rate of travel (hours per mile) between capacity and free flow conditions on the facility is known. Substituting x=1.00 in the above equation and solving for J_A yields:

$$J_A = 2Q(t_c - t_0)^2 \qquad \textbf{Equation 10}$$

where t_c = the rate of travel at capacity (hours per mile).

The equation explicitly takes into account the delays caused by queuing and can be applied to any facility type. The assumptions are that there is no queue at the start of the analysis period, and there is no peaking of demand within the analysis period (T).

Dowling [**15**] investigated the comparative accuracy and model run time performance of the BPR and Akcelik curves and found that the Akcelik curve results in significantly improved traffic assignment run times and provides more accurate speed estimates over a range of demand conditions than the Standard BPR curve.

2.5 NCHRP 7-13 Curves

Lomax et. al.**[16]** used linear regression to fit a set of peak hour speed flow curves for arterials and freeways to various data sets they obtained as part of their research. The curves predict speed based on the volume/capacity ratio, signal spacing, and frequency of access points. These linear equations were then manually smoothed into a series of curves for use in looking up speeds as a function of signal density, access density, and volume/capacity ratios for use in situations when direct data collection is not possible.

For Freeways, the best predictor of speed uses daily traffic volume per lane and access frequency. Separate equations were developed for freeways that account for the effects of freeway bottlenecks.

No Bottleneck Effect:

$$\text{Speed (mph)} = 91.4 - 2.0\,[\text{ADT / Lane (1,000s)}] - 2.85\,[\text{AccessPointsPerMile}] \qquad \textbf{Equation 11}$$

With Bottleneck Effect:

$$\text{Speed (mph)} = 86.4 - 1.5\,[\text{Effective ADT / Lane (1,000s)}] - 2.85\,[\text{AccessPointsPerMile}] \qquad \textbf{Equation 12}$$

Where:

$$\text{Effective ADT / Lane} = \text{Bottleneck ADT / Lane}\left[W_1 - W_2 * d\right]$$

W_1 = weighting factor for magnitude of bottleneck = 1.1

= (1.4 when ADT per lane exceeds 30,000)

W_2 = weighting factor for distance to bottleneck = 0.1

d = distance to beginning of bottleneck

Lomax, et. al. developed a set of composite curves based on their data sets. The composite curves show a non-linear relationship between signal density and average speed for the various traffic volume levels.

For HCM94[1] Class I Arterials (using ADT/Lane as a surrogate for the v/c ratio):

$$\text{Speed (mph)} = \frac{60}{\left(\frac{60}{\text{free} - \text{flow speed}}\right)(1 + \text{effective signal density})^{0.3}\left(1 + \left(\frac{\text{ADT / Lane}}{10{,}000}\right)^{4}\right)^{0.7}} \quad \textbf{Equation 13}$$

For HCM 94 Class II/III Arterials (using ADT/Lane as a surrogate for the v/c ratio):

$$\text{Speed (mph)} = \frac{60}{\left(\frac{60}{\text{free} - \text{flow speed}}\right)(1 + \text{effective signal density})^{0.3}\left(1 + \left(\frac{\text{ADT / Lane}}{8{,}000}\right)^{4}\right)^{0.7}} \quad \textbf{Equation 14}$$

2.6 Van Aerde Car Following Model

Van Aerde **[17]**proposed a flexible single regime car following model that can be used to predict the speed of traffic as a function of volume or density. The analyst inputs 4 target points that the curve must match:

C = capacity (vph)

S_f = Free-Flow Speed (km/h)

S_c = speed at capacity (km/h), and

D_j = maximum “jam” density of vehicles (veh/lane/km)

These target points are used to compute four parameters: k, p1, p2, p3 of the Van Aerde equation as follows:

$$\mathbf{k = (2S_c - S_f) / (S_f - S_c)^2} \quad \textbf{Equation 15}$$

[1] Highway Capacity Manual, 1994 Edition. Arterial Class definitions were changed in the 1997 edition of the HCM.

$p_2 = 1/\{D_j*(k+1/S_f)\}$ **Equation 16**

$p_1 = k*p_2$ **Equation 17**

$p_3 = (1/S_c) * \{S_c/C - p_1 - p_2/(S_f-S_c)\}$ **Equation 18**

Substituting the parameters (p_1, p_2, p_3) into the following equation gives two speed predictions, the higher speed is for uncongested conditions, the lower speed value is for forced flow (queuing) conditions.

$s = \{-b +- sqrt(b^2-4ac)\} / (2a)$ **Equation 19**

where:

a = 1 - v*p3 **Equation 20**

b = v*p3*Sf - v*p1 - Sf **Equation 21**

c = v*p2 + v*p1*Sf **Equation 22**

Note that this formula cannot accommodate forecast volumes in excess of capacity. If the demand is forecasted to exceed capacity then the user must use the shock wave analysis for freeways (NCHRP Report 255). Van Aerde's equation can be used in the shock wave analysis in-lieu of the speed flow curves cited in the discussion of NCHRP Report 255.

2.7 The HPMS Analytical Process

The Highway Performance Monitoring System Analytical Process [**18**] provides a process for estimating link speeds as a function of an initial running speed plus various adjustments for pavement conditions, curves, grades, speed change cycles, stop cycles, and idle time. The initial running speed is determined from a look-up table based on the facility type and the congestion level.

The speed adjustments are applied in sequence, first the initial running speed is reduced according to pavement conditions, which is then further reduced for the effect of curves, etc.. The speed adjustment for curves is applied only if the safe speed on the curve is lower than the reduced speed based on pavement conditions. The speed adjustment for grades is applied only to trucks. The adjustment for speed change, stop cycles, and idle time is a function of facility type and volume/capacity ratio.

2.8 Margiotta Equations

Margiotta, et. al. **[19]** presented regressed equations for estimating two-way speeds and delay based on two completed Federal Highway Administration (FHWA) projects: *Speed Determination Models for the Highway Performance Monitoring System (HPMS)*,1994; and *Roadway Usage Patterns: Urban Case Studies*, 1993. These studies developed a new measurement of daily congestion which estimates the cumulative effects of congestion on vehicle speeds over the course of entire day: the average annual daily traffic-to-two way capacity ratio (AADT/2C).

The basic travel time equation is:

$$t = t_0 + \left[(D / KVMT)*VMT / 1000\right]$$ **Equation 23**

where:

t = mean travel time (averaged over both directions).

t_0 = free flow travel time including signal delay at low flows.
D/KVMT = total delay both directions per 1000 vehicle miles traveled.
VMT = total two-way volume times length of link in miles.

The equations do not provide for the estimation of delay or speeds in a single direction on a facility. Another problem with this formulation is that the delay due to queuing is sensitive to the length of the link. According to this approach, two links at the same demand/capacity ratio will have different delays. The shorter link will have less delay than the longer link.

Margiotta, et. al. used multiple and nonlinear regression analysis to develop equations that predict delay for three facility types (freeways, signalized arterials, and unsignalized streets) for three separate time periods (peak period, peak hour, and daily) and three analysis periods (weekday, weekend/holiday, and combined). The peak period is the combination of the peak three hours in the morning (7-10 AM) and the peak three hours in the evening (4-7 PM). The peak hour is considered the peak hour in the PM (4-5 PM or 5-6 PM depending upon the facility. Once delay is estimated, speeds are calculated as a function of delay. For freeways, delay is that due to congestion, including queuing and queue dissipation. For signalized streets, delay is the additional travel time beyond that which would result if all vehicles could traverse the section at the free-flow speed, including not only the time spent sitting at red lights, but also the time lost while decelerating to a stop and then accelerating back to free-flow speed. The effects of signal progression were also incorporated into the signalized arterial equations. For unsignalized streets, delay takes into account unsignalized intersections and number of stop signs per mile. The equations are valid for AADT/2C levels up to 18.

Since the final speed equations are broken down by facility type, time periods, and analysis period, we refer readers to the literature source for a complete table of the equation coefficients. Here we present a couple sample equations for peak period weekday delay.

Peak Period Weekday Delay for Freeways: **Equation 24**

$$D/KVMT = 0.0001732632 * (AADT/2C)^5 - 0.0000116968 * (AADT/2C)^6 + 0.0000001974 * (AADT/2C)^7$$

Peak Period Weekday Delay for Signalized Arterials (AADT/2C <= 7): **Equation 25**

$$D/KVMT = (1 - e^{-.3n'}) * (NOQ + Q)$$

$$NOQ = 32.6326 + 0.27187282 * (AADT/2C)^2 - 0.01054104 * (AADT/2C)^3$$

$$Q = 0.0000288004 * (AADT/2C)^6 - 0.0000013948 * (AADT/2C)^7$$

where:

D/KVMT = delay for both directions (hours per 1,000 vehicle-miles)

AADT/2C = average annual daily traffic-to-two way capacity ratio

n' = adjusted signals per mile, adjusted for progression (ideal = 2n / (n+2), fixed time = n)

n = actual number of signals per mile

3. Multi-Step and Link/Node Speed Estimation Procedures

3.1 Highway Capacity Manual Methods

The 1994 Highway Capacity Manual is the single most frequently used source of techniques for predicting speeds for all planning purposes except regional traffic forecasting. The comparative complexity of its procedures discourage their direct use in regional traffic models.

The HCM procedures vary by facility type.

3.1.1 Freeways

The 1994 Highway Capacity Manual provides separate procedures for computing the average speed of traffic on basic sections, weaving sections, and ramp merge/diverge sections of freeways.

The average speed on a basic freeway section is computed in three steps.

Step 1: Convert predicted hourly volume to "ideal" volume.

Step 2: Compute Free-Flow Speed (The 1997 HCM uses the NCHRP 3-45 method).

Step 3: Compute Volume/Capacity Ratio and Look-Up Speed.

Note that the demand must be less than capacity in order to be able to use the charts to find mean speed.

The computation of speeds for weaving sections (Chapter 4 of the HCM) is not feasible for planning applications since it requires knowledge of the lane striping (lane adds and drops) in the weave section. The methodology is limited to weaving sections under 2,500 feet in length.

The analytical procedure for ramps (Chapter 5 of the HCM) produces speed estimates for only the two right-most lanes of the freeway. The two regression equations for predicting the speed of traffic on the freeway apply only to the immediate vicinity of an on-ramp or off-ramp (1500 feet upstream of an off-ramp, and 1500 feet downstream of an on-ramp)(see Table 5-4 HCM). The equations are applicable only for "stable flow regimes" (LOS better than E)(speeds greater than 42 mph).

These equations for ramp merge/diverge areas are not practical for planning purposes because they cover a small portion of the freeway (the 1,500 foot ramp influence area) at each ramp merge and diverge area, they apply only to speeds greater than 42 mph, and the equations do not predict speeds for vehicles outside of the rightmost two lanes on the freeway.

Chapter 6 of the HCM provides a procedure for analyzing a freeway composed of these three different section types, but provides no guidance on how the results might be combined to obtain an overall average speed (or level of service) for the entire freeway.

3.1.2 Multi-Lane Highways

The procedure for determining the congested speed for multi-lane highways is similar to that for freeways. The predicted volume must be converted to an ideal flow rate using Equation 7-3 of the HCM. The actual flow rate is converted to an equivalent ideal flow rate using the Peak Hour and the Heavy Vehicle factors. The lane width, shoulder width, and driver population are used to estimate the free-flow speed.

Once the user has computed the ideal flow rate and the actual free-flow speed, Figure 7-3 HCM can be entered to obtain the congested speed. This figure is almost identical to the freeway speed-flow curves for a four lane freeway, with the exception that the multi-lane highways figure allows for lower free-flow speeds down to 45 mph. Note that this figure also does not provide for v/c ratios greater than 1.00.

3.1.3 Rural Two-Lane Highways

Figure 8-1 of the HCM provides a speed-flow curve that applies to ideal conditions (design speed 60 mph or better, lane widths 12 feet or more, shoulders 6 feet wide or more, passing allowed everywhere, only passenger cars, 50/50 directional split, level terrain).

The user must convert the predicted volume to the equivalent hourly flow rate for the peak fifteen minutes in terms of passenger car equivalents. The user then divides the equivalent hourly flow rate (in pcu's) by the directional distribution adjustment factor (F_d), the width adjustment factor (F_w), and the heavy vehicle adjustment factor (F_{hv}) to obtain the ideal flow rate for entering Figure 8-1 (see equation 8-1 HCM for details).

This speed estimation method only applies to facilities with design speeds of 60 mph and volumes less than capacity. No adjustment process is provided for estimating average speeds for facilities with lower design speeds.

3.1.4 Urban and Suburban Arterials

The average travel speed for signalized facilities is computed according to the HCM method in 6 steps:

1. Convert daily traffic to peak hour,
2. Convert two-way peak hour volume to peak direction volume,
3. Subtract turning volumes made from exclusive lanes,
4. Compute arterial running time based on intersection spacing and the mid-block free-flow speed,
5. Compute intersection approach total delay,
6. Compute arterial average travel speed.

3.2 Courage et. al. Modification to HCM Arterial Method

Courage, Showers, and McLeod **[9]** noted that while the HCM method for urban arterials tended to estimate correctly the running speed, it tended to over-estimate the delay at the signals. They recommended two adjustment factors to reduce the uniform and incremental delay terms in the delay equation. The "incremental delay adjustment factor" was designed to compensate for the effect of closely spaced signals on the incremental delay. The "floating car adjustment factor" was designed to compensate for the presumed bias of floating cars in measuring uniform delay. This bias is computed as a function of the quality of progression.

The revised delay formula recommended by Courage et al. is:

$$D = 1.3 * (F_{fc} * d_u * DF + F_{ss} * d_i) \qquad \textbf{Equation 26}$$

where:

D = total approach delay, in sec/veh;
F_{fc} = floating car adjustment factor;
d_u = approach uniform delay, in sec/veh;

F_{ss} = signal spacing adjustment factor;
d_i = approach incremental delay, in sec/veh;
DF = delay adjustment factor.

The floating car adjustment factor is calculated as follows:

$$F_{fc} = \frac{g/c * x^2 * (1 - g/c * R_p) * (1 - g/c * x)}{[1 - g/c * x * R_p] * (1 - g/c)} \qquad \textbf{Equation 27}$$

where:

g/c = ratio of green time to cycle length
x = volume/capacity ratio (volume/saturation ratio divided by the g/c ratio)
Rp = platoon ratio (proportion of through vehicles arriving during green divided by the g/c ratio

The signal spacing adjustment factor is calculated as follows:

$$F_{ss} = \text{Min}\ [\frac{\text{signal spacing}}{\text{reference length}}, 1.0] \qquad \textbf{Equation 28}$$

where:

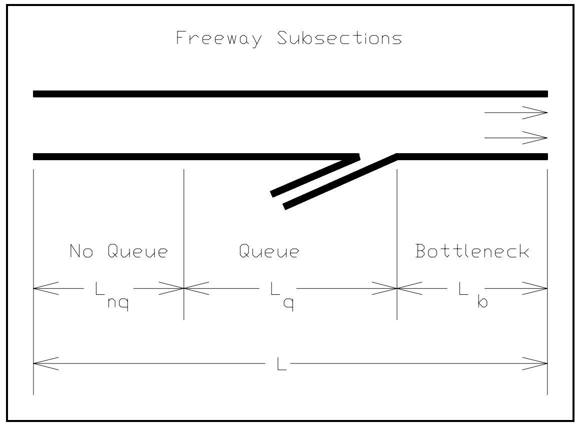
signal spacing = distance between signals
reference length = 1/2 mile (0.8 km) for HCM 94 class I arterials
= 1/4 mile (0.4 km) for HCM 94 class II arterials

The HCM method predictions of mean travel speed for Ventura Boulevard were, on the average, consistently 4.5 mph (7.2 kph) lower than the floating car speed measurements. The Courage et. al. recommended modifications routinely increased the HCM estimated speeds by 25% to 30%. The result was a modest over-correction of the HCM estimated speeds. The Courage modified HCM estimates were an average of 2.2 mph (3.5 kph) higher than the floating car measured speeds.

3.3 NCHRP 255 Procedures To Extend The HCM

Pedersen and Samdahl **[20]** developed a recommended set of procedures for computing speed, delay, and queue length for freeways that extend the 1994 Highway Capacity Manual methods to over-capacity conditions. These procedures were originally developed by Curry and Andersen **[21]**. One procedure uses “shock wave” analysis to predict queuing on freeways. The other procedure uses deterministic queuing to predict delay on interrupted flow facilities.

3.3.1 Freeway Shock Wave Analysis Procedure



This procedure uses the lower limb of the speed-flow curve for freeways that was reported in the 1985 HCM but is no longer included in the 1994 edition of the HCM.

The freeway is split into three subsections. The first subsection is the bottleneck where the upstream demand exceeds capacity(often the section of freeway just downstream of an on-ramp). The second subsection is the queue immediately upstream from the bottleneck (often the section immediately upstream from an on-ramp). The third subsection is the remaining portion of the freeway upstream of the queue (this subsection may not exist if the queue extends the full length of the freeway study section). The freeway study section must be extended if the computations show that the queue extends upstream beyond the initially selected freeway study section.

The average speed over the entire freeway section is then determined by averaging the speed in each subsection as shown in the following equation:

$$ARS = \frac{L}{\frac{L - L_b - L_q}{ARS_{nq}} + \frac{L_q}{ARS_q} + \frac{L_b}{ARS_b}}$$ **Equation 29**

where:

ARS	= Average Running Speed of entire freeway section
ARS_b	= Average running speed of bottleneck subsection of freeway
	= speed at capacity
ARS_q	= Average running speed in queue subsection upstream of bottleneck
ARS_{nq}	= Average running speed in subsection upstream of queue
L	= Length of entire freeway section
L_b	= Length of bottleneck section
L_q	= length of queue

The bottleneck and non-queuing subsection speeds can determined from the speed-flow curves shown in Chapter 3 of the HCM. The average speed within the queue section must be determined from the lower limb (the forced flow) portion of the speed-flow curve contained in the 1985 HCM.

The following equation provides an approximate fit to the lower limb of this curve.

$$ARS_q = A * \exp[\ln B * (\frac{v}{c})^{127}]$$ **Equation 30**

where: A = 5, B = 6, v/c is the flow rate under queuing conditions.

This curve approaches 30 mph at v/c = 1.00, and 5 mph at v/c = 0.00. Parameters “A” and “B” can be modified according to the following equations if different speeds are desired:

A = the speed at v/c = 0.00

B = {the speed at v/c = 1.00} divided by "A"

The length of queue (Lq) is computed as follows:

Lq = {QR* T}/{2DQ} **Equation 31**

where:

Lq = the average queue length during the analysis period (miles)
QR = the queuing rate (veh/hr)
= upstream demand - bottleneck capacity
T = length of time that the level of demand occurs (length of peak hour or peak period) (hrs)
note that the queue is building and not dissipating during this period.
DQ = change in vehicle density between queue and upstream non-queued subsection
= {Bottleneck Capacity}/ARSq - {Upstream Demand}/ARSnq

3.3.2 Arterial Queuing Analysis for Over-Capacity

The average running speed for the arterial is computed using the same equation as contained in Chapter 11 of the HCM:

$$\text{SPEED} = \frac{[3600 * \text{Length}]}{[(\text{RunningTimePerMile}) * (\text{Length}) + D]}$$ **Equation 32**

The difference is in the calculation of intersection delay (D) for those intersections on the arterial where the though movement volume/capacity ratio is greater than 1.00 (over congested intersections).

Step 1. Look-up the running speed for the link feeding the over congested intersection, the speed will be based on free-flow speed and signal density.

Step 2. Adjust the vehicle arrival rate for the fact that as the queue extends back from the intersection, vehicles join the queue "earlier" than they would have if the queue were at the intersection stop line.

$$\text{AAR} = \text{Demand} * \left\{1 + \frac{(\text{Demand} - \text{Capacity})}{\text{Lanes} * \text{Speed} * 240 - \text{Demand}}\right\}$$ **Equation 33**

Where:

AAR = Adjusted Arrival Rate (veh/hr)
Demand = predicted arrival rate of vehicles at congested intersection stop line (veh/hr)
Capacity = saturation flow rate per lane times the number of through lanes times the g/c ratio for the approach (vphpl)
Lanes = number of through lanes on the approach (one direction)
Speed = average running speed for the approach found in step 1
240 = assumed queue density of 240 vehicles per lane per mile (22ft/veh.)

Step 3. Compute the Queue Length.

$$Q = 0.5 * \left\{ T * (AAR - Capacity) + Capacity * \frac{Cycle - Green}{3600} \right\}$$ **Equation 34**

where:

Q	= the mean queue length (vehicles)
T	= Duration of Analysis period (hrs)
AAR	= Adjusted Arrival Rate (veh/hr)(from step 2)
Capacity	= maximum flow rate per lane times the number of lanes (veh/hr) (see step 2).
Cycle	= the signal cycle length (sec)
Green	= effective green time for through vehicles (sec)

Step 4. Compute average Delay (D) at over congested intersection.

D = 3600 * Q/Capacity **Equation 35**

where:

D	= average delay (sec)
Q	= mean queue length (veh)(from step 3)
Capacity	= saturation flow per lane * Lanes * Green/Cycle (veh/hr)

3.4 Ruiter Adaptation of HCM

Ruiter [**22**] demonstrated how the analysis procedures contained in the 1985 Highway Capacity Manual (HCM) could be used to develop facility specific speed-flow relationships through the use of pre-selected default values for various input items required by the HCM. Default values for various HCM input items are selected based on facility type, facility type subgroup, and area type. Ruiter then shows how the substitution of the default values results in one or more simplified equations that can be used to predict link speed. Ruiter illustrates the development of a simple equation combined with a look-up table for use in predicting freeway speeds. He also illustrates the development of a set of equations for computing signalized arterial speeds. None of these equations can be generalized, since they depend on the specific default values selected, however; the equation development procedure can be applied to any situation where the HCM techniques can be applied.

Ruiter suggests two equations for extending the HCM speed predictions to conditions where demand exceeds capacity.

For freeways and expressways:

$$S_p = S_{p1} * (0.555 + 0.444*(V/C)^{-3})$$ **Equation 36**

where: S_{p1} = speed at v/c = 1.0

For arterials and collectors:

$S_p = S_{p1.2} * (0.663 + 0.583 *(V/C)^{-3})$ **Equation 37**

where: $S_{p1.2}$ = speed at v/c = 1.2

These equations were developed for use in the Phoenix metropolitan area [**23**]. Ruiter recommends that peak spreading be applied to the demand volumes to reduce the over prediction of delay for high demand volumes that would result with these equations.

3.5 Horowitz Adaptation of HCM Intersection Delay Methods

Horowitz [**24**] has developed specifications for the adaptation of HCM intersection delay methods to travel demand forecasting models. He developed specifications for signalized, two-way stop, and all-way stop controlled intersections.

One novel concept that he proposed was "adaptive control", the type of control at the intersection (stop or signal) varies with the demand at the intersection. As demand increases the control logically transitions from sign control to signal control.

He recommended the following adaptations to the HCM signalized intersection delay estimation method.

1. Replace the saturation flow adjustment factors (lane width, grade, parking, buses, trucks, area type) with the final adjusted saturation flow rate for the through lanes on each approach of the intersection. This through movement saturation flow rate would be manually adjusted upwards by the planner as necessary to account for the effects of exclusive right lanes.
2. The exclusive left turn lane saturation flow rate should be computed using only two of the available cases in the HCM. Shared left-thru lanes should be converted to exclusive lanes only if warranted by the signal phasing. Conversion as a result of the HCM critical v/c computations would result in discontinuities in the delay estimates as the demand model cycles through different demand levels as part of an equilibrium assignment process.
3. The travel demand software would calculate the signal phasing, cycle length and green times based on the demand/saturation ratio for each approach (no specifics provided). The signal timing plan would not necessarily be optimal to minimize delay. The travel demand software would decide in left turn protection is warranted by the demands.
4. The peak hour factor should be set to 1.0 for whole hour analysis. The lane utilization factor should be set to 1.0 for entire approach analysis.
5. The delay function should compute total control delay including acceleration and deceleration delay.
6. He suggests that the delay adjustment factor look-up tables be replaced with linear equations related to the volume/capacity ratio.

He recommends various adjustments to the 1985 HCM procedures for all-way and partial stop controlled intersections which are no longer applicable.

3.6 Dowling & Skabardonis Queuing Analysis Method

One of the traditional problems with the incorporation of queuing analyses in transportation demand modeling has been the difficulty of tracking both the temporal duration and the geographical extent of the queue. Dowling & Skabardonis [**25**] demonstrated that reasonably accurate estimates of total system delay could be obtained by ignoring the geographical extent of the queues.

The method involves extending the peak hour demand forecast to a multi-hour peak period using locally available data on travel demand by hour of the day. Peak period demand is forecasted for each hour of the peak period based upon the peak hour forecast.

Average link speeds are then computed for each hour of the peak period using the hourly demands. If the demand during a particular hour exceeds the link capacity, the delay due to queuing is computed and added to the link travel time. Queues are carried over to the subsequent hour of the peak period.

The post processor was written as a macro in the MINUTP software package and tested against the FREQ and TRANSYT-7F traffic simulation models on a section of freeway and arterial street in Hayward, California.

All queues are stored on the link where the demand exceeds capacity. Queues are not propagated upstream, nor are they used to reduce downstream flows. The result of these simplifications is a series of over estimates and under estimates of the impacts of queuing that appeared to cancel out, at least under the limited testing performed by Dowling & Skabardonis.

3.7 Boston Central Artery Post Processor

Bechtel/Parsons Brinkerhoff and Cambridge Systematics [**26**] developed and applied a post-processor process that adjusted the forecasted link volumes and speeds output by the TRANPLAN software package for the Boston Central Artery Project. The various highway links in the model network were first grouped into five link types. The demand model forecasts were reviewed and revised to correct for any volume calibration errors observed in the base year model run. The corrected volumes were then input into the specially developed speed-flow equations derived from the 1985 Highway Capacity Manual and queuing theory (see previous chapter discussion on the speed formulae). The revised volumes and speed estimates were then output to a TRANPLAN readable file which was then read back into TRANPLAN.

The travel time prediction equations were developed for the following link types:

- Type 1 Links: Links where the travel time is constrained by signalization,
- Type 2 Links: Links where the travel time is constrained by geometrics,
- Type 3 Links: Expressway and ramp links with $v/c < 0.7678$,
- Type 4 Links: Expressway and ramp links with $v/c >= 0.7678$, and
- Type 5 Links: Links where the times are unconstrained.

Link types 1 and 2 use the same travel time formula but with different default values for some of the parameters.

Note that the third term in the equation is a deterministic queue delay formula for conditions when the v/c is greater than 1.00.

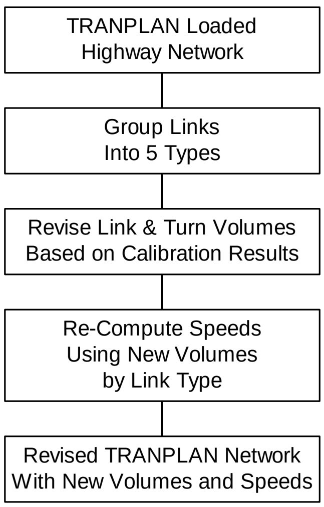


Figure 2. Boston Central Artery Post Processor

$$T = T_0 + \frac{0.38 * C * (1 - \frac{g}{C})^2 * PF}{(1 - \frac{g}{C} X)}$$
$$+ 173 X^2 \left[(X-1) + \sqrt{(X-1)^2 + 16 \frac{X}{cap}} \right] * PF \qquad \textbf{Equation 38}$$
$$+ 1800 * \left(\frac{V}{cap} - X \right)$$

where:

- T = Congested Travel Time (seconds)
- T_0 = Free-flow travel time (seconds)
- C = cycle length (seconds)
- g = green time (seconds)
- PF = Progression adjustment factor (set to 1.0)
- X = Minimum of volume/capacity ratio or 1.00
- V = Demand volume (vehicles per hour)
- cap = Capacity (vehicles per hour)

The travel speed on Type 3 links is determined by computing the volume/capacity ratio and looking up the speed in the 1985 Highway Capacity Manual. When the v/c ratio reaches 0.7678, then the formula for link type 4 is used to compute the speed.

The travel speed on link types 4 and 5 is computed using the BPR formula with an “a” coefficient of 0.15 and a “b” power of 6.

3.8 DTIM2 Speed Post Processor

SAI created a computer program for Caltrans, called the Direct Travel Impact Model (DTIM2) [**27**], that reads the loaded highway network produced by transportation planning software (TRANPLAN, MINUTP, and EMME2), and computes the corresponding pollutant emissions by 2 km grid cells within the region. The DTIM model contains an optional speed post-processor developed by Dowling [**28**] that uses 1985 Highway Capacity Manual techniques and queuing analysis to compute more accurate estimates of link speeds by hour of the day, over a 24 hour period.

The DTIM2 speed processor contains a set of speed-flow curves and equations for signalized and unsignalized facilities. These curves and equations have been verified on California freeways, rural highways and signalized arterials. Congested speeds on unsignalized facilities are estimated using variations of the BPR curve fitted to the speed-flow curves contained in Chapter 3 of the 1994 Highway Capacity Manual (HCM).

The DTIM data collection effort showed that rural highways have speed-flow curves similar to freeways when adjusting for the different free-flow speeds. Thus only a single set of speed-flow curves are provided for freeways and unsignalized highways.

Congested speeds on signalized facilities are estimated using the 1994 HCM procedure for signalized arterials. This procedure estimates speeds and signal delay based on signal spacing, capacity, and signal timing. The processor provides all of the needed signal timing and signal spacing data according to the facility type and area type of each highway link. The user can also directly input this signal data for specific links. The user can also edit the file of default signal data by area type and facility type to suit the conditions specific to the study area.

The HCM procedures contained in the DTIM2 speed processor are valid only for volumes less than capacity. The speed processor thus also contains a queuing analysis algorithm for use when volumes exceed capacity. The queuing algorithm splits the day into one hour long time slices. The total demand is allocated to each time slice according to peaking factors provided by the user.

Experience to date with the DTIM2 speed processor has found that it estimates speeds significantly lower than models using traditional BPR curves. This makes it difficult for planning agencies to switch to the speed processor because of inconsistency problems with previous forecast work by the agency.

3.9 NCHRP 3-55(2)

NCHRP 3-55(2) also recommended a post processor procedure that estimates the space mean speed and level of service for one direction of a facility over the entire peak period taking into account delays due to signal control and to queuing.

Unsignalized Facilities

The recommended procedure for unsignalized facilities is based on the analysis procedures contained in Chapters 3, 7, and 8 of the 1994 Highway Capacity Manual. The capacity and delay impacts of ramp merge, diverge points and weaving sections are neglected in this procedure.

The facility is divided into subsections (within which demand and capacity are relatively constant). The traffic demand in the peak period (if more than one hour long) is divided into a sequence of hourly demand rates. A simplified HCM analysis is then applied to each segment for each hour of the peak period. Excess demand in one hour on one segment is carried over to the following hour (but the queue is not propagated to upstream segments in order to save on computational complexity).

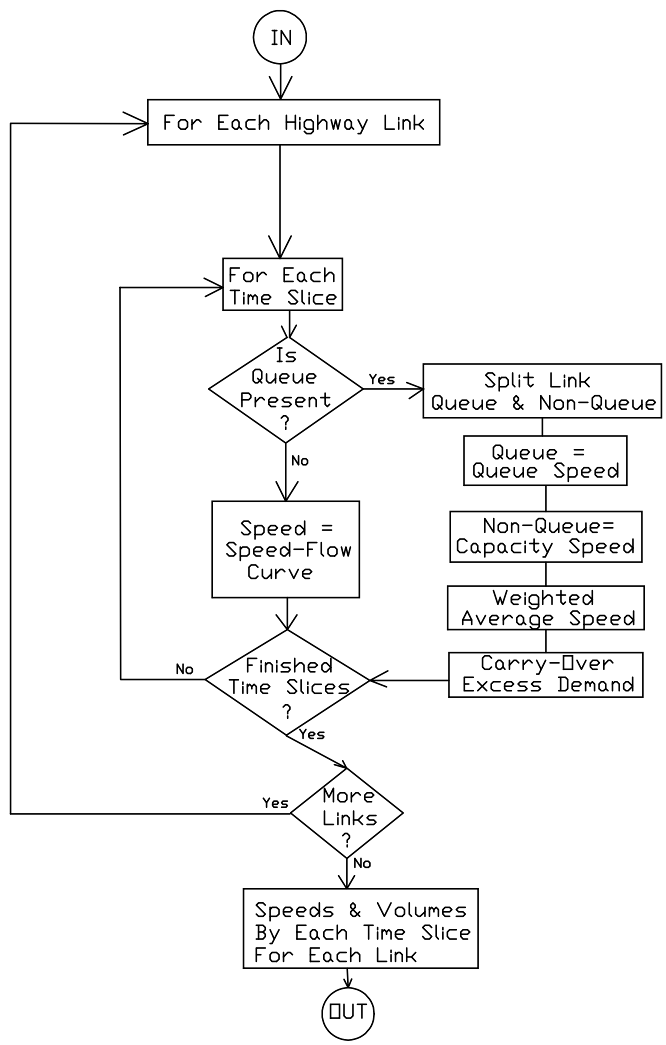


Figure 3. Speed Processor Flow Chart

If a queue is determined to exist, then the queuing delay (due to demand exceeding capacity) is computed using the following equation:

$$d_q = 3600 * T * \left(\frac{V_{t-1} + V_t}{2c} - 1 \right) \qquad \textbf{Equation 39}$$

where:

d_q	= Mean delay due to excess demand (sec).
T	= Duration of time period (hrs)
3600	= Converts hours to seconds.
V_{t-1}	= Leftover demand from previous time period (t-1).
V_t	= Additional demand occurring in current time period (t).
c	= Capacity of segment in subject direction (veh/hr)

The segment running times are computed for each segment (i) and time period (t) using the following equation:

$$R_{i,t} = 3600 * \frac{(1 + a(\frac{v}{c})^b_{i,t})}{S_f} \qquad \textbf{Equation 40}$$

where:

$R_{i,t}$	= Mean segment running time per unit length for segment “I” and time period “t” (sec/mi, sec/km)
S_f	= Mean segment free flow speed (mph or kph)
$v/c_{i,t}$	= Ratio of volume to capacity for the segment
a	= 0.20
b	= 10

The space mean speed over the entire peak period and the total study section length of a freeway, multi-lane-highway, or two-lane rural road is estimated using the following equation. Delays due to demand exceeding capacity on any one segment are added to the individual segment travel times, which are then summed over the entire study section to obtain the total travel time over the length of the study section. The total travel time is then divided into the total study section length to obtain the space mean speed for the study section.

$$s = \frac{3600 * N_t * \sum L_i}{\sum_{i,t} R_{i,t} * L_i + \sum_{i,t} dq_{i,t}} \qquad \textbf{Equation 41}$$

where:

s	= Space mean speed over the length of the facility (mph or kph).
L_i	= Length of segment “I” (mph or kph).
$R_{i,t}$	= Running time for segment “I” during time period “t” (sec/mile or sec/km).
$Dq_{i,t}$	= Delay due to queuing on segment “I” and time period “t”(sec).
Nt	= Number of time periods being analyzed.

Procedure for Signalized Facilities

The NCHRP 3-55(2) recommended speed estimation procedure for signalized facilities requires the estimation of signal timing for the facility. The g/C ratio (green time per cycle) for the through movement and the cycle length must be estimated for each intersection of the facility. The queue overflow and queue delay at the intersections are then computed.

The running time is computed based upon the mid-block free-flow speed, which is in turn computed based upon the posted speed limit. The node delay for signalized intersections is computed using equations adapted from Chapter 11 of the Highway Capacity Manual.

The space mean speed in one direction over the length of a signalized facility and over an entire analysis period is computed using the following equation:

$$SPEED = \frac{3600 * N_t * \sum L_i}{\sum_{i,t} R_{i,t} * L_i + \sum_{j,t} dn_{j,t} + \sum_{j,t} dq_{j,t}} \qquad \textbf{Equation 42}$$

where:

Speed = Space mean speed over the length of the facility (mph or kph).
N_t = Number of time periods (t) within analysis period.
L_i = Length of segment "I" (mph or kph).
$R_{i,t}$ = Running time for segment "I" (sec/mile or sec/km).
$Dn_{j,t}$ = Delay at node "j" for through traffic in the subject direction during time "t".
$dq_{j,t}$ = Delay due to demand exceeding capacity at node "j" during time period "t".

Dowling compared the speeds estimated using the NCHRP 3-55(2) post-processor process against field data and found that the post-processor is superior to the standard BPR equation, but not as accurate as using the HCM directly to estimate speeds.

4. Level of Service Estimation Techniques

4.1 DeArazoza / McLeod Speed Limit Deviation

DeArazoza and McLeod **[29]** chose to use speed instead of percent passing delay as the level of service measure for 2-lane sections of U.S. 1 in the Florida Keys. They developed a novel level of service hierarchy based upon deviations from the posted speed limit that was easier for the general public to understand and perceive while driving the highway. The deviations were selected to correspond as much as possible with the thresholds contained in the HCM. The speed level of service measure was also easier to measure in the field than percent time delay.

4.2 Florida DOT Service Volume Method

The Florida Department of Transportation (FDOT) developed a Level of Service Manual **[30]** which consists of generalized level of service tables that planners can look-up to find the maximum service volume, and software that planners can use to create customized service volumes for specific facility characteristics and areas. The following table shows the FDOT Generalized Level of Service Table for peak hour directional volumes for urbanized area. The tables were generated by creating different sets of default input values for each facility and area type, and substituting these defaults into the Highway

Capacity Manual methods. Additional adjustments are made for divided and undivided streets, the presence of left turn bays, one way streets, and the type of area (urbanized, transition, not urbanized and rural undeveloped). The tables are to be used only for preliminary estimates.

Table 2. Florida Generalized Peak Hour Directional Volumes for Urbanized Areas

			Level of Service				
Facility	**lanes**	**Divided?**	**A**	**B**	**C**	**D**	**E**
Freeways (Group 1)[2]	4	n/a	1100	1760	2640	3350	4040
	6	n/a	1660	2640	3970	5030	6340
	8	n/a	2210	3530	5290	6700	8460
	10	n/a	2760	4410	6620	8380	10570
Freeways (Group 2)[3]	4	n/a	1060	1700	2550	3230	3900
	6	n/a	1600	2560	3840	4860	6130
	8	n/a	2130	3410	5110	6480	8170
	10	n/a	2670	4260	6390	8100	10210
State Multi-lane Highways	2	No	460	720	980	1280	1710
	4	Yes	1110	1850	2590	3110	3700
	6	Yes	1670	2780	3890	4660	5550
Class Ia[4] Interrupted Flow	2	No	*	660	810	880	900
	4	Yes	*	1470	1760	1890	1890
	6	Yes	*	2280	2660	2840	2840
	8	Yes	*	2840	3280	3480	3480
Class Ib[5] Interrupted Flow	2	No	*	*	460	760	840
	4	Yes	*	*	1020	1640	1800
	6	Yes	*	*	1550	2510	2710
	8	Yes	*	*	1890	3060	3320
Class II[6] Interrupted Flow	2	No	*	*	*	620	800
	4	Yes	*	*	*	1390	1740
	6	Yes	*	*	*	2130	2640
	8	Yes	*	*	*	2600	3230
Class III[7] Interrupted Flow	2	No	*	*	*	690	780
	4	Yes	*	*	*	1540	1700
	6	Yes	*	*	*	2340	2570
	8	Yes	*	*	*	2860	3140

n/a = not applicable. * = Level of service cannot be achieved.

[2] Group 1 freeways are located within an urbanized area with over 500,000 population and the freeways lead to or are within 5 miles of the primary Central Business District.

[3] Group 2 freeways are freeways not falling within Group 1.

[4] Class Ia arterials have less than 2.50 signals per mile.

[5] Class Ib arterials have 2.50 to 4.50 signals per mile.

[6] Class II arterials have more than 4.50 signals per mile and are NOT located within a primary central business district of an urbanized area with over 500,000 population.

[7] Class III arterials have more than 4.50 signals per mile AND are located within the primary central business district of an urbanized area with over 500,000 population.

4.3 Dowling LOS Criteria for Multi-Facility Transportation Systems

Dowling **[31]** presents LOS criteria that can be used at the system level to estimate system performance. The focus is on highway systems, but a suggestion on how the LOS criteria could be extended to multi-modal analyses and other analyses where cost is a significant factor is included. Dowling recommends that mean system speed and its inverse, mean travel time per mile, be tested for their potential utility as the primary measure of the quality of system performance.

The 1965 Highway Capacity Manual established the concept of "Level of Service" for measuring the quality of the driver's experience on a highway facility. The Highway Capacity Manual used level of service to translate the numerical results of traffic operations analyses into letter grades more readily understandable by the general public. The assignment of letter grades for level of service has been enormously successful, having been incorporated into numerous state and local legal codes for measuring the impacts of new development and monitoring congestion.

The Highway Capacity Manual however provides level of service criteria only for individual intersections, segments, and facilities. There is no guidance provided for combining the results into an overall assessment of system operations in a corridor or large area. Indeed, because the Highway Capacity Manual employs different measures of level of service for different facility types (e.g. density for freeways and speed for arterials), it is not possible to simply sum or average the results for individual facilities to obtain system performance. The Highway Capacity Manual level of service measures in particular do not lend themselves to the analysis of multi-modal systems.

This paper develops a systematized foundation for defining system level of service based on observed driver behavior rather than the attitudinal surveys which have been used in the past. A traveler utility maximization formula, similar to ones used in mode choice analysis, is suggested for computing and comparing level of service between facilities and between different modes within a transportation system. The concept is generalizable to the analysis of toll facilities and congestion pricing.

Mean system travel time and travel cost (which have been shown to be the most important factors affecting travelers' mode choice) are suggested as the primary measures of level of service for transportation systems.

One significant outcome of this behavior based, utility maximization approach to level of service is the conclusion that level of service is relative, and not absolute. The perceived quality of service depends on the alternatives available to the traveler and their previous traveling experiences.

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