Improving Road Link Speed Estimates for Air Quality Models

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A methodology developed to improve travel speed estimates for air quality modeling is described. The proposed methodology, which is applied after the traffic assignment process, consists of an arterial module and a freeway module. It modifies speeds through the use of a new set of congestion functions based on the Davidson formulation, a modified version of the Dowling and Skabardonis postprocessor methodology, and a modified version of Boston's Central Artery/Tunnel freeway queuing procedure. The methodology was applied to the greater Toronto area, and the arterial module of the methodology was validated by using observed speed data from the region. The model improved average travel speed estimates for arterial roads significantly compared with those calculated from the transportation planning model.

The greater Toronto area (GTA), similar to many large conurbations around the world, suffers air pollution problems. In 1989 the ozone air quality criterion (AQC) at the 20 monitoring stations in GTA was exceeded at least once, whereas in downtown Toronto the AQC was exceeded 265 times. Road vehicles and, more specifically, automobiles contributed significantly to the degradation of air quality in the area. In Ontario automobiles have accounted for about 33 percent of the volatile organic compounds (VOCs) and 20 percent of the nitrogen oxides (NO_x), the precursors of ground-level ozone.

In 1990 the Canadian Council of Ministers of the Environment (*1*), in recognition of the negative impacts of elevated levels of ground-level ozone, developed a NO_x/VOC management plan. Improvements in mobile emission inventories and implementation of effective transportation control measures were among some of the initiatives recommended by the plan for nonattainment regions. The achievement of these two initiatives requires an air quality model to be linked to a traffic model, in which the latter provides the air quality model with traffic-related inputs, average speeds, and traffic volumes.

Average travel speeds, representing the driving cycle, have a significant effect on automobile emissions. Hydrocarbon emissions are most sensitive to low speeds [i.e., those that occur at high volume-to-capacity (*V/C*) ratios] and to a lesser extent to very high speeds. In other words errors in speed estimation at high *V/C* ratios have a more significant impact on the accuracy of air pollution emission estimates than those at uncongested conditions.

The use of regional transportation planning models in air quality analysis is essential for two reasons. First, the quality of mobile emissions inventories is improved by using link-level travel characteristics (*2,3*). Second, air quality analysis of ozone requires areawide analysis (*4*) and photochemical models (e.g., the Urban Airshed Model), which are used to conduct such analysis, require traffic-related data for grid cells of 5×5 km. Traffic operations models, on the other hand, could be used to estimate these data, but only for limited scale applications. For example Matzoros and Van Vliet (*5*) applied an air pollution model that they developed coupled with SATURN, the traffic operations model, to the Manchester, England, central area. The model was tested against observed data and showed reasonable agreement. However for large metropolitan areas the use of traffic operations models is not feasible because of their extensive data requirements such as traffic signal data, which include cycle length, green time to total cycle length, sequence of the traffic signal phases, and so on.

The analytical methodologies used to estimate regionwide average travel speeds vary with the air quality modeling technique used. For example TRFCONV (an air quality program developed for use in Phoenix, Arizona, for carbon monoxide emissions and applied in Tucson, Arizona, and Las Vegas, Nevada) (*6*) uses a nominal average daily speed on each link, whereas DTIM (an air quality program developed and used in many areas in California) (*7*) uses *V/C* versus speed curves to estimate speeds. Existing methodologies except for those of Dowling and Skabardonis (*8*) and Dehgani et al. (*9*) do not address the effect of *V/C* ratios of greater than one on speed estimates and the related emissions calculations. The latter two procedures, which are reviewed in more detail in the following sections, apply simple queuing analysis to adjust travel speeds estimated by regional transportation planning models. In addition most of the above procedures use the Bureau of Public Roads (BPR) congestion function or modified versions of it, despite its drawbacks when dealing with arterial streets.

This research, recognizing the importance of valid average travel speed estimates to automobile emission estimates, proposes a methodology for obtaining improved speed estimates. This analytical methodology combines parts of the procedures developed by Dowling and Skabardonis (*8*) and Dehgani et al. (*9*) and uses calibrated congestion functions based on the Davidson's function.

DOWLING AND SKABARDONIS POSTPROCESSOR METHODOLOGY

Dowling and Skabardonis (*8*) have developed a postprocessor methodology to improve average travel speed estimates that result from the transportation planning model calculations. The postprocessor approach was developed in response to the requirement for a more stringent set of air quality guidelines for new highway projects. This methodology uses a modified congestion-flow curve and simple queuing analysis to improve travel speed estimates. Hourly traffic volumes on each link are compared with the corresponding capacity of that link, and if demand exceeds capacity a queue is calculated and average speeds are recalculated. Recal-

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culated speeds are the weighted averages of queue speeds and uncongested speeds, as explained later in this section.

Dowling and Skabardonis have reviewed a number of existing link congestion functions (LCFs), sometimes called *speed-flow curves*, and have shown that the original (BPR) function, which is shown in Equation 1, overpredicts congested travel speeds and does not accurately represent speed-flow conditions when queuing exists. A modified version of the BPR function was fitted to the 1985 *Highway Capacity Manual* (HCM) freeway speed-flow curves, and the modified function had a V/C coefficient of 1 and V/C power of 10 (instead of 4).

$$\text{Speed} = (\text{free flow speed})/[1 + 0.15 \times (V/C)^4] \quad (1)$$

They also investigated the use of the 1985 HCM procedure for determining average speeds on arterials to establish an LCF for arterials. However that attempt was unsuccessful because of the wide variation in estimated average speeds for different green times per cycle, traffic signal cycle times, and V/C ratios. The authors decided to use the modified BPR congestion function for arterials as well.

Because the postprocessor operates after traffic assignment, it must assume that queuing occurs on the same link where demand exceeds capacity. The effect of this assumption on the accuracy of the queuing analysis is more significant for a freeway section than for an arterial section. More details on the spatial treatment of the postprocessor of queuing can be found elsewhere (*8*).

Dowling and Skabardonis (*8*) suggested that the temporal variation in demand be treated by dividing the peak period into a sequence of 1-hr-long time slices. Within each time slice the demand and capacity are assumed to be constant, and for each 1-hr slice the average link speed (AVSPD) is calculated as follows:

$$\text{AVSPD} = \text{QUSPD} \times (\text{QULEN/LEN}) + \text{UNSPD} \times [1 - (\text{QULEN/LEN})] \quad (2)$$

where

QUSPD = average speed in queue = capacity/lane × 7.6 m/vehicle (25 ft/vehicle),
UNSPD = uncongested speed = free flow speed/[1 + $(V/C)^{10}$],
QULEN = average queue length = AVGOU × 7.6 m/vehicle (25 ft/vehicle),
LEN = link length [in km (mi)],
AVGOU = average queue = (queue at start of time slice + queue at end of time slice)/2,
Queue at end of time slice = Q1 + demand rate × (1 hr) − capacity rate × (1 hr), and
Q1 = queue at start of time slice.

The authors used a spacing of 7.6 m/vehicle (25 ft/vehicle) in the calculation of queue length and queue speed. However the more precise (typical) spacing for vehicles queuing on an arterial street at a traffic signal is 6.7 m (22 ft), or a density 150 vehicles/km (240 vehicles/mi) (*10*), and it might be that the authors wanted to use a more conservative value for this calculation.

The postprocessor methodology was applied to the highway network of the city of Hayward, California, and was compared with the output of the traffic operational models FREQ and TRANSYT7F by using the output of their transportation planning model for both a four-lane freeway and a four- to six-lane arterial street. The results of the comparison indicated that the processor methodology improved the average speeds that resulted from the planning model. However the authors pointed out two limitations of this methodology: (a) applying the BPR congestion function with a power of 10, which was calibrated for freeways, to arterials, and (b) applying the 7.6-m (25-ft) spacing value, which is suitable only for arterials, to freeways. The first limitation would result in the overestimation of speeds on arterials, whereas the second would result in the underestimation of congested speeds on freeways.

BOSTON'S CENTRAL ARTERY/TUNNEL FREEWAY QUEUING PROCEDURE

Boston's Central Artery/Tunnel (CA/T) freeway queuing procedure was developed to provide Boston's CA/T project with a tool of reasonable accuracy to assess traffic operations and air quality impacts when congested traffic flows occurred [level of service (LOS) F]. The original formulation of this queuing methodology was developed by J. Kahng and J. Setteducato of Parson's Brinkerhoff Quade and Douglas, Inc., and J. Gluck of Urbitran Associates on the basis of the queuing procedure employed in the FREQ8PE freeway simulation model (*9*). It should be noted that the queuing procedure used in FREQ was originally developed by Makigami and Woodie (*11*). The procedure was subsequently simplified, calibrated, and applied to the CA/T project by Dehgani et al. (*9*).

Queuing analysis similar to that of the Dowling and Skabardonis (*8*) processor was conducted for the freeway segments when the estimated volume exceeded capacity. Because of the sensitivity of the analysis to roadway capacity, Dehgani et al. (*9*) considered capacity as a varying magnitude that changes as roadway geometry and vehicle mix change from section to section. The necessary adjustment was achieved by using capacity reduction factors at weaving areas and isolated merge sections. These capacity reduction factors were the outcome of an iterative process reconciling the 1985 HCM (*12*) weaving areas LOS analysis and the 1965 HCM (*13*) capacity reduction factors for weaving sections. Table 1 shows the capacity reduction figures developed for both weaving areas and isolated merge sections.

By using Table 1 a calibrated unit capacity is assigned to each freeway segment on the basis of its LOS, which is estimated by using 1985 HCM weaving area analysis. Queues are formed when forecast volume exceeds capacity (modified by using Table 1), and the queue lengths are then estimated. To accomplish this the au-

TABLE 1 Capacity Reduction per Lane in Weave Sections and Isolated Merge Sections (*9*)

LOS	Weave Sections	Isolated Merge Sections
C	< 80	80 - 100
D	180 - 220	80 - 100
E	220 - 270	100
F	> 270	100

Note: entries are in vehicles/hour

thors estimated two densities in vehicle per lane-mile values for both jam density (stop-and-go traffic) and capacity density (traffic flowing at capacity). The two density values were 113 and 38 vehicles per lane-km (180 and 60 vehicles per lane-mi), respectively. These values are based on figures given in the 1985 HCM and local observations from Boston. A storage capacity is then derived by subtracting the capacity density from the jam density. The excess demand (QU) forming the queue is divided by the total storage capacity (S_c) to yield the maximum queue length as follows:

$$\text{QULEN} = (\text{QU}/S_c) \times 0.5 \times 1{,}000 \text{ (m/km)} \quad (3)$$

where

QULEN = queue length (in m),
QU = excess demand forming queue per lane, and
S_c = storage capacity = (113 − 38) = 75 vehicles/km (120 vehicles/mi).

It should be pointed out that the result of dividing 1,000 by S_c is a density of 13.3 m (44 ft) per vehicle, which is very close to the value of 12.2 m (40 ft) value per vehicle for stop-and-go movement in the queue used in the analysis of breakdown conditions in Chapter 6 of the 1985 HCM. The 0.5 value in Equation 3 is based on the assumption that the delay per queued vehicle varies linearly from 0 sec for the first vehicle to a maximum value for the last vehicle in the queue.

The final step is to calculate the delay associated with the queue and the average queue speed. This is done by calculating two terms, the unconstrained travel time (Tu) and the average delay time (Td). The unconstrained travel time (characteristic of the flow at capacity) is based on a speed of 48 km/hr (30 mph), which is a threshold speed with LOS F and is calculated as follows:

$$\text{Tu} = (\text{QULEN}/48 \text{ km/h}) \times (3{,}600/1000) \quad (4)$$

The estimation of Td, in seconds, attributed to the queue is calculated as follows:

$$\text{Td} = (\text{Vd}/\text{Cd}) \times (3{,}600)(\text{sec/hr}) \quad (5)$$

where Vd is (first vehicle delay + last vehicle delay)/2 or Vd is QU × 0.5, and Cd is the downstream capacity.

Similar to the unconstrained travel time, the average delay time is based on the assumption that the delay per queued vehicle varies linearly from 0 sec for the first vehicle in the queue to a delay time of 2 Td for the last vehicle in the queue. Finally the queue speed (QUSPD) is calculated by dividing the queue length (QULEN) by the total travel time (Td + Tu) as follows:

$$\text{QUSPD} = [\text{QULEN}/(\text{Tu} + \text{Td})] \times (3{,}600/1{,}000) \quad (6)$$

The CA/T queuing model was applied by using 1987 data as a base year, and it showed reasonable agreement with observed queuing spots as well as queue lengths and speeds.

PROPOSED SPEED IMPROVEMENT METHODOLOGY

The proposed procedure, similar to the above-mentioned procedures, is applied after the traffic assignment process, and its main objective is to improve average travel speed estimates. The proposed methodology combines both of the above-mentioned procedures and avoids some of their limitations. It consists of two modules, namely the arterial module and the freeway module. The arterial module modifies speeds on links with any V/C ratio, whereas the freeway module modifies links with V/C ratios equal to or greater than 1. The arterial module modifies speeds through the use of a new set of congestion functions based on the Davidson formulation and a modified version of the Dowling and Skabardonis (*8*) methodology. On the other hand the freeway module applies a modified version of the CA/T queuing procedure to modify speeds.

Arterial Road Module

The arterial road module uses the same steps followed in the Dowling and Skabardonis (*8*) postprocessor. Queues are formed on links when demand exceeds capacity, and queue lengths and speeds are calculated on the basis of a density value for vehicles queuing on an arterial street at a traffic signal. However the proposed methodology differs from the Dowling and Skabardonis methodology in the following aspects:

1. It uses a new set of congestion functions based on the Davidson formulation instead of the modified BPR function to calculate uncongested speeds in Equation 2. These congestion functions were calibrated on the basis of observed speed and volume data obtained for the Toronto region, and a detailed description of the calibration process is presented later in this paper.
2. A spacing of 6.7 m (22 ft) per vehicle instead of the 7.6-m (25-ft) spacing used by Dowling and Skabardonis (*8*) is adopted. This 6.7-m magnitude, or 150-vehicle/km (240-vehicle/mi) density, was suggested by Pedersen and Samdahl (*10*) to calculate queue lengths and queue speeds on arterials.
3. Average travel speed is calculated as the average of queue speed and uncongested speed [i.e., (QUSPD + UNSPD)/2] rather than the weighted average used in Equation 2. This modification was made to avoid amplifying any errors in speed estimation because of the aggregate nature of the network (as explained below).
4. Because only one peak hour instead of the peak period is dealt with, average queue (AVGQU) is equal to demand minus capacity (similar to QU in the CA/T procedure).

The aspect of dealing with one peak hour implicitly assumes that demand is constant during that hour, and this contradicts findings of several empirical researchers (*14,15*), who have shown that some commuters shift their departure times during peak hours in response to traffic congestion. This limitation warrants further research on how to incorporate the proposed methodology within the transportation planning system and to account for departure time models.

A number of researchers have identified several limitations of the BPR congestion function. These limitations include that the function underestimates travel time as it approaches oversaturation conditions (especially for arterials) compared with the travel times suggested by traffic flow/queuing theory (*8,16*). In addition the function does not have a parameter that accounts for different types of roadways or different types of traffic operations (*17*). The effect of this is that the use of the BPR function, or the modified forms of it, results in an underestimation of the travel time on the road network, particularly on the arterial road network.

Because of the limitations of the BPR function mentioned above and because of the theoretical appeal of the Davidson function because it is based on queuing theory, a new set of congestion functions based on the Davidson formulation, defined in Equation 7, were calibrated and used in the proposed procedure.

$$t = t_0\{1 + J[V/(C - V)]\} \tag{7}$$

where

t = travel time,
t_0 = free flow travel time, and
J = Davidson parameter (unitless), which determines the rate of change of the function.

Figure 1 shows the effect of the Davidson parameter on the shape of the curve.

The J-parameter, which is also called the *delay parameter* or the *quality of service parameter* (1 − J), can differentiate between different roads that have the same capacity and free-flow speeds but that have different midblock edge frictions. Therefore different J-parameters should be calibrated for different facility types, especially because it is not feasible to calibrate a function for each link on the network. The issues and problems associated with the calibration of the Davidson function are discussed by Rose et al. (*18*), Rose (*19*), Tisato (*16*), and Akcelik (*20*).

Traffic studies in which both traffic volume and travel time data were collected together are not available for the Toronto region. The lack of such studies has led several researchers such as Rose et al. (*18*) to consider collecting each of these two data sets separately as the most likely source of data required for LCF calibration.

The original Davidson function was used in the postprocessor rather than the modified Davidson function introduced by Akcelik (*21*), because the queuing analysis within the postprocessor deals with V/C values of greater than 1. The Akcelik modification avoids computational problems of the original function at V/C ratios equal to or greater than 1. However the computational problems for a V/C ratio of exactly 1 had to be solved. This was solved by restricting the traffic volumes to 0.9 of the capacity, because this value produced travel times that were consistent with the travel times calculated from the queuing analysis for a V/C just greater than 1.

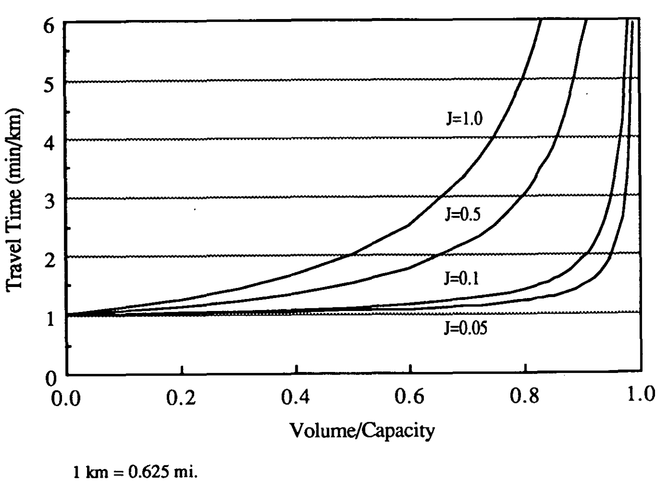

FIGURE 1 Effect of J-parameter on Davidson function.

Matching both sources of data was difficult and time-consuming because traffic volume data are collected on a yearly basis, whereas travel time data are collected for a particular year (1991). Traffic volume information for 1991 was checked against traffic volume information for other years for its reasonableness (i.e., unusually low volumes that occurred because of construction for example). V/C ratios versus average travel speeds were then plotted for different levels of aggregation and scale (e.g., link versus corridor data and a.m. peak versus summation of data for both peak hours). The best-fitting curve between V/C ratio and average speed was obtained by using the data of road segments, both traffic directions (e.g., northbound plus southbound), and both peak hours (i.e., a.m. peak plus p.m. peak). This yielded the expected V/C-versus-speed trend and included V/C ratios of between 0.52 and 0.87. This V/C range covers most of the V/C range identified by Rose (*19*) and Rose and Raymond (*22*) as the most critical for the accuracy of the J-parameter estimation, which is between 0.66 and 1.0.

Nonlinear regression analysis was used to calibrate the Davidson function. Both free-flow travel time and capacity were externally determined as recommended by Tisato (*16*) and Akcelik (*20*) to avoid overprediction of capacity in the calibration process as it occurred to Taylor (*23,24*) in his analysis of the Davidson function. Free-flow travel times and capacities used in the present research were adopted from the Data Management Group (DMG) (*25,26*) road network; their data were collected from the six regional municipalities of the GTA. It should be noted that the capacities used in the calibration were in agreement with the definition of capacity that was recommended by Akcelik (*21*) and Tisato (*16*) for the Davidson function (i.e., the absolute capacity, which is the saturation flow multiplied by the ratio of green time to the total cycle time).

The speed-delay data used for calibration were for Jane Street and Yonge Street, two major north-south arterials in Toronto. The latter street (or more precisely the stretch of street for which the data were available) is a central-area arterial with very intense land use activities on both sides of the road. The other street is a higher-class arterial located outside the central area of Toronto but within metropolitan Toronto. Other speed-delay data were available for a number of central business district (CBD) arterials but could not be used because of the difficulty of matching the data with their corresponding traffic volumes.

Table 2 shows the Davidson J-parameter magnitudes estimated by nonlinear regression analysis. The third J-parameter shown in Table 2 was obtained by a procedure suggested by Rose (*19*) on the basis of the numbers of signals per mile.

The J-parameters listed in Table 2 may be compared with those estimated by Taylor (*23*) for Melbourne, Australia, which ranged

TABLE 2 Estimated Davidson J-Parameters for the Toronto Area

Area	Davidson J-Parameter
CBD	0.211
Metropolitan Toronto	0.187
Outside Metropolitan Toronto	0.170

from 0.350 to 0.486 for arterials. The lower magnitudes obtained from the Toronto data are consistent with the comments by Tisato (*16*), who showed that Taylor overestimated the capacities. Furthermore Rose and Raymond (*22*) showed that the J-parameter and capacities are highly positively correlated. In other words the overestimation of capacity would result in an overestimation of the J-parameter. In addition the J-parameters estimated in the present research are based on corridor data, whereas the J-parameters estimated by Taylor were based on link data, which might also contribute to this difference.

Figures 2 and 3 show the two calibrated congestion functions and the observed data for Yonge and Jane streets, respectively. Figures 2 and 3 indicate that the Davidson function calibrated for Jane Street is much better than that calibrated for Yonge Street.

Freeway Module

The procedure applied in the freeway module is similar to the freeway queuing analysis applied to the CA/T project except for a few changes and minor modifications to suit the nature of the present research. The first difference is that there is no weaving area analysis applied in this module. Instead only one fixed freeway capacity of 1,800 vehicles/hr is used along with only one capacity reduction factor of 200 vehicles/hr, as opposed to several values as shown in Table 1. The 1,800-vehicle/hr value of freeway link capacity was obtained from DMG (*25,26*). The reason for using one as opposed to several capacity reduction values is the size of the freeway network for which the analysis is conducted. The CA/T project consisted of a freeway stretch of less than 16 km (10 mi), whereas in the present research it is the entire GTA freeway network. In other words it is not feasible to apply the 1985 HCM weaving analysis for the GTA network or any other regional network as a first step to estimating the capacity reduction figure as was done for the CA/T project. The single capacity reduction value used is 200 vehicles/hr, which is an average value for the values given in Table 1 for LOSs E and F for both weaving sections and isolated merge sections (i.e., the downstream capacity used in the procedure is 1,600 vehicles/hr).

The second difference is the speed used to estimate the T_u. Dehgani et al. (*9*) used 30 mph as a threshold value for LOS F. However from the 1985 HCM for an 80-km/hr (50-mph) design speed this value should be less than 45 km/hr (28 mph). Since the GTA freeway network includes all three freeway types, an average value of 45 km/hr was used.

The last of these differences is with respect to the calculation of the resulting travel speed. Travel speed estimated by the CA/T procedure is the queue speed calculated from Equation 6, whereas the travel speed estimated by the proposed procedure is the average of queue speed (calculated from Equation 6) and the uncongested speed (calculated from the BPR function shown in Equation 9). This modification has been made to compensate for the underestimation of travel speeds on the freeway links that would occur for the following reasons:

1. Assigned traffic volumes on freeway links were slightly overestimated. Thus the average travel speeds on these links would be underestimated.
2. If the HCM (*12*) speed-flow curves are assumed to represent real life, then the BPR function to the power of 6 with 60-mph free-flow speed, for example, underestimates speeds compared with those estimated by the 112-km/hr (70-mph) design speed [which is compatible with a 96-km/hr (60-mph) free-flow speed] speed-flow curve for V/C ratios of 0.7 and above, as shown in Figure 4.

MODEL APPLICATION AND EVALUATION

The proposed methodology was applied to the GTA road network by using the 1986 Transportation Tomorrow Survey data. The GTA consists of metropolitan Toronto and five surrounding regional municipalities. The number of trips generated by the GTA in 1986 was about 2 million during the 3-hr morning peak period. The GTA network was modeled by using EMME/2, the regional transportation model, and included 127 zones and 3,750 links.

The link congestion functions used in the GTA regional transportation model are modified forms of the BPR congestion function, and they are classified for different road categories. The GTA congestion functions have the following forms:

$$\text{Speed} = (\text{free flow speed})/[1 + 1 \times (V/C)^4] \text{ (for arterials)} \quad (8)$$

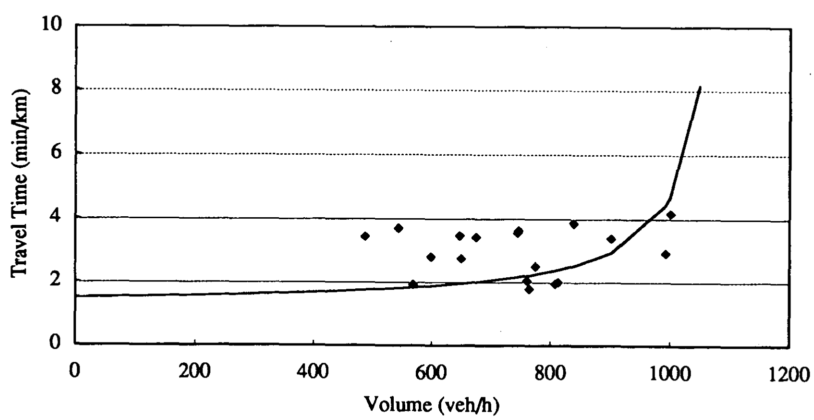

FIGURE 2 Calibrated Davidson function for Yonge Street (J-parameter = 0.211).

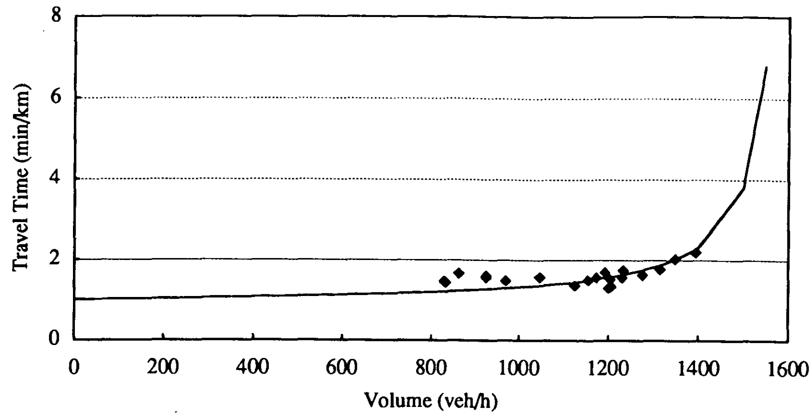

FIGURE 3 Calibrated Davidson function for Jane Street (J-parameter = 0.187).

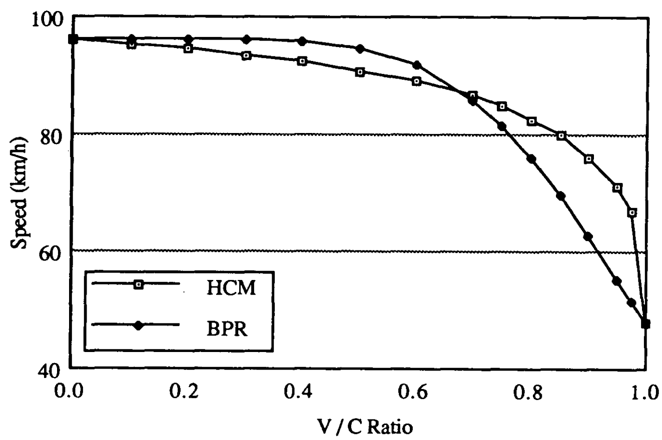

FIGURE 4 Comparison of speed flow curves.

$$\text{Speed} = (\text{free flow speed})/[1 + 1 \times (V/C)^6] \text{ (for freeways)} \quad (9)$$

Different free-flow speeds are assigned to different roads within each category. The power of 6 for the V/C term for freeways was adopted because it improved the assigned traffic volumes on freeway sections (*25*).

Because of the large size of the GTA road network and the constraints of the academic version of EMME/2 used in the present research, parallel major and minor arterials were combined to form traffic corridors (*27*), whereas freeway sections were modeled directly. The capacities of the arterial corridors are the sum of the capacities of parallel major and minor arterials, which have approximately the same link lengths. Similar practice has been reported in the literature by Eash et al. (*28*).

The proposed postprocessor was programmed in FORTRAN and was applied to average speeds and traffic volumes that resulted from EMME/2 applied to the GTA road network. The average speeds estimated by this postprocessor were compared with observed average speeds for a number of CBD arterials and Jane Street, as shown in Table 3.

TABLE 3 Comparison Among Observed, EMME/2, and Postprocessor Speeds

Route	Observed Speed (km/h)	EMME/2 Speed (km/h)	Program Speed (km/h)	Percentage Improvement
Jane St. (NB)	36.6	43.9	38.1	15.8
Jane St. (SB)	41.9	51.4	46.4	11.9
Yonge St. (NB)	20.8	31.9	30.1	8.7
Yonge St. (SB)	26.3	44.3	42.3	7.6
University Ave. (NB)	22.6	25.7	23.1	11.5
University Ave. (SB)	24.3	43.2	36.4	28.0
Jarvis St. (NB)	17.1	49.9	48.1	10.5
Jarvis St. (SB)	30.7	47.2	41.1	19.8

1 km = 0.625 mi.

It should be noted that average speeds presented in Table 3 are average values for routes (extending over 5 to 10 major intersections) rather than individual link speeds. Average speeds along routes have been used because Florian and Nguyen (*29*) have suggested that the average speeds calculated by EMME/2 for routes are quite realistic, whereas there are some discrepancies in the speeds calculated on individual links within a route.

Table 3 shows that average speeds estimated by EMME/2 by using the modified BPR congestion functions are overestimated and that the postprocessor improves these estimates significantly. These improvements (percent difference compared with the observed speed) vary from a low of 7.6 percent to a high of 28 percent, with an average percent improvement of about 14. Since all these routes on average have V/C ratios of less than 1, these improvements may be attributed to the use of the calibrated Davidson function instead of the BPR function.

The performance of the methodology for oversaturated conditions was then evaluated. For arterials the estimated travel speeds ranged between 8.5 km/hr (5.3 mph) and 18.6 km/hr (11.6 mph), with an average speed of 10.1 km/hr (6.3 mph). Observed travel speeds, on the other hand, ranged from 8.2 km/hr (5.1 mph) to 27.7 km/hr (17.3 mph), with an average value of 18.6 km/hr (11.6 mph). This indicates that the methodology underestimates travel speeds on arterials when V/C is greater than 1. This is consistent with the Dowling and Skabardonis (*8*) conclusion that the procedure underestimates speeds. However this conclusion cannot be generalized because of the very few observations for oversaturated conditions available. One additional observation regarding the performance of the arterial module was that for a V/C of greater than 1.4 (which is unreasonably high) the program calculated higher travel speeds than that estimated by EMME/2, and this was the only occasion when the program speeds were higher than EMME/2 speeds.

Since the proposed freeway procedure deals only with sections with a V/C of greater than 1, the speeds estimated by the methodology for V/C ratios of less than 1 are exactly the same as the speeds produced by EMME/2. The speeds calculated by the postprocessor for V/C ratios of greater than 1 ranged from 15 km/hr (9.4 mph) to 32 km/hr (20 mph), with an average value of 25 km/hr (15.6 mph). Unfortunately, speed-delay data were not available for freeways to allow the same comparisons as for the arterials. However the results seem reasonable because the speeds calculated for freeways were higher than those for the arterials and their values were less than 45 to 48 km/hr (28 to 30 mph), which are the threshold values for freeway sections for LOS F in HCM (*12*).

This analysis indicates that no strong conclusions can be made about the overall capabilities of the methodology until comprehensive observations of network flows and speeds are available.

CONCLUSIONS

This paper proposed a methodology that improves link travel speeds estimated by transportation planning models. The proposed procedure applies simple queuing analysis and uses calibrated congestion functions based on the Davidson function. The analysis conducted on the methodology shows that it improves travel speeds significantly compared with those estimated by the transportation planning model. It also shows that for arterials with V/C ratios of less than 1 the procedure overestimates travel speeds, whereas for V/C ratios of greater than 1 it underestimates travel speeds, and the net effect of this might be the estimation of better areawide travel speeds.

Freeway speed observations that would allow for a systematic evaluation of the freeway module are not available. However the results seem reasonable when compared with the arterial module and the 1985 HCM.

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