Simple Method of Predicting Travel Speed on Urban Arterial Streets for Planning Applications

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Travel speed is a key measure of effectiveness in evaluating urban arterials. The *Highway Capacity Manual* (HCM) methodology of predicting speeds along urban streets requires complex calculations and input not typically available in long-range planning. The use of default values is therefore necessary. This paper demonstrates that a simple and practical method of estimating travel speed along urban arterial streets is possible. An equation derived from the HCM delay formula and calibrated with the results obtained from CORSIM, a microsimulation model, is proposed. The equation requires neither signal characteristics nor detailed traffic and geometry information about arterial intersections. The proposed model of travel speed along urban arterials uses only input available to planners. The model is evaluated with the results obtained from a field study in Lafayette, Indiana. Despite its limited scope of input and simple structure, the model properly replicates the trends found in the field. The model overestimates the actual speeds by 18%, and a simple adjustment factor removes the bias.

Chapter 15 of the 2000 edition of the *Highway Capacity Manual* (HCM) provides a method of evaluating traffic conditions along urban streets (*1*). As in the previous versions, the average speed along the analyzed arterial is estimated on the basis of detailed calculations of average delays experienced by through vehicles at each passed intersection. The delay is calculated by using Chapter 16 for signalized intersections. The travel time between the intersections is added to the delays to obtain the total travel time, which is then converted to the travel speed. Although this approach is acceptable for operational analysis and short-range planning, its use in long-range planning applications is difficult. Specific signal parameters, such as cycle, phases, splits, and progression quality, are typically not available in long-range planning, and the use of assumed values that affect the results significantly and may reduce their accuracy is required.

This paper raises an important methodological issue of complexity versus accuracy and is intended to demonstrate that a simple and practical method of estimating average speeds along urban arterial streets is possible. The basis for this optimism comes from existing arterial signals optimization procedures and modern signal control that make control parameters a function of traffic. If this premise is correct, control parameters do not have to be included as explicit input, since they are already implicitly represented by traffic characteristics, arterial geometry, and traffic management policy.

A method practical for long-range planning applications should require only the input available to planners: the number of lanes, the distance between signalized intersections, one-way traffic volumes, and speed limits. To check the feasibility of such a method, an analytical formula for average travel time along a signalized urban street was derived for a noncongested case, as is typically considered in planning. This expression was then calibrated and evaluated with data generated with the microsimulation model CORSIM. Traffic signal timings were obtained from the optimization package PASSER for coordinated signals. Finally, a field study was conducted to compare performance of the model with the speeds observed in the field.

The primary research objective is to check how well a considerably simplified analytical model can replicate results of a complex microsimulation model considered realistic by many researchers and supported by FHWA.

Although several models have been proposed for signalized arterial streets, their use in long-range planning is problematic. The Singapore Center for Transportation Studies (CTS) model (*2, 3*) estimates arterial travel time as a function of traffic density and minimum stopped delay per intersection under free-flow conditions. Because the authors of the CTS model do not provide any method for predicting these two quantities, the model may be used for existing arterial streets but not for future conditions. Other models for travel time for arterial streets, such as those proposed by Davidson (*4*) and Akcelik (*5*), implicitly require control parameters among inputs through the use of the degree of saturation. The degree of saturation is a function of signal timing, which must be assumed for the remote future. In addition, the second type of model was derived from the queuing theory for isolated bottlenecks, which means that the effect of signals progression is not properly incorporated. Therefore, existing models of arterial speeds cannot be easily used for long-range planning. The present paper attempts to fill this gap.

Many existing microsimulation methods lack comprehensive field evaluations of their ability to replicate the operational measures of effectiveness of large urban highway systems, including signalized arterials. This gap is caused by the prohibitively large scope and amount of data required for the validation task. Furthermore, some types of data, such as driver personality and preferences, are difficult to observe or are unobservable.

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EQUATION

An analytical model to estimate speed on arterials will be derived from the HCM delay formula. Average through vehicle travel speed is used as the measure of effectiveness—the only determinant of the level of service for urban arterials, according to the HCM.

Some authors use the inverse of speed, called pace, to model traffic flows (*6*). Pace is equivalent to travel time along a distance unit (for example, 1 mi). The travel pace along an arterial street is the time required to traverse 1 mi under free-flow conditions (free-flow pace) plus the additional delay caused by traffic signals. The free-flow pace is the inverse of cruise speed. The average additional pace in one direction is calculated by summing the control delays at the signalized intersections divided by the total length of the arterial street. Therefore, the total travel pace can be estimated as

$$p = \frac{3{,}600}{V_0} + \frac{\sum_i d_i}{L} = \frac{3{,}600}{V_0} + \frac{n \cdot d}{L}$$
$$p = \frac{3{,}600}{V_0} + \frac{d}{l} \tag{1}$$

where

- p = travel pace (s/mi),
- V_0 = cruise speed (mph),
- d_i = average control delay at signalized intersection i (seconds per vehicle),
- L = total length of the urban street (mi),
- n = number of signalized intersections,
- d = average control delay experienced at a signalized intersection along the urban street (seconds per vehicle), and
- l = average distance between adjacent signalized intersections (mi).

The average control delay is given in the HCM (*1*) by

$$d = d_1(\mathrm{PF}) + d_2 + d_3 \tag{2}$$

where

- d = control delay per vehicle (seconds per vehicle),
- d_1 = uniform control delay assuming uniform arrivals (seconds per vehicle),
- PF = progression adjustment factor,
- d_2 = incremental delay, and
- d_3 = initial queue delay.

In this study noncongested conditions are considered; therefore, the modeling task is reduced to the terms PF and d_1. Although nonzero overflow queue is possible even under noncongested conditions because of short-term traffic fluctuation, this type of queue is lower along arterials than at isolated intersections (*7*) and its estimation requires accurate and detailed information that is not available in the planning applications, which is why the second delay term (d_2) was dropped from the model. Nevertheless, the effect of the nonzero overflow queue can be incorporated indirectly, as explained in the remainder of the paper.

PF accounts for the effects of signal progression, and this factor closely depends on the proportion of vehicles arriving at the intersections during a green signal. This information is obviously not available in the planning phase. It is known, however, that signal optimizers favor directions with stronger flows; thus PF tends to be lower for such directions. A function $1 - a_1F_i/(F_1 + F_2)$, where F_i is the flow in the analyzed direction and F_1 and F_2 are one-way flows along the arterial street, behaves according to this assumption and is used in the postulated PF function. Another well-known fact is that a growing distance between intersections weakens the progression effect because of the growing dispersion of vehicles. The two effects can be modeled with the following postulated function:

$$\mathrm{PF} = \left(1 - \frac{a_1F_i}{F_1 + F_2}\right) \cdot \exp(a_2 l) \tag{3}$$

where a_1 and a_2 are model parameters. According to the HCM, the uniform control delay (d_1) is given as a function of cycle length (C), ratio between effective green time and cycle length (g/C), and degree of saturation (X).

$$d_1 = \frac{0.5C\left(1 - \frac{g}{C}\right)^2}{1 - \left[\min(1, X)\frac{g}{C}\right]} \tag{4}$$

The expression $1 - g/C$ in the numerator of Equation 4 is equal to the proportion of the cycle that is effectively lost, t_L/C, plus the proportion of cycle assigned to the side street as effective green, $(F_s/n_s)/(S_sX_s)$, where t_L is the total lost time in the cycle, C is the cycle length, F_s is the stronger one-way volume crossing the major street, S_s is the saturation flow rate of a single traffic lane, X_s is the degree of saturation of the side street approach, and n_s is the number of through lanes on the side street. Furthermore, because noncongested traffic is considered, $\min(1, X) \cdot g/C = X \cdot g/C = F_i/(n_i \cdot S_i)$, where F_i is the one-way volume in considered direction i, n_i is the number of lanes on the major road in direction i, and S_i denotes the corresponding saturation flow rate per lane. Equation 4 can be rewritten as

$$d_1 = \frac{0.5C\left(\frac{t_L}{C} + \frac{F_s/n_s}{X_sS_s}\right)^2}{1 - \frac{F_i/n_i}{S_i}} = \frac{\frac{t_L^2}{2C} \cdot \left(1 + \frac{C}{t_LX_sS_s} \cdot \frac{F_s}{n_s}\right)^2}{1 - \frac{1}{S_i} \cdot \frac{F_i}{n_i}} \tag{5}$$

The expressions in Equation 5 include variables that are typically unknown in planning: $t_L^2/(2C)$, $C/(t_LS_sX_s)$, and $1/S_i$. In Equation 6, these expressions are replaced with model parameters a_3, a_4, and a_5, respectively. These parameters will be calibrated and will represent the removed variables.

$$d_1 = a_3 \cdot \frac{\left(1 + a_4\frac{F_s}{n_s}\right)^2}{1 - a_5\frac{F_i}{n_i}} \tag{6}$$

Dropping terms d_2 and d_3 in Equation 2, inserting Equations 3 and 6, and reordering and renaming the model parameters yield the following final equation for the control delay:

$$d = a_1 \cdot \exp(a_2 l) \cdot \left(1 - \frac{a_3F_i}{F_1 + F_2}\right) \cdot \frac{\left(1 + a_4\frac{F_s}{n_s}\right)^2}{1 - a_5\frac{F_i}{n_i}} \tag{7}$$

Equation 7 has been derived for a single intersection. It can be applied to an average intersection of the arterial after replacing the variables on the right-hand side with their average values for the arterial street. The effect of the nonzero overflow queue can be partly incorporated through a proper value for the slope parameter in the above expression.

To be consistent with the HCM measure of effectiveness, the average travel speed of through vehicles in direction *i,* which is essentially the reciprocal of the travel pace calculated with Equations 1 and 7, is applied. Therefore, the final expression for the travel speed in direction *i* is

$$V_i = \frac{3{,}600}{p_i}$$

$$V_i = \frac{3{,}600}{\dfrac{3{,}600}{V_0} + \dfrac{a_1}{l} \cdot \exp(a_2 l) \cdot \left(1 - \dfrac{a_3 F_i}{F_1 + F_2}\right) \cdot \dfrac{\left(1 + a_4 \dfrac{F_s}{n_s}\right)^2}{1 - a_5 \dfrac{F_i}{n_i}}} \qquad (8)$$

where

V_i = travel speed in direction *i* (mph),
F_s = flow crossing the major road (vehicles per hour) (select the stronger one-way volume on each side street and calculate the average),
n_s = average number of through lanes in one direction on side streets, and
n_i = average number of through lanes in the considered direction on the major streets.

Other variables are as defined earlier.

SIMULATION

Randomly generated data from a reasonable range of possible scenarios were used to calibrate and evaluate the equation. A field study was also conducted to evaluate the model. Because of its wide acceptance and realistic representation of traffic movements, CORSIM, an FHWA microsimulation model, was used to generate data. Since the model cannot optimize signal settings, the signal timing was optimized with PASSER II-90 (*8*). The signal timing was faithfully implemented in CORSIM. This section describes the simulation experiment design and the results.

The data sets obtained through the use of PASSER II and CORSIM are believed to be reasonable representations of the outcome of the typical signal coordination policy followed by many highway agencies. The policy applies semiactuated coordinated signal timings optimized with PASSER II. Semiactuated coordinated signals are commonly used in the real world and can provide significant improvements in operation compared with the use of fully actuated, noncoordinated, or pretimed coordinated timing (*9*).

A total of 238 cases were generated for the simulation runs. Approximately two-thirds were randomly selected for model development, while the remaining cases served the purpose of model evaluation. Five signalized intersections were defined in each scenario with varying signal spacing and speed limit based on the HCM arterial classification (*10*) as shown below (*6*).

	Criterion	
Design Category	*Signals per Mile*	*Speed Limits (mph)*
High-speed	1–2	45–55
Suburban	1–5	40–45
Intermediate	4–10	30–40
Urban	6–12	25–35

Typical four- or six-lane urban arterial streets with a left-turn bay and four-lane side streets were assumed for all scenarios. The study considers traffic moving along the northbound direction as the arterial forward direction, while eastbound and westbound represent the side street directions.

Table 1 summarizes the input and output values simulated and used in calibrating and evaluating the model. The summary includes descriptive statistics and the correlation coefficients. Correlation coefficients greater than 0.8 exist between *l* and V_0, *l* and simulated *V,* and V_0 and simulated *V.* The strong positive correlation between the average signal spacing and the speed for both cruise speed and simulated speed are due to the method used to generate the data. The statistics reconfirm the well-known fact that densely

TABLE 1 Correlation Coefficients and Descriptive Statistics

Variable	F_i	n_i	F_s	$\frac{F_i}{F_1+F_2}$	l	V_0	Simulated V
F_i	1.000	0.279	0.712	0.385	–0.216	–0.180	–0.377
n_i		1.000	0.227	0.022	–0.202	–0.237	–0.206
F_s			1.000	–0.048	–0.195	–0.127	–0.392
$\frac{F_i}{F_1+F_2}$				1.000	–0.025	–0.063	–0.006
l					1.000	0.802	0.901
V_0						1.000	0.859
Simulated V							1.000
Mean	1,229	2.61	81	50.89	0.44	40.13	27.65
Std. dev.	630	0.49	398	14.38	0.31	7.71	10.45
Count	152	152	152	152	152	152	152
Minimum	235	2	182	20.80	0.09	25	9
Maximum	3,001	3	1,720	79.20	0.99	54	44

spaced signals are associated with lower speed limits (as shown in the table above).

A coordinated semiactuated signal was used in the study. The signal timing parameters required for the simulation included phase sequence, cycle length, signal offsets, and force-off times. These parameters were extracted and computed from PASSER II-90 by generating and optimizing similar network scenarios. The output from PASSER II-90 included vehicular delay and speed estimates. These results were not used in the study because of the following well-known PASSER weaknesses: (*a*) PASSER II-90 does not explicitly model platoons of vehicles; (*b*) discrete, deterministic models were used in the algorithm; and (*c*) no initial queue was assumed at the start of green (*11*). CORSIM was considered a more suitable tool for estimating vehicular delay at coordinated signalized intersections. Signal timing parameters obtained from PASSER II-90 that maximize the overall arterial progression were entered into CORSIM, where a similar set of networks was generated. The results obtained from CORSIM, initially presented by links, were then aggregated by arterial approach to be used for analysis.

CALIBRATION

The maximum likelihood technique was applied to calibrate the model. The estimators under maximum likelihood yield desirable properties that are consistent, asymptotically normal, and asymptotically efficient (*12*). The estimated parameters were then checked for their statistical significance. Finally, to illustrate the performance of the calibrated model, the model was evaluated on the basis of (*a*) simulation experiments and (*b*) actual travel speeds obtained from a field study.

A total of 152 cases were used to develop the model, which establishes a link between through vehicle travel speed on arterial streets and a series of explanatory variables. The normal distribution was assumed for the dependent variable. Equation 9 presents the calibrated model having a standard error of 2.5 mph.

$$V_i = \frac{3{,}600}{\dfrac{3{,}600}{V_0} + \dfrac{8.18}{l} \cdot \exp(0.21l) \cdot \left(1 - \dfrac{0.62F_i}{F_1 + F_2}\right) \cdot \dfrac{\left(1 + \dfrac{F_s}{2{,}000n_s}\right)^2}{1 - \dfrac{7F_i}{10{,}000n_i}}} \qquad (9)$$

The goodness-of-fit index, ρ^2, a measure of the fraction of an initial log likelihood value explained by the model, was 0.81. Figure 1 shows the plots of simulated values from CORSIM and predicted speed from the model. Most of the data points fall close to the 45-degree line, which indicates a good match. No systematic bias (asymmetric spread of points along the line) is visible.

The significance of the model parameters was further tested by examining the likelihood ratio test, which is analogous to the *F*-test used in regression models. In this case, the log likelihood functions are compared for the unrestricted model (L^U) and restricted models of interest (L^R). An unrestricted model is a full model, like the one shown in Equation 9, while a restricted model is the full model with one parameter omitted. The test statistic for the null hypothesis that the restrictions are true is $-2(L^R - L^U)$, which is asymptotically distributed as χ^2 with $K_U - K_R = 1$ degree of freedom; K_U and K_R are the numbers of estimated coefficients in the unrestricted and restricted models, respectively. The results, shown in Table 2, indicated that all the parameters in the model are significant at the 95% confidence level ($\chi^2_{.05,1} = 3.84$) and should be kept.

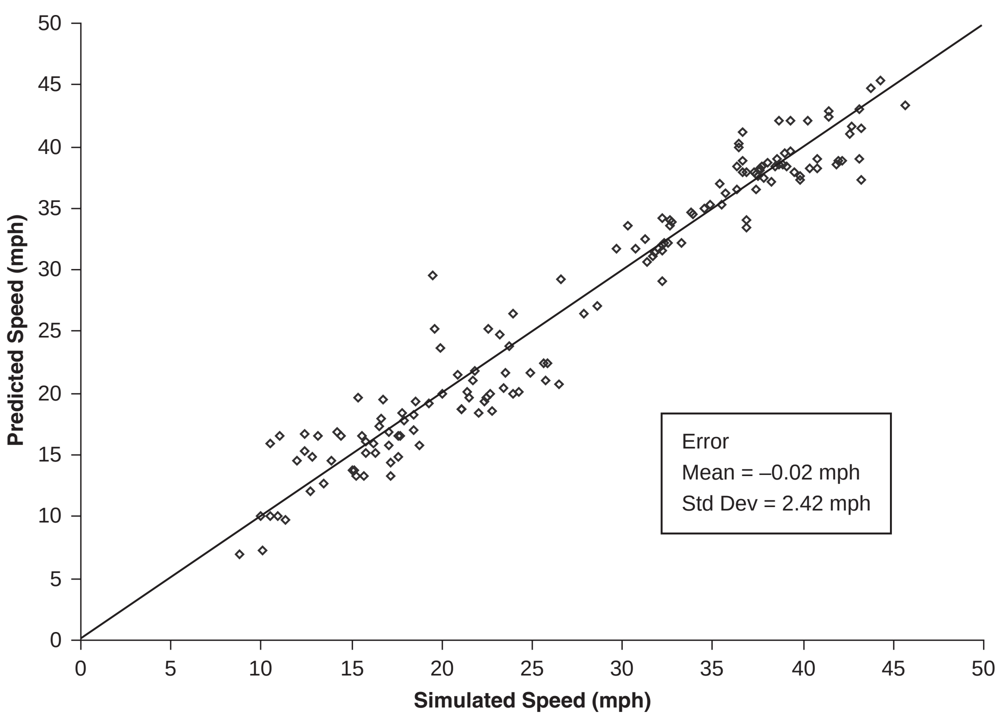

FIGURE 1 Comparison of simulated speed and model estimates.

TABLE 2 Summary of Likelihood Ratio Tests

Parameter	Value	L^R	$-2(L^R - L^U)$	P-Value
a_1	8.18	−401.32	102.7	<.001
a_2	0.21	−352.48	5.02	.025
a_3	0.62	−359.98	20.02	<.001
a_4	0.0005	−362.04	24.14	<.001
a_5	0.0007	−378.37	56.80	<.001

EVALUATION

Simulation Experiments

Eighty-six data points not used to calibrate the model formed a sample to check its predictive accuracy. The speeds calculated with Equation 9 are compared with the speeds simulated with CORSIM in Figure 2. The points spread along the curve in a similar pattern as the calibration data points in Figure 1, and the prediction accuracy is only slightly higher (standard deviation = 2.7 mph) than the calibration results (standard deviation = 2.4 mph). Neither obvious bias nor excessive spread along the diagonal line is visible. Figure 2 indicates that the equation has good predictive capability.

Field Study

A field study was conducted to compare the actual speed with the speed calculated on the basis of Equation 9 along three selected arterials—US-52, SR-26, and SR-38 (Figure 3)—in Lafayette, Indiana. A probe car was driven five times on each arterial in each direction. The following data were collected in the field for each arterial:

1. Distance between intersections,
2. Travel time between intersections,
3. Cruise speed,
4. Number of through lanes on the arterial,
5. Number of through lanes on the side streets, and
6. Traffic volume counts in the peak hour (5 to 6 p.m.) and in the off-peak hour (10 to 11 a.m.).

The speeds of the probe car were recorded on each link when they were not affected by queues and red signals. The average speeds for each direction on each arterial and for the peak and the off-peak hour were used as an estimate of the cruise speed.

Observers at selected locations (Figure 3) counted vehicles during the peak and off-peak periods to estimate arterial traffic volumes along the selected arterials:

- For US-52, between Wabash National and McCarty Lane intersections;
- For SR-26, between Farabee Lane and 36th Street; and
- For SR-38, between Kingsway Street and Maple Points intersections.

The travel speeds on the arterials were estimated by using the total distance and total travel time measurements. The travel speed was calculated for each arterial street, in each direction, and for the peak and off-peak periods. Altogether, 12 observations (3 arterials × 2 directions × 2 periods) of travel speed were obtained. The volumes along the side streets were estimated by converting the

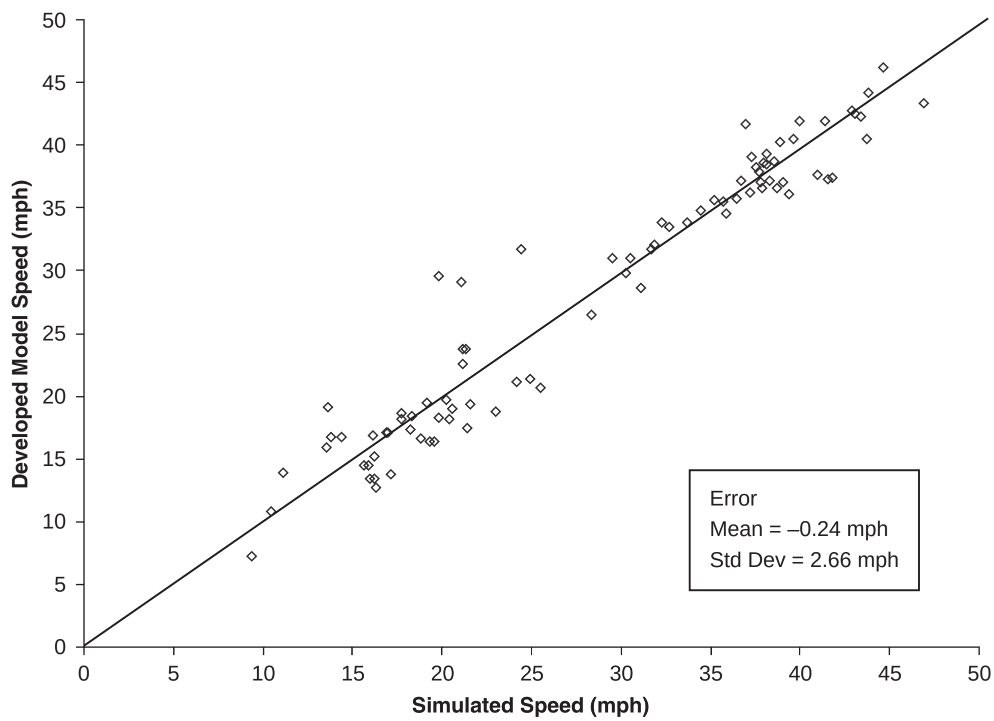

FIGURE 2 Comparison of simulated speed and model speed in calibration.

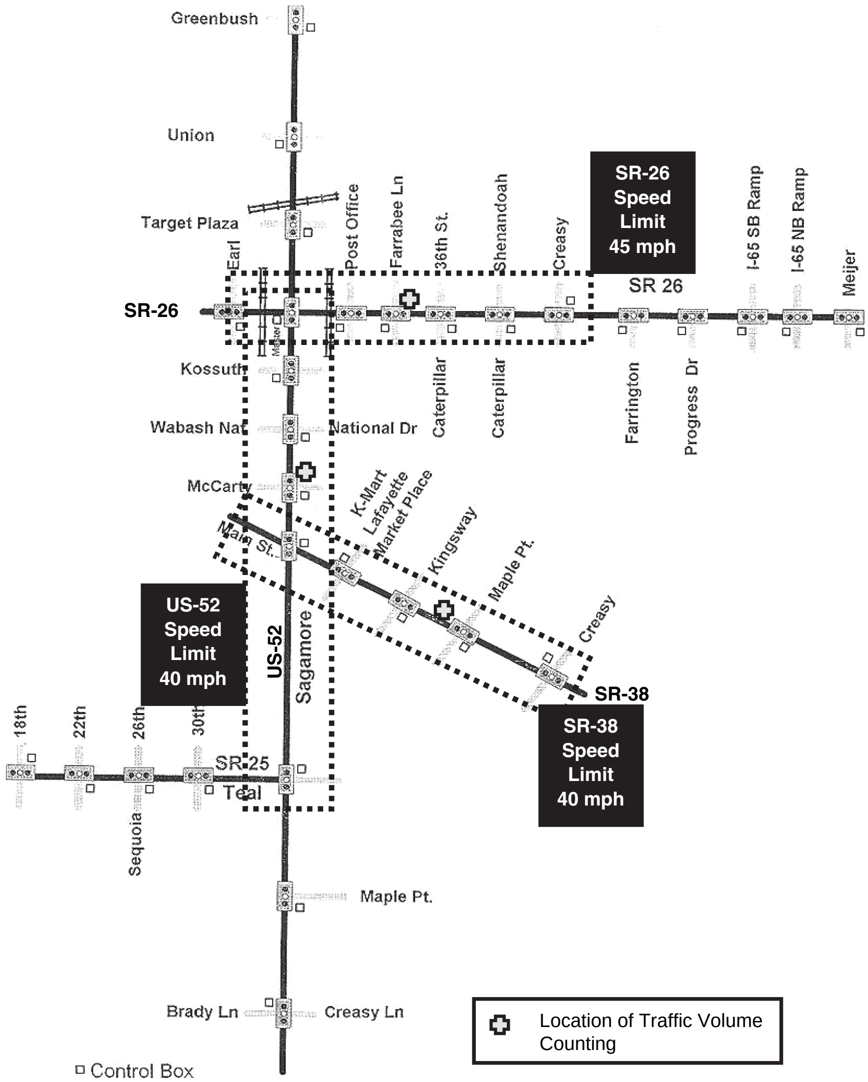

FIGURE 3 Arterial streets chosen for field study.

annual average daily traffic to the corresponding hourly volumes on the basis of conversion factors (*1*). This approach introduced a likely source of error in the model-predicted speed, but the sensitivity analysis indicated that the error is limited.

Table 3 and Figure 4 compare the measured and predicted travel speeds. The model successfully replicates the trend in the results. A high correlation of .911 was found between the model-predicted speed and the actual field speed. Nevertheless, the speed overestimation is noticeable (i.e., 3.9 mph). The simplest adjustment was to subtract this bias from the original predictions. Therefore, the standard error of prediction after this adjustment was 1.8 mph.

Another way to account for the overestimation bias is to multiply the original predictions by an adjustment factor of 0.847:

$$\text{adjusted speed }(\text{mph}) = 0.847 * \text{model predicted speed }(\text{mph})$$

The speeds corrected with the adjustment factor were compared with the actual travel speeds obtained from the field study (Figure 5). A relatively high R^2 of .74 and a standard error of 1.9 mph characterize the performance of the adjusted model, which may be considered satisfactory.

TABLE 3 Summary of Calculated Actual Travel Speed, Model-Predicted Speed, and Adjusted Model Speed

Route	Time/ Direction	Travel Speed (mph)	Cruise Speed (mph)	Predicted Speed (mph)	Adjusted Speed (mph)
SR-26	P-NB	17.90	38.20	20.80	17.62
SR-26	OP-NB	18.30	40.79	24.12	20.43
SR-26	P-SB	13.40	39.23	19.76	16.74
SR-26	OP-SB	20.40	41.10	23.81	20.16
SR-38	P-NB	24.20	39.77	26.43	22.39
SR-38	OP-NB	26.40	38.60	27.49	23.28
SR-38	P-SB	23.30	38.54	26.15	22.15
SR-38	OP-SB	25.30	39.80	28.05	23.75
US-52	P-NB	18.90	41.27	24.74	20.95
US-52	OP-NB	22.20	41.77	27.37	23.18
US-52	P-SB	18.50	37.80	21.99	18.62
US-52	OP-SB	20.94	39.25	25.63	21.71

P = peak; OP = off peak.

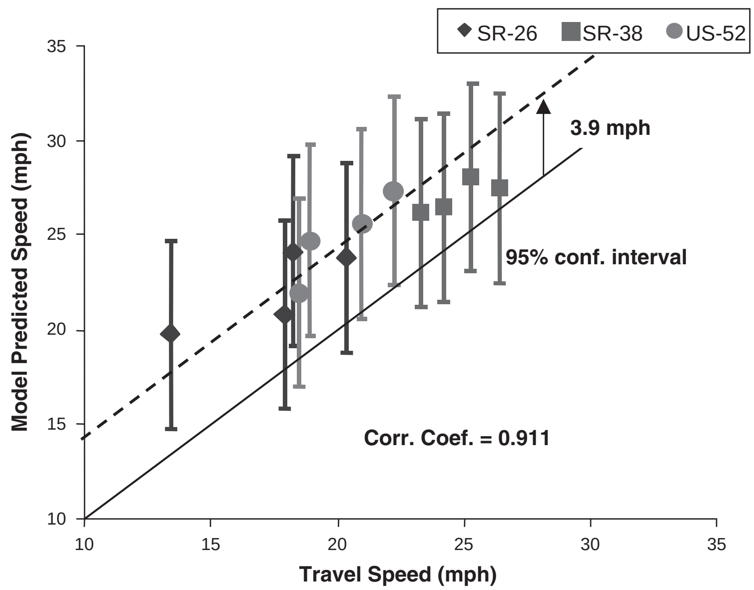

FIGURE 4 Comparison of travel (actual) speed and model-predicted speed.

The overestimation of speed can be related to the assumption of optimal coordination. In the model development stage, the signal timing parameters were optimized by using PASSER II-90 and the obtained parameters implemented in the simulation experiments conducted in CORSIM. The coordination of the three studied arterial streets may not be as good as was originally assumed when the model was developed. Consequently, the actual travel speeds were typically lower than the speeds predicted with the model.

CONCLUSION

The research results indicate a possibility of developing a simple and practical method of predicting travel speed on urban streets for planning purposes. The single equation developed takes inputs available in long-range planning: cruise speed, distance between intersections, number of through lanes, and one-way volumes. Despite the limited input and simple structure of the model, the trend in the predicted speeds matches the trend in the actual travel speeds well.

The assumption of optimal coordination used to develop the model may cause bias. The observed overestimation can be easily removed from the model by subtracting 3.9 mph or by reducing the results by 18% (adjustment factor 0.847). The standard deviation of 1.9 mph after the adjustments is small.

The model incorporates the effect of progression as a function of the directional imbalance of arterial volumes, which is an important contribution to the modeling of urban arterial operations for long-range planning. Incorporating the effect of congestion would be another improvement for other applications. This can be accomplished by adding a delay term.

The cruise speed is the most important input of the model. For the purpose of evaluation, cruise speeds measured in the field were used.

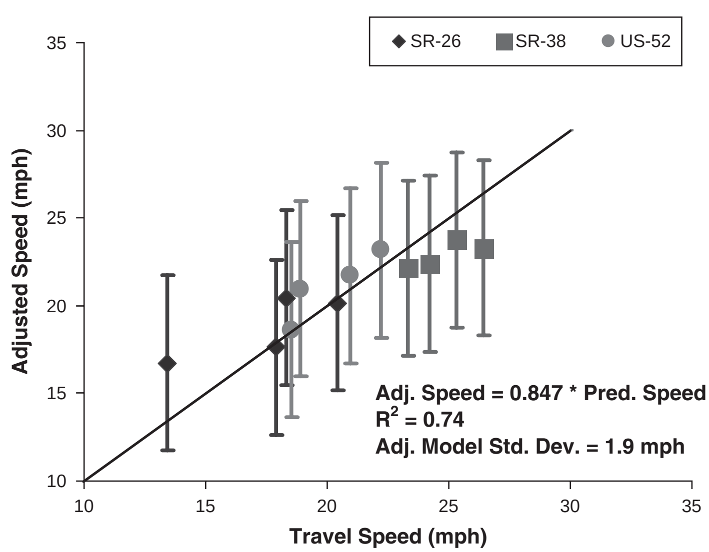

FIGURE 5 Comparison of travel (actual) speed and adjusted model speed.

Of course, planners do not have this option. As an alternative, the speed limit or a value somewhat lower than the speed limit appears to be a good approximation. All the measurements gave values of cruise speed around 40 mph (see Table 3). Two of the selected arterial streets had a 40-mph speed limit, while the third (SR-26) had a posted speed limit of 45 mph. The direction and time of day had a weak impact on the cruise speeds.

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