Link Performance Functions for Urban Freeways with Asymmetric Car–Truck Interactions

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Link performance functions, which capture the relationship between travel time per unit distance and traffic volume per unit time on the links of a network, constitute an essential element in the equilibrium assignment of traffic flows to congested transportation networks. Results are presented of the empirical development and calibration of performance functions that capture the dependence of travel time on the respective volumes of passenger cars and trucks sharing the physical right-of-way on urban freeway sections. The data used for model calibration, individual vehicle trajectories on urban freeway sections, originally collected for FHWA, are developed from a secondary data base. Despite the data limitations, useful relations applicable to a broad range of freeway traffic conditions are developed that yield insights into the effect of trucks on freeway performance. Two types of functions are presented: (a) linear functions over the range of operating volumes extending up to 1,300 vehicles per hour per lane and (b) nonlinear functions, based on the widely used Bureau of Public Roads form, over the full range of flow values. A secondary analysis of the relation between truck and car average travel times is also discussed. These functions are intended for use in network equilibrium studies requiring the assignment of explicit car and truck flows and therefore involving asymmetric interactions between these vehicle classes (or equivalently between links). Such problems arise in the context of the evaluation of truck-related highway improvements, which is a problem of current interest to highway agencies.

Link performance functions constitute an essential element in the equilibrium assignment of traffic flows in congested transportation networks. These functions capture the relationship between travel time per unit distance and traffic volume per unit time on the links of a network. There has been considerable development in theoretical and algorithmic aspects of the network equilibrium problem over the past decade [state-of-the-art reviews have been published by Friesz *(1)* and Sheffi *(2)*]. Recent advances have addressed very general formulations that recognize the presence of multiple user classes interacting in their shared use of the physical right-of-way of the links and, more generally, where asymmetric interactions exist between the network's links. A special case of this problem is that in which separate passenger-car and truck flows must be assigned to a highway network.

The advances have not been accompanied by any significant research into the form and parameter values of the link performance functions, which are essential for the application and further theoretical development of network equilibrium models. The most comprehensive study published after 1975 was conducted by the research group of the University of Montreal, in conjunction with the application of EMME (a multimodal network equilibrium model), using data from the city of Winnipeg *(3)*. Prior to that, Branston's *(4)* work is probably the latest serious investigation of this subject. In neither of these studies, however, was the issue of interaction among vehicle classes addressed.

Judging by the effort invested over the past decade in the network equilibrium problem, it seems surprising that the question of properly specified and calibrated link performance functions has received so little attention in the community of transportation researchers and practitioners. To a large extent, the issue is an empirical one that can only be addressed by using observations of actual traffic behavior on highway links. The sheer size of the data and corresponding cost requirements for a systematic investigation of this problem have probably been serious hindrances.

One particular area in which network equilibrium applications lack any significant observationally calibrated link performance functions is that involving the assignment of cars and trucks, with the interaction resulting from the joint use of roadways by both classes explicitly captured in the link performance functions. Such interactions are asymmetric because the effect of an additional truck in the traffic stream on the average car travel time is different from that of a car on truck travel time. This problem arises, for example, in studies of truck-related improvements in highway networks, as described by Mahmassani et al. earlier *(5)* and in another paper in this Record. This paper presents a first step toward addressing this gap through the calibration of such functions with car and truck flows on freeway sections by using secondary data in the form of individual vehicle trajectories initially collected for an entirely different purpose *(6)*.

This paper is organized as follows. First, the pertinent conceptual background related to link performance functions and truck effects is given. Next, the data are described so that the inherent limitations for this purpose will be understood. In the fourth section, linear functions are presented for the range of operating volumes extending up to 1,300 vehicles per hour per lane to gain insight into the underlying traffic behavior. This is followed by the calibration results for a nonlinear function defined over the full range of flow values. A useful relation between truck and car travel times is then presented, followed by concluding comments.

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BACKGROUND ON LINK PERFORMANCE FUNCTIONS

It is generally recognized that as the volume of traffic on a link increases, so does the average travel time along that link. One difficulty with performance functions for urban freeways is the complex traffic flow dynamics under heavily congested conditions, in which traffic operates in a highly unstable regime. In particular, the presence or absence of queues at freeway bottlenecks affects the travel time associated with an observed volume on a particular section and raises methodological issues in properly defining the observations for studying the relation of interest, as discussed recently by Hurdle and Solomon *(7)* and alluded to by Branston *(4)*. However, it is not the purpose of link performance functions, intended for use in traffic assignment applications, to capture the dynamic aspects of traffic flow on the facility. Traffic assignment models are essentially planning tools, concerned primarily with presumed steady-state conditions. This is an important consideration in defining the observations used for the calibration of these functions.

The dependent variable in the link performance functions of interest here is the average travel time incurred by vehicles traversing a particular link. The average travel time is actually the reciprocal of the space mean speed of the vehicles traveling over the highway section under consideration. This average travel time is related to the prevailing traffic volume per time unit [or "rate of flow" in *Highway Capacity Manual* (HCM) terminology *(8)*]. Of particular concern in this study are the differential effects of the respective car and heavy-truck (and heavy-vehicle) volumes using the link. In current practice, trucks and heavy vehicles are converted into passenger-car equivalents (pce's) using multipliers reported in the traffic engineering literature, particularly in the HCM *(8)*. Values reported in the HCM are intended to capture the amount of "capacity" taken up by a truck relative to a car, and thus do not necessarily ensure that a truck's impact on travel time (or speed) is correctly reflected. Furthermore, considerable debate exists in the traffic engineering community regarding the appropriateness of the 1985 HCM pce values, which apparently tend to underestimate effects of trucks and heavy vehicles *(9, 10)*. In the transportation planning and traffic assignment literature, virtually no effort has specifically calibrated link performance functions with explicit car and truck volumes.

Network equilibrium models require that the link performance functions be monotonically increasing functions of flow, and commonly used solution algorithms require these functions to be continuously differentiable. When the function has more than one argument, such as flows of multiple vehicle classes, it is required that the Jacobian matrix (of first-order partial derivatives with respect to the flow variables) have a positive diagonal. The most commonly used functional form for link performance functions is that of the Bureau of Public Roads (BPR) *(11)*:

$$T = T_0\,[1 + \alpha\,(V/\kappa)^{\beta}] \tag{1}$$

where

T = average travel time per unit distance (at prevailing volume V),
T_0 = travel time per unit distance under free-flowing conditions,
α, β = link-specific parameters to be calibrated, and
κ = "capacity" of the link (pce's per time unit).

In the original BPR form, κ was intended as the so-called "practical capacity" of the link *(11)*. Other researchers have defined it as the "steady-state capacity" [e.g., Steenbrink *(12)*], which is effectively equivalent to the maximum service flow (MSF) corresponding to level-of-service E in 1985 HCM terminology *(8)*. Essentially, κ is a link-specific parameter that could be estimated like the other parameters; this is often inconvenient given the above functional form. The main concern is to use a particular definition consistently for calibration and subsequent application.

When multiple classes of vehicles are present in the traffic stream, say $V_1, V_2, \ldots, V_K$, then the standard approach is to replace the volume V in the foregoing equation by $(\eta_1 V_1 + \eta_2 V_2 + \ldots + \eta_M V_M)$, where η_i is the pce factor for vehicle class i, $i = 1, \ldots, M$. As noted earlier, the source for these pce factors is the HCM, which suffers from the limitations mentioned earlier from the standpoint of link performance modeling.

In this paper, functions of the same basic BPR form but with separate car and truck volume components and no prior restrictions on the pce multipliers have been calibrated and found to provide relatively good agreement with the data. In addition, linear functions, applicable only over a limited range of traffic volumes corresponding to stable, mildly congested conditions, are reported for those facilities for which data were available. Next the data used in this study are described and some of the key features are highlighted.

DATA CHARACTERISTICS

The data used in this study were developed from a large data base intended as a source of information on urban freeway truck characteristics *(6)*, collected for the Federal Highway Administration in 1981-1982 on 11 different freeway facilities in four major metropolitan areas: Houston, Dallas, Atlanta, and Detroit. Because the original intent of the data was to examine the effect of different geometric features on freeway performance, the facilities selected exhibited four different basic geometries: merge, diverge, weave, and basic (pipe) freeway sections. The sections selected are generally characterized by level terrain, to avoid grade-induced complications. All facilities included in the analysis are six-lane facilities (three in each direction), with 12-ft lane width and adequate shoulders and medians.

The data contain more than 0.5 million individual vehicle trajectories, constructed from records of the activation of detectors consisting of low-profile tapeswitches affixed to the road surface and configured in standard traps within the travel lanes. The passage time of each vehicle at each trap is thus available for the duration of the observation periods, which range from 1 to more than 16 hr at the various locations, resulting in grand totals of 561,227 individual vehicle traces observed over 240 hr. With that information, the travel time per unit distance of a vehicle could be obtained by subtracting the time at which the entry trap was activated from the time at the exit trap and dividing the resulting value by the known distance between the entry and exit traps.

As noted in the previous section, the data needed for the calibration of the link performance functions must necessarily be in aggregate form, because the intent is not to explain the considerable variation in individual vehicle performance nor to predict the minute-by-minute dynamics of traffic flow in the facility, but to characterize the effect of a prevailing average volume level on the average travel time experienced by users of the facility. The average volume on the link corresponding to a particular aggregation period is defined as the number of vehicles passing a certain point on the link (typically the entry or exit points) during that period divided by the length of that period. The average travel time per unit distance for that period is then the reciprocal of the space mean speed, as noted in the previous section. The selection of the length of the aggregation period, over which the individual vehicle data are aggregated to form valid observations for performance function calibration, is not a straightforward matter. Ideally, as noted by Branston *(4)*, "long time intervals" must be used in order to approximate steady-state conditions and avoid dealing with the accompanying dynamic phenomena. On the other hand, if the sampling interval is too long, averages might include distinctly different operating conditions. Furthermore, longer intervals might result in fewer observations (in the calibration data set, given a fixed total number of individual vehicle traces), covering a spectrum of operating conditions that is too limited to properly identify the underlying relation. Judgment needs to be applied in this regard given the particular conditions under consideration. In this study, different sampling interval lengths were tested, and an aggregation period of 60 min was ultimately selected. However, 10-min data were also used in some instances where observations would have been too limited or where conditions were sufficiently stable to yield good estimates of the pertinent average quantities.

CALIBRATION RESULTS

It is accepted in traffic engineering practice that average travel time (or speed) on freeways is only mildly sensitive to volume over a relatively wide range of volume levels, beyond which it increases rapidly and nonlinearly. This phenomenon is shown in Figure 1, a scatterplot of the average travel time (per unit distance) versus the corresponding prevailing volume (in vehicles per hour per lane), for the data points obtained at the test locations included in the data base. Similar patterns could be observed for each individual test section, though the extent of data availability across the volume spectrum varied greatly across locations *(13)*. Plots similar to Figure 1 for each section revealed that the first portion of the curve, referred to hereafter as the "linear" portion, extended up to volumes of about 1,300 vehicles per hour per lane. Unfortunately, data points in the "nonlinear" range were very sparse for many of the test locations and could not support the reliable estimation of site-specific link performance functions that apply across the full spectrum of traffic volume levels. This is because high volume conditions were either not attained or not sustained for a sufficiently meaningful period of time at many of the locations

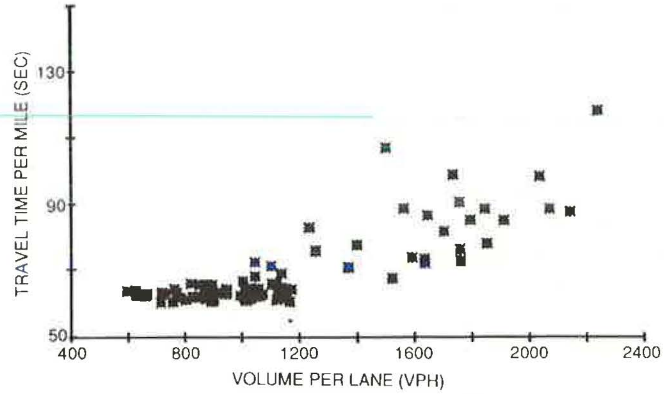

FIGURE 1 Travel-time–volume relationship, all data.

under consideration. Despite the fact that the data set was far from ideal in this regard, it is believed that useful insights and relations could still be obtained from the analysis presented here.

By pooling observations from sections with similar geometries, it was possible to calibrate the desired nonlinear performance functions, applicable over all volume levels, for pipe and merge sections. In addition, more detailed analyses were performed on the "linear" portion only for total vehicular volume levels below the 1,300-vph-per-lane threshold, to gain insight into the effect of trucks on freeway performance. Furthermore, the resulting equations may be of direct use in network assignment applications if the average operating conditions for certain facilities remain in the "linear" range. Relevant results from the analysis of the linear range are presented next, followed by the calibrated nonlinear functions.

Analysis of Linear Portion

The estimation results for three different specifications are reported for the range of vehicular volumes below 1,300 vph per lane:

1. A linear (in parameters as well as in variables) model with both car and truck volumes as the independent variables, as follows:

$$T_i = T_0 + C_1.V_{1i} + C_2V_{2i} + \varepsilon_i \tag{2}$$

where

T_i = average vehicular travel time per unit distance (sec/mi) for the *i*th observation,
V_{1i}, V_{2i} = respective volumes of cars and trucks (vph per lane),
C_1, C_2 = parameters to be estimated, and
ε_i = random disturbance term, assumed, as usual, to be normally distributed with zero mean.

2. A linear model with the total volume in pce's *(V_i)* as the only explanatory variable; it is intended as a byproduct of the analysis to provide a useful model for the "linear" range that

would be very closely compatible with current practice and therefore uses prespecified pce multipliers from the 1985 HCM to convert trucks to pce's. The model therefore has the following form:

$$T_i = T_0 + C.V_i + \varepsilon_i \qquad (3)$$

In this case, a pce factor of 1.7 was found to be applicable according to Table 3.3 in the 1985 HCM *(8)*.

3. A linear-in-parameters specification where the square of truck volume enters the model instead of V_2 in Equation 2. Actually, a number of such intrinsically linear specifications were explored, but only the results from this particular one are worthy of reporting.

The first specification (Equation 2) is the principal one for the purposes of this discussion. Results for the other models are only summarized in this paper. Because operating conditions were relatively stable at these volume levels, 10-min data were used in estimating the foregoing models.

The least-squares estimates for T_0, C_1, and C_2 in Equation 2 for pipe and diverge sections are presented in Table 1. All parameters are statistically significant (different from zero) at any reasonable level of significance, as is the overall regression. In Table 1, T_0, which corresponds to the free mean travel time per unit distance (i.e., reciprocal of the free mean speed), is expressed in seconds per mile; taking the inverse of the estimated values and converting to miles per hour yields respective values of about 63 and 61 mph, which is what one would expect for U.S. urban freeways.

Note that the volume effect coefficients C_1 and C_2 are expressed in seconds per mile per 100 cars or trucks; that is, they capture the expected changes in travel time with a change of link volume by 100 cars or trucks per lane. To formally establish what the numerical estimates for these coefficients strongly suggest, namely, that cars and trucks have different effects on average travel time, the hypothesis that $C_1 = C_2$ was tested for both facility types by using the general F-test for linear models *(14)*. To perform this test, the parameters of the "restricted" model (i.e., with $C_1 = C_2$) are estimated, yielding the sum of squared errors Q^R; similarly, the sum of squared errors for the "unrestricted" model, already estimated, is denoted Q^U. The test statistic is then calculated as $F^* = [(Q^R - Q^U)/r]/[Q^U/(n - k)]$, where n is the number of observations, k is the number of parameters to be estimated in the "unrestricted" model (in this case, $k = 3$), and r is the number of restrictions (in this case, $r = 1$). Under the null hypothesis that the restriction is true, this statistic is F-distributed with $(r, n - k)$ degrees of freedom. The

TABLE 2 RESULTS OF SIGNIFICANCE TESTS OF DIFFERENCE BETWEEN CAR AND TRUCK VOLUME EFFECTS ON TRAVEL TIME ($C_1 = C_2$)

Site Type	$n - k$	r	Q^R [a]	Q^U [b]	F^* [c]	$F_{(0.05,r,n-k)}$ [d]
Pipe	129	1	241.8	233.8	4.41	3.90
Diverge	143	1	235.0	205.0	20.90	3.90

[a] Q^R is the sum of squared errors for restricted model.
[b] Q^U is the sum of squared errors for unrestricted model.
[c] F^* is the calculated value for F-test statistic.
[d] $F_{(0.05,r,n-k)}$ is the theoretical value for F-distributed statistic with r df for numerator and $n-k$ df for denominator at the 5 percent significance level.

results of this test for both types of sections are summarized in Table 2, revealing that, as expected, the null hypothesis should clearly be rejected, and implying that the truck effect on average travel time is in general different from that of passenger cars. Of course, C_1 is considerably smaller in magnitude than C_2. The ratio C_2/C_1 may be interpreted as a "volume effect" pce of trucks in the traffic stream in the volume range under consideration. Note, however, that this pce definition, which is the relevant one from the standpoint of link performance functions, is not altogether consistent with that used to come up with the HCM values. In particular, values of 3.6 and 13.9 are obtained for this ratio for the pipe and diverge sections, respectively, a far cry from the 1.7 suggested by the 1985 HCM for these types of facilities and typically used in current traffic assignment practice.

It should also be noted that the differentiation on the basis of geometric features, as is done in this study between pipe and diverge sections, constitutes a level of detail that is not usually associated with link performance functions in the context of network assignment problems. It was possible here because the data were available in that form. However, in practice it is unlikely that freeway links will be defined in that manner. In that case, the equation calibrated for pipe sections will be the more appropriate one to use.

As noted earlier, the foregoing linear specification was also estimated with the a priori restriction that the ratio C_2/C_1 was equal to the HCM value of 1.7 (i.e., the specification of Equation 3). The results are shown in Table 3. Naturally, because this model is a restricted version of the previous one, it cannot provide a better fit to the data. Furthermore, the volume effect pce values found earlier are quite different from the HCM value of 1.7 that would be used in conventional capacity analysis. The principal reason for including these results here is their potential usefulness in applications where the only infor-

TABLE 1 RESULTS OF PARAMETER ESTIMATION FOR LINEAR MODEL

Section Type	T_0 [a]	C_1	C_2 [b]	R^2	No. of Observations
Pipe	56.86	0.322	1.16	0.471	132
Diverge	58.93	0.205	2.84	0.574	146

NOTE: Range = $V \leq 1{,}300$ vph/lane.
[a] T_0 is expressed in seconds per mile.
[b] The coefficients C_1 and C_2 are expressed in seconds per mile per 100 vehicles.

TABLE 3 RESULTS OF PARAMETER ESTIMATION FOR RESTRICTED LINEAR MODEL WITH HCM pce VALUES

Section Type	T_0	C_1 [a]	R^2	No. of Observations
Pipe	56.36	0.515	0.419	132
Diverge	59.63	0.348	0.523	146

NOTE: Range = $V \leq 1{,}300$ vph/lane.
[a] The coefficient C_1 is expressed in seconds per mile per 100 vehicles.

mation available is given in pce flows; this could arise when the agency providing the data used in network assignment has already applied HCM pce factors, or when future-year forecasts do not break down the projected traffic into its constituent elements.

Given the empirical basis of these (and most other link performance) functions, several alternative specifications have been considered and estimated in the course of this analysis. In particular, specifications including power terms of the two principal independent variables, as well as multiplicative interaction terms, were tested. In general, these specifications were inferior to the simple linear model presented earlier. It is worthwhile to comment on the results of one specification, where the squared value of the truck volume is used, as follows:

$$T_i = T_0 + C_1.V_{1i} + C_2.(V_{2i})^2 + \varepsilon_i \quad (4)$$

where all terms are as defined previously. Table 4 summarizes the parameter estimation results. The model did not exhibit a discernible improvement in terms of statistical performance relative to the earlier linear version (slight improvement for diverge sections, but inferior performance for pipe data). However, its implications appear intuitively plausible and worthy of further examination.

TABLE 4 RESULTS OF PARAMETER ESTIMATION FOR MODEL 3

Section Type	T_0	$C_1{}^a$	$C_2{}^b$	R^2	No. of Observations
Pipe	58.38	0.315	0.212	0.425	132
Diverge	59.77	0.184	2.14	0.597	146

NOTE: Range = $V \leq 1,300$ vph/lane.
[a] C_1 is expressed in seconds per mile per 100 vehicles.
[b] C2 is expressed in seconds per mile per (100 vehicles)^2.

Although the values of T_0 and C_1 are directly comparable to those obtained with the previous model (Equation 2), the interpretation of C_2 is not as straightforward. The assumption here is that the marginal effect of truck volume is proportional to the prevailing volume of trucks, with $\partial T/\partial V_2 = 2C_2V_2$. Other similar assumptions, but with truck effect proportional to car or total volumes, were also considered but did not perform satisfactorily. The values reported in Table 4 indicate that the volume effect pce of trucks (now given by $2C_2V_2/C_1$) can vary over a rather wide range. For example, when truck volume is 100 vph (per lane), the ratio of the (marginal) truck effect to that of cars is 23.26 for diverge sections and 1.34 for pipe sections. Unfortunately, the limited nature of the data precludes more definitive conclusions, but suggests that the truck effect on freeway performance may not be captured very well by the HCM values for certain geometric features.

Nonlinear Performance Function

A nonlinear function of the BPR type, applicable over the full volume spectrum, is presented here. As noted earlier, adequate data for this analysis were available only for pipe and merge type locations. Furthermore, the number of corresponding observations was severely limited, especially for the higher-volume portions of these curves. Nevertheless, the resulting calibrated functions are useful, because they satisfy the desired properties for network equilibrium applications, in addition to providing insights on the functional form and the relative magnitude of the coefficients.

Before the BPR-type specification was estimated, an extensive exploratory analysis was conducted with linear-in-parameters specifications using polynomial regressions in which power terms of independent variables (ranging from first to fifth power), as well as multiplicative interaction terms, were included. The principal general results of this analysis can be summarized as follows: (a) higher-power terms appear to contribute more to explaining the variation in the dependent variable (travel time) than lower-power terms do; (b) different powers of the same independent variable exhibit high correlation with each other; (c) the multiplicative interaction terms are also highly correlated with the other independent variables; and (d) the foregoing leads to at least one negative value among the estimated coefficients whenever more than two of these terms are used; such negative values are not plausible and cannot be accepted in a well-specified model.

Consistent with the foregoing results and with the findings of the earlier analysis for the lower volume range, a BPR-type model was specified as follows:

$$T_i = T_0 \{1 + \alpha[(V_{1i} + \eta.V_{2i})/\kappa]^\beta\} + \varepsilon_i \quad (5)$$

where α, β, and η are parameters to be estimated, and all other terms are as previously defined. Note that the capacity κ, as discussed in the second section, is also a parameter describing the link. In the present analysis, it could not be identified separately; rather, the term $(\alpha T_0/\kappa^\beta)$ was treated as a single parameter value in the least-squares estimation. The value of α was then recovered by setting the value of κ at its HCM value of 2,000 pce's/hr, as discussed hereafter.

Nonlinear least-squares estimators for these parameters were obtained by performing a numerical search for the global optimum over a grid of η- and β-values and using linear least squares to estimate the resulting linear specification for each combination of η- and β-values. The parameter estimates are given in Table 5 for pipe and merge sections separately, and are also pooled for both types of sections. The values obtained in this table are plausible and consistent with prior engineering knowledge. For instance, the values of β (4.7, 4.5, and 4.8) are in the range of 4 to 6 reported by other researchers.

To formally establish that the truck effect on travel time is significantly different from that of cars, an F-test of the hypothesis that the corresponding restriction is true (i.e., that $\eta = 1$) was performed. The test procedure is similar to that used earlier for the linear models, because it is still applicable, with minor adjustments, to the nonlinear case [details on the test procedure in conjunction with nonlinear models have been discussed by Amemiya *(15)*]. The results, presented in Table 6, indicate that the volume effect of trucks is significantly different from that of cars for the pipe sections and for the pooled data, but not for merge sections taken separately. Therefore trucks appear to be more disruptive relative to cars on pipe sections than on merge

TABLE 5 RESULTS OF PARAMETER ESTIMATION FOR NONLINEAR MODEL

Section Type	T_0	$C_1{}^a$	α^b	η	β	R^2	No. of Observations
Pipe	60.62	0.81×10^{-14}	0.438	2.2	4.7	0.908	43
Merge	61.77	0.41×10^{-13}	0.477	1.5	4.5	0.655	65
Pooled Data	61.51	0.38×10^{-14}	0.431	2.1	4.8	0.861	108

NOTE: Applicable over full volume range.
[a] $C = \alpha \cdot T_0/\kappa^{\beta}$.
[b] The value of α reported here is recovered from C assuming that κ = 2,000 pce per hour per lane.

TABLE 6 RESULTS OF SIGNIFICANCE TESTS OF DIFFERENCE BETWEEN CAR AND TRUCK VOLUME EFFECTS ON TRAVEL TIME (NONLINEAR MODEL)

Site Type	$n-k$	r	Q^R [a]	Q^U [b]	$F*$ [c]	$F_{(0.05,r,n-k)}$ [d]
Pipe	39	1	2181.7	1562.9	15.44	4.08
Merge	61	1	1854.0	1849.0	0.17	4.0
Pooled data	104	1	4150	3459	20.78	3.92

[a] Q^R is the sum of squared errors for restricted model.
[b] Q^U is the sum of squared errors for unrestricted model.
[c] $F*$ is the calculated value for F-test statistic.
[d] $F_{(0.05,r,n-k)}$ is the theoretical value for F-distributed statistic with r df for numerator and $n-k$ df for denominator at the 5 percent significance level.

sections. The pce values obtained here also appear to be smaller than the values obtained in the linear model calibrated for the lower volume range. Both conclusions are consistent with the view held by traffic researchers *(9, 16)* that the constraining effect of trucks on travel time is greater at higher speeds than at lower speeds (associated with higher volumes), where vehicles are already operating in a constrained mode.

As noted in conjunction with the models developed for the linear portion, the results calibrated for merge sections are not likely to be useful in the context of traffic assignment applications, because links are rarely defined at this level of detail. The functions calibrated for either pipe sections or pooled data would be more appropriate for such applications.

The functions presented so far yield the travel time per unit distance for an average vehicle. In applications involving the explicit differentiation between cars and trucks as separate user classes, there is concern that the average travel time experienced by cars may be different from that experienced by trucks. This problem is addressed in the next section, in which the results are given of an investigation of the relation between the respective averages for both vehicle classes. This provides the basis for obtaining separate estimates of these quantities given the average vehicular travel time determined from the foregoing performance functions.

RELATION BETWEEN CAR AND TRUCK TRAVEL TIMES

To examine whether the same value for the average travel time applies for both cars and trucks using a given link, the average travel time was calculated for cars and trucks separately for each observation period. In a plot of these averages for all the 60-min data points, which includes the best-fitting straight line (Figure 2), the linearity of the resulting relation between these two quantities is striking. This suggests a simple relation that allows the calculation of the average travel time experienced by either class of vehicles given that of the other or, as in this case, given that of an average vehicle. The following equation was thus calibrated:

$$T_{Ti} = B_0 + B_1 T_{Ai} + \varepsilon_i \tag{6}$$

where T_{Ti} and T_{Ai} are the respective average travel times per unit distance for trucks and cars for the ith observation period and B_0 and B_1 are parameters to be estimated.

The least-squares estimates, based on observations from all section types, are −4.78 for B_0, and 1.075 for B_1 (the corresponding R^2 is 0.80). Both parameters are statistically significant (different from zero) at any reasonable level of confidence. However, the hypothesis that B_1 = 1.0 (against the alternative that $B_1 \neq 1.0$), tested using the standard quasi-t-test, can be rejected at the 11 percent significance level but not at the 10 percent level. A slope of 1.0 would of course imply that average travel time for trucks increases at the same rate as car travel time; the results obtained here seem to suggest that overall, truck travel time increases at a slightly faster rate than that of cars. However, some differences in this pattern were

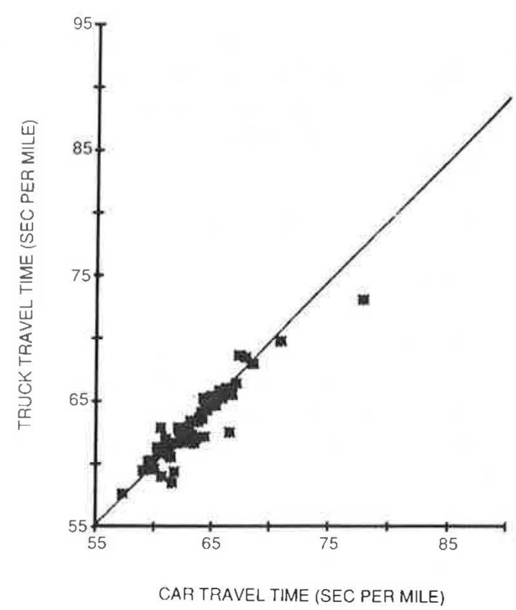

FIGURE 2 Truck versus car average travel time, all data.

observed across different geometric features, though this level of detail is not central to the focus of this paper and is presented elsewhere *(13)*.

The negative value of the estimated intercept B_0 is also worthy of note. Of course, its literal interpretation (i.e., average truck travel time when car travel time is equal to zero) is meaningless in this context, given the range of operating conditions encountered on freeways and in the estimation data base. Essentially, the meaningful range of average travel times to which this analysis is applicable has a lower bound of about 50 sec/mi (corresponding to an average speed of about 72 mph). The reason for the negative intercept is that truck drivers tend to go faster than passenger car drivers when traffic conditions are essentially free flowing, therefore allowing higher speeds (lower travel times per unit distance) to be reached. However, this trend is reversed at lower speeds, when the positive contribution from $(B_1.T_A - T_A)$ offsets the negative intercept (in Equation 6). In other words, when unimpeded, truck drivers go faster, on average, than passenger car drivers; however, their speed deteriorates more rapidly than that of cars with increasing congestion in the facility, given their lower acceleration capability and lack of maneuverability, to where average truck travel time per unit distance exceeds the corresponding value experienced by cars. This effect should be even more noticeable on steep grades, which were not available in this data base.

However, the net travel time differentials between cars and trucks predicted by Equation 6 are minute, and can, for all practical purposes, be ignored in the context of traffic assignment applications. Nevertheless, the equation can be used to calculate the respective average travel times for cars and trucks given the average vehicular travel time obtained by the link performance functions presented earlier. This is accomplished by noting that the average vehicular travel time T is the weighted average of the respective car and truck travel times; that is,

$$T = (V_1/V)T_A + (V_2/V)T_T \qquad (7)$$

where $V = V_1 + V_2$.

If T_A and T_T are related by Equation 6, then substituting this relation into Equation 7 and some algebraic manipulation yield expressions for these two quantities in terms of V_1, V_2, and T (itself obtained from a function such as Equation 5, given V_1 and V_2), as follows:

$$T_T = (B_1 V\,T + B_0 V_1)/(V_1 + B_1 + B_1 V_2)$$

$$T_A = (V\,T - B_0 V_2)/(V_1 + B_1 V_2)$$

However, it would be simpler, and justified in light of these results, to use T for both vehicle classes unless a particular application involves links with unusual geometric features that might more severely bring out limitations in truck performance characteristics.

CONCLUDING COMMENTS

Results have been presented of the empirical development and calibration of link performance functions that capture the dependence of travel time on the respective volumes of passenger cars and trucks sharing the physical right-of-way. These functions are intended for use in network equilibrium studies requiring the assignment of explicit car and truck flows, and therefore involving asymmetric interactions between these vehicle classes (or equivalently between links). This problem arises, for example, in the context of the evaluation of truck-related highway improvements, which is a problem of current interest to highway agencies.

This work is primarily exploratory in nature, given its reliance on less than ideal secondary data initially developed for the microscopic analysis of certain aspects of truck traffic on freeways. Nevertheless, useful relations applicable to a broad range of freeway traffic conditions were developed. These performance functions, based on the widely used BPR form, can be used directly in current traffic assignment models. Useful insights were also obtained regarding the effect of trucks on freeway performance; for example, the estimated parameter values appeared to suggest that there may be differences in the marginal effect of trucks (relative to that of cars) at high versus low speeds.

The work presented here must be viewed as only a first step toward better understanding of the interaction of various vehicle classes in determining the performance of transportation facilities. In the context of equilibrium studies in urban networks, link performance functions that capture interactions among vehicle classes in signalized arterials and urban streets have not received adequate attention. It is for the latter type of facilities that the models used in current practice may be severely underestimating the effect of slow or heavy vehicles, especially in light of the continuing trend toward using larger trucks for urban goods movement.

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