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Foreword

The papers in this Record consider several environmental issues including energy, environmental analysis, air quality, historic preservation, and noise abatement.

In the first group of papers, which addresses energy issues, one paper estimates automobile fuel consumption in urban areas, another analyzes constraints on ownership and home recharging of electric vehicles, and the third describes compressed natural gas fleet conversion and operation.

A major environmental concern continues to be wetlands: the first paper in the environmental section suggests a wetlands monitoring program, and the second proposes a method for identifying, inventorying, and mapping wetlands using aerial photography and geographic information systems. The third paper is a further study of the effects of calcium magnesium acetate, used as a highway deicing chemical, on groundwater. The final paper on environmental concerns analyzes total petroleum hydrocarbons from disposal of road sweepings, vactor sludges, and ditch spoils from highway maintenance operations.

In the third section, papers look at ways to measure and improve air quality and reduce emissions from motor vehicles. Highway network travel data and the accuracy of MOBILE4-based emissions are evaluated. Toll plaza redesign is studied in efforts to reduce carbon monoxide emissions. Another paper proposes a methodology for improving the ability of current traffic travel speed models to estimate emissions from longer highway segments. A companion paper compares vehicular emissions in free-flow and congested traffic using MOBILE4 and the Highway Performance Monitoring System. The next paper puts forth ways to reduce carbon monoxide levels at drive-up facilities.

The final two papers describe Finland's Highway and Traffic Museum and present a field evaluation of the acoustical performance of parallel highway noise barriers.

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PART 1

Energy

Estimating Automobile Fuel Consumption in Urban Traffic

T. F. Fwa and B. W. Ang

A study conducted to develop fuel consumption models for passenger cars in the urban traffic of Singapore is described. The models were developed using multiple-trip travel data derived from a survey involving 114 cars. Each participating car went through three survey cycles; each cycle covered at least 300 km (186 mi). The entire survey generated 11,557 trips. The day-to-day vehicle usage and travel patterns of participants during the period of survey were not affected by participating in the survey. Altogether 16 makes of cars were represented. Three categories of data were recorded: vehicle information, fuel consumption, and trip diary. The fuel consumed in each survey cycle was derived from pump readings. Special measures were taken in survey design and gasoline refueling to ensure sufficient accuracy in fuel measurements. Test results indicate that the survey approach can be used to calibrate a basic average-speed fuel consumption model, and the survey data could be used effectively to identify the effects of vehicle-specific parameters and climatic conditions on fuel consumption.

Many automobile fuel consumption models have been proposed by various researchers. These models can be classified into two broad categories: (a) instantaneous models that relate fuel consumption to the time history of speed variations, and (b) average-speed models that estimate fuel consumption from average speeds of trips (*1,2*). Instantaneous models are suitable for fuel consumption studies that deal with individual intersections or road sections for which detailed information on speed variations can be obtained reliably. Average-speed models are useful for assessing total fuel consumptions in large road networks for which detailed vehicle and journey information for individual trips is not readily available.

This study adopts the average-speed approach to model automobile fuel consumption in Singapore City. Fuel consumption models are established using aggregated multiple-trip travel data derived from a survey involving 114 cars. A unique feature of this survey was that no special-purpose fuel consumption measurement device was instrumented to measure the fuel consumed in each individual trip. Instead, the fuel consumed by each vehicle was derived from pump readings when the fuel tank of the vehicle was filled during the survey period. The feasibility of using this approach to calibrate a fuel consumption model for actual traffic conditions is examined in this paper.

BASIS FOR CALIBRATION OF FUEL CONSUMPTION MODELS

This section explains the extension of basic average-speed models for single trips to cover the multiple-trip survey of this study in which only the final total fuel consumptions of sets of multiple trips are known. This forms the basis on which the field survey of this study was designed to provide the necessary data for calibrating fuel consumption models.

The most basic form of average-speed model for automobile fuel consumption in urban traffic conditions correlates fuel consumption with the reciprocal of average journey speed (*3–6*):

$$F = a_0 + \frac{a_1}{V} \tag{1}$$

where

F = average fuel consumed for a unit distance in a trip,
V = average speed of the trip, and
a_0, a_1 = vehicle-specific coefficients.

Coefficient a_1 is commonly taken to be an estimate of vehicle idle fuel rate, and coefficient a_0, to be related to fuel required to overcome drag, inertia, and grade resistance (*5–8*). Equation 1 has been found to give satisfactory estimates of fuel consumption for journey speeds between 10 and 60 km/hr (approximately 6 and 37 mph).

In addition to the speed variable, studies have also shown that fuel consumption in urban traffic conditions can also be correlated with vehicle characteristics, such as engine capacity or vehicle mass, in an approximately linear relationship (*5,7,8*). One may, therefore, estimate fuel consumption per unit distance from the following general expression:

$$F = a_1 + \frac{a_1}{V} + a_2 K \tag{2}$$

where coefficient a_2 and variable K represent the effect of vehicle operational characteristics.

Equations 1 and 2 provide an estimate of the fuel consumed by a vehicle in a single trip. For a trip i with trip length and average speed equal to L_i and V_i, respectively, the total fuel consumed for the trip is

$$F\,L_i = a_0 L_i + a_1 \frac{L_i}{V_i} + a_2 K\,L_i \tag{3}$$

The fuel consumed by the same vehicle for a total of N trips is given by

$$\sum_{i=1}^{N} F\,L_1 = \sum_{i=1}^{N} a_0 L_i + \sum_{i=1}^{N} a_1 \frac{L_i}{V_i} + \sum_{i=1}^{N} a_2 K\,L_i \tag{4}$$

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Fuel consumption rate F can be derived by dividing the expression in Equation 4 by the total distance traveled, L_T, as follows:

$$F = \frac{1}{L_T}\left\{\sum_{i=1}^{N} F\, L_i\right\}$$

$$= a_0 + a_1\left\{\frac{1}{L_T}\sum_{i=1}^{N}\frac{L_i}{V_i}\right\} + a_2K \tag{5}$$

The second term in the right-hand side of Equation 5 can be simplified as follows:

$$a_1\left\{\frac{1}{L_T}\sum_{i=1}^{N}\frac{L_i}{V_i}\right\} = a_1\left\{\frac{\frac{L_1}{V_1}+\frac{L_2}{V_2}+\cdots+\frac{L_i}{V_i}+\cdots+\frac{L_N}{V_N}}{L_T}\right\}$$

$$= a_1\left\{\frac{t_1+t_2+\cdots+t_i+\cdots+t_N}{L}\right\}$$

$$= a_1\left\{\frac{t_T}{L_T}\right\}$$

$$= \frac{a_1}{V_T} \tag{6}$$

where

t_i = travel time of trip i,
$t_T = \Sigma_{i=1}^{N} t_i$ = total sum of travel times of N trips, and
$V_T = \frac{L_T}{t_T}$ = average speed for N trips.

Equation 5 can therefore be represented simply as

$$F = a_0 + \frac{a_1}{V_T} + a_2\,K \tag{7}$$

The coefficients a_0, a_1, and a_2 in Equation 7 are identical to the corresponding coefficients of Equation 2. This is of practical significance because virtually all controlled fuel consumption experiments reported used trip-specific data and trip-specific relationships (such as Equation 2) to calibrate fuel consumption models. The given derivation shows that the same form of models could also be calibrated from Equation 7 using aggregated fuel consumption and travel data that cover multiple trips.

The use of Equation 7 allows the volume of fuel consumed to be measured after many trips have been made. This reduces considerably the level of accuracy needed to achieve in each fuel volume measurement. The fuel consumption rate, F, in Equation 7 is computed by dividing the total fuel consumption over N trips by the total distance traveled in as many trips. The average speed, V_T, is obtained as the ratio of total distance traveled to the sum of travel times of all N trips. It can be shown that V_T is numerically equal to the travel-time weighted average of trip average speeds.

DESIGN OF SURVEY

A major objective of the present study was to calibrate an aggregate average-speed fuel consumption model for the urban traffic of Singapore based on actual travel patterns of survey participants in actual traffic conditions.

The entire design of the survey revolves round the choice of a so-called survey cycle. A survey cycle defines the time period over which appropriate aggregated fuel consumption and travel characteristics data are measured. Because no special fuel measurement device was used in this study, and because the amount of fuel consumed was to be derived from pump readings, a logical choice of survey cycle was one that began from the time the fuel tank of a study vehicle was filled to the time of the next fill-up.

Length of Survey Cycle

It is desirable to have a survey cycle cover more than a week so that the effects of weekday and weekend travel would both be incorporated. Too long a survey cycle is undesired as it may impose undue stresses on the participants. From considerations of the travel patterns of local car users and the fuel tank capacity of normal makes of car in Singapore, it was decided that a survey cycle should cover a total travel distance of at least 300 km (186 mi). For most survey participants, a survey cycle could be completed between 1 and 2 weeks. On the average, each cycle would consume about 30 L (7.93 gal) of gasoline.

Recording of Survey Data

Each survey participant was required to go through three consecutive survey cycles. In each cycle, three categories of data were recorded: vehicle information, fuel purchase, and trip diary. The vehicle information covers year of manufacture, original vehicle registration date, make, model, engine capacity, curb weight, type of transmission and steering, air conditioner, and tire type. Fuel purchase data were recorded during each visit to the service station for filling up the fuel tank. The data entered were unit fuel price, total purchase price, total purchase volume, petrol brand and grade, vehicle odometer reading, and time and date of purchase. The trip diary contains detailed information of each trip in a survey cycle. The trip information includes the date, the times and odometer readings at the start and the end of each trip, the number of adults and children carried, estimates of the proportions of trip distance by type of road taken, and trip purpose.

A trip was considered to have commenced once the engine was switched on. The time period during which the vehicle engine runs continuously, inclusive of idling and moving phases, defines the travel time of the trip. The travel distance of each trip is derived from odometer readings in the trip diary.

Fuel Consumption Measurement

The quantity of fuel consumed in each survey cycle of a vehicle is equal to the amount of fuel needed to fill its fuel tank again at the end of the trip. As illustrated in Figure 1, the study required a survey cycle to begin and end at a service station at which refueling took place. For each participating vehicle, there were four refueling sessions. The first refueling established the so-called full-tank condition that defined the fuel level that each refueling must reach. The amounts of fuel consumed in the first, second, and third cycles were equal to the respective quantity purchased in the second, third, and fourth refuelings.

In view of the importance in ensuring that the same fuel level was reached in each refueling, a set of instructions was specially prepared describing in detail the steps to be taken during refueling. Each participant was asked to identify visually a reference mark on the wall of the outlet tube leading to the fuel tank. Refueling to this reference mark was to be achieved in a two-step procedure. First, participants were to start refueling by selecting the full-tank option of the gasoline pump. At the end of this step, when the pump stopped automatically, the fuel tank would still be able to take in another 1 to 2 L of gasoline. The second step therefore involved manually operating the pump to bring the fuel to the reference mark.

Survey Participants

The survey required considerable effort on the part of participants to record the details of every trip over 3 to 5 weeks. Great emphasis was therefore placed on selecting survey participants to ensure that quality survey data were returned. It was on this ground that the target population of survey was restricted to car owners with tertiary education qualification and who drove their cars themselves. The study included a random sample of 114 willing participants. The sample is believed to be representative of the middle-income car-owners of Singapore. It is important to note that the participants' day-to-day travel patterns and vehicle usage during the period of survey were not affected as a result of participating in the survey.

ANALYSIS OF SURVEY DATA

Because each participant went through three survey cycles, the entire survey produced a total of 342 data points for the calibration of Equation 7. The total number of trips included was 11,557. This means that there was an average of about 34 trips in a survey cycle. Altogether 16 makes of car were represented. Table 1 presents the frequency distribution of engine capacity of the cars that participated in the survey. The frequency distribution appears to provide a fair representation of the size of cars found on Singapore roads. A report by the Singapore Registry of Vehicles showed that at the end of 1989, 95.9 percent of the passenger cars and vans in Singapore had less than 2000 cm^3 in engine capacity (*9*). The main features of the travel characteristics revealed by the survey are depicted in Figure 2. Each of the three frequency distribution plots are derived from 342 data points, one point

TABLE 1 Frequency Distribution of Cars by Engine Capacity

Engine Capacity	No. of Cars	Percentage
≤ 1,000 cm^3	29	25.4
1,001 to 1,600 cm^3	81	71.1
1,601 to 2,000 cm^3	4	3.5

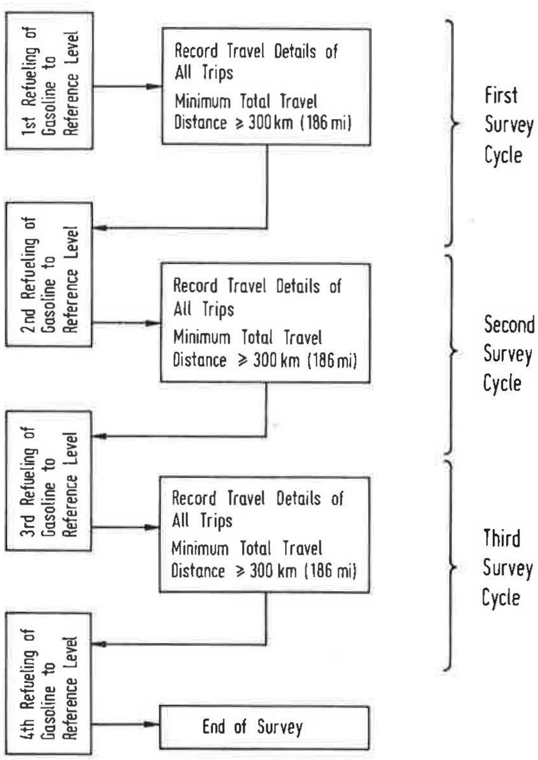

FIGURE 1 Main components of survey procedure for each participant.

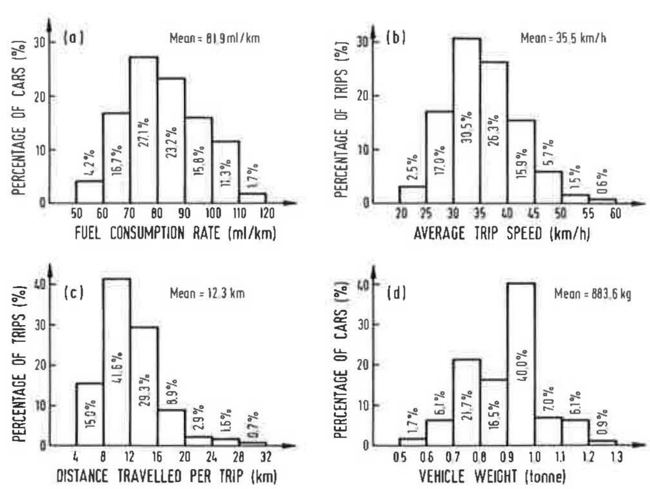

FIGURE 2 Frequency distributions of travel characteristics of cars surveyed.

TABLE 2 Summary of Statistics of Principal Variables

Variables	Mean	Standard Deviation	Minimum Value	Maximum Value
Fuel Consumption (ml/km)	82.0	13.8	50.3	118.3
Engine Capacity (cm^3)	1,293.4	248.5	665	1,998
Vehicle Weight (kg)	883.7	136.8	570	1,375
Trip Average Speed (km/h)	35.73	6.91	20.32	61.27
Passenger Load	1.74	0.487	1.00	4.75
Age of Car (years)	4.26	2.90	0.14	9.89
Odometer Mileage (km)	80,840	58,130	2,180	243,600

from each of the 342 survey cycles. The fuel consumption computed lay between 55 and 120 mL/km (approximately 20 and 50 gal/1,000 mi). The average speed varied from 20 to 60 km/hr (approximately 12 to 38 mph). Table 2 summarizes the statistics of the principal variables considered in this study.

Calibration of Basic Average-Speed Model

A regression analysis was performed to fit the basic average-speed model defined by Equation 1 to the survey data. The resulted regression equation is

$$F\,(\text{mL/km}) = 54.2 + \frac{1{,}009.8}{V_T\,(\text{km/hr})} \qquad (8)$$

or

$$F\,(\text{gal/1,000 mi}) = 23.0 + \frac{266.7}{V_T\,(\text{mph})} \qquad (9)$$

These equations have coefficients of correlation $r = .65$ and $r^2 = .42$. The relatively low r^2-value is not unexpected, because the model represented by Equation 1 is defined for a single vehicle type, whereas the survey data were derived from 114 vehicles. The regression equations therefore serve merely as an assessment of the aggregate fuel consumption characteristics of the sample of vehicles studied. Each of these vehicles was likely to have a different idle fuel consumption rate (and hence different values of coefficient a_1) and a different vehicle weight and engine capacity (and hence different values of a_0). There were therefore inherent spreads of fuel consumption performance among different vehicles. Although aggregate models such as Equations 8 and 9 cannot be used for accurately describing the fuel consumption characteristics of individual vehicles, they are useful for assessing the impact of traffic management policies on the fuel consumption of the entire traffic stream (*1,2,10*).

It is of interest to compare the values of coefficients a_0 and a_1 derived above with those obtained in other studies. Table 3 summarizes the results of a number of studies reported in the literature. It is seen that the value of a_0 varies from 43.4 to 121.8 mL/km, and the value of a_1 from 983 to 3780 mL/hr. The factors contributing to these variations include differences in vehicle design, traffic flow conditions, layout of street networks, road conditions, and drivers' behavior.

Two different forms of fuel consumption models are found in Table 3: vehicle type-specific models and traffic stream aggregate models. The vehicle type-specific models suggest that, in general, both a_0 and a_1 values increase as vehicle size becomes bigger. Equations 10, 12, and 17 in Table 3 are aggregate models that provide estimates of fuel consumption for car populations of the Netherlands, Australia, and the United States, respectively. These three models are plotted in Figure 3 along with the Singapore model derived in this study. From a comparison of the four models, the following observations can be made:

- The overall idle fuel rate (i.e., coefficient a_1) of an average car in Singapore is of the same order as the rates of small-capacity cars represented in Equations 11, 15, and 19 (see Table 3). It is considerably lower than an average car in the United States and Australia. It compares better with models in the Netherlands (see Equations 10a and 10b).
- A comparison of coefficient a_0 of the models also leads to a similar conclusion. It may be said that the drag, inertia, and grade resistance encountered by an average car in Singapore is comparable to its counterpart in the Netherlands but much lower than that in Australia or the United States.
- In general, the fuel consumption model for an average car in Singapore approximates those derived for small cars in other studies. These include the small car model (Equation 11) derived by Biggs and Akcelik for Australian cars (*2*), and Equations 15 and 19 derived by Everall (*8*) and Evans and Herman (*12*), respectively.

Further Analysis of Average-Speed Model

A verification of the average-speed model and an assessment of the validity of the fuel consumption survey procedure adopted in this study can be made by incorporating vehicle-specific variables in the analysis. Assuming that the idle fuel consumption rate of a vehicle is linearly related to its engine capacity (*4*), the following covariance model may be used:

TABLE 3 Values of Coefficients in Average-Speed Model of Equation 1

Study	$F(ml/km)=a_0+\frac{a_1}{V\ (km/h)}$	Eq.No.	Remarks
OECD [1]	$F = 43.4 + \frac{1359}{V}$	(10a)*	Developed in 1981, average for cars on main roads of Netherlands.
	$F = 64.5 + \frac{1166}{V}$	(10b)*	Developed in 1981, average for cars on other roads of Netherlands.
Biggs and Akcelik [2, 7]	$F = 59.0 + \frac{1280}{V}$	(11)	Developed in 1985, for small cars in Australia.
	$F = 73.8 + \frac{1600}{V}$	(12)*	**Developed in 1985, for medium cars in Australia, recommended as default model for all cars in general urban environment.**
	$F = 88.6 + \frac{1920}{V}$	(13)	Developed in 1985, for large cars in Australia.
OECD [6]	$F = 70.0 + \frac{990}{V}$	(14)	Developed in 1980, for car model Renault R12.
Everall [8]	$F = 56.4 + \frac{1159}{V}$	(15)	Developed in 1968, for Vauxhall Viva, engine capacity 1057 cm^3.
	$F = 85.0 + \frac{1913}{V}$	(16)	Developed in 1968, for Ford Zephyr, engine capacity 1703 cm^3.
FHWA [10]	$F = 85.2 + \frac{2825}{V}$	(17)*	Developed in 1981, average model for USA cars.
Chang et al [11]	$F = 112 + \frac{3780}{V}$	(18)	Developed in 1976, for test car, engine capacity 6600 cm^3.
Evans and Herman [12]	$F = 45.6 + \frac{983}{V}$	(19)	Developed in 1978, for test car, vehicle weight 1035 kg.
	$F = 121.8 + \frac{2420}{V}$	(20)	Developed in 1978, for test car, vehicle weight 2488 kg.

Note: Equations marked with * are aggregate models for the entire traffic stream considered; other equations are models derived especially for specific vehicle types.

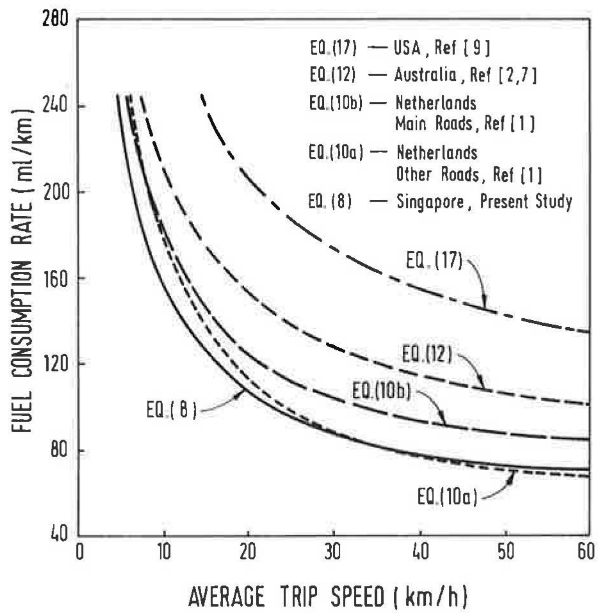

FIGURE 3 Comparison of aggregate fuel consumption model of Singapore with models of other countries.

$$F = b_0 + b_1\left(\frac{E}{V_T}\right) + \sum_{i=1}^{113} (b_2)_i X_i \tag{21}$$

where

X_i = indicator variable that can assume a value of 0 or 1, for i = 1, 2, ... 113;
E = engine capacity (cm^3 or in^3);
b_0, b_1 = regression constants; and
$(b_2)_i$ = regression constants, for i = 1, 2, ... 113.

In Equation 21, X_1 to X_{113} are indicator variables that isolate the effects of the 114 vehicles surveyed. For any given data set, only the one X_i that represents the vehicle from which the data set was obtained can have the value 1; all other X_i terms equal 0. In other words, the fuel consumption of vehicle i is given by

$$F_i = b_0 + b_1\left(\frac{E}{V_T}\right) + (b_2)_i \tag{22}$$

Comparing Equations 1 and 22, it is seen that $a_0 = b_0 + (b_2)_i$ and $a_1 = b_1E$. Equation 22 can therefore be seen as a collection of fuel consumption models meant for the individual vehicles surveyed in the study.

Fitting the survey data to the regression model of Equation 21 yields a coefficient of multiple determination, r^2, equal to .89. This result lends support to the average speed model of Equation 1 and the derivation of Equation 7. It also suggests that the multiple-trip survey technique employed in this study could be effectively used to calibrate fuel consumption models.

Extended Average-Speed Models

The model of Equation 22 has limited practical application because the use of indicator variables X_i could only differentiate the effects of individual survey vehicles. It does not identify the physical parameters that are responsible for the differences in the $(b_2)_i$ coefficients, hence the differences in fuel consumption of the vehicles surveyed. Instead of using qualitative variables, a number of researchers suggested that the drag, inertia, and grade resistance experienced by a vehicle was proportional to its weight (*1,5,7,11*). This means that we can replace Equation 22 with the following model:

$$F = b_0 + b_1\left(\frac{E}{V_T}\right) + b_2 W \tag{23}$$

where W is the vehicle weight in kilograms or pounds.

Fitting the survey data to Equation 23 by regression analysis leads to the following expression:

$$F\,(\text{mL/km}) = 33.6 + 0.7639\,\frac{E\,(\text{cm}^3)}{V_T\,(\text{km/hr})} + 0.0226\,W \quad (\text{kg}) \tag{24}$$

or

$$F(\text{gal/1,000 mi}) = 14.3 + 3.3065\frac{E\,(\text{in}^3)}{V_T\,(\text{mph})} + 0.0044\,W \quad (\text{lb}) \tag{25}$$

Taking W as the unladen vehicle weight in this expression yields an r^2-value of .54. Adding passenger loads to the individual survey data and entered as values of W made negligible improvement in the r^2-value. Equation 24 or 25 can therefore be used to estimate the fuel consumption in the urban traffic of Singapore for a passenger car with known engine capacity, E, and unladen weight, W.

Effects of Other Factors

Besides traveling speed, vehicle weight, and engine capacity, the survey data contain additional data allowing the effects of several other factors to be evaluated. These include effects of vehicle age, power steering, automatic transmission, and climatic conditions. Adding these factors to the model of Equation 23, however, would only improve the r^2-value from .54 to .56. For the practical purpose of fuel consumption estimation, the simple model of Equation 23 appears to be adequate. For the sake of completeness in presentation, the effects of the other factors mentioned are briefly discussed in this section.

Vehicle Age and Total Mileage Traveled

Vehicle age refers to the time in months since a vehicle was registered by the first owner. Alternatively, the total odometer mileage could be used as an indirect measure of vehicle age. These two variables were highly correlated. Either of these two variables, when included in the model of Equation 23, was found to be statistically significant at a 99 percent confidence level. The corresponding effect on fuel consump-

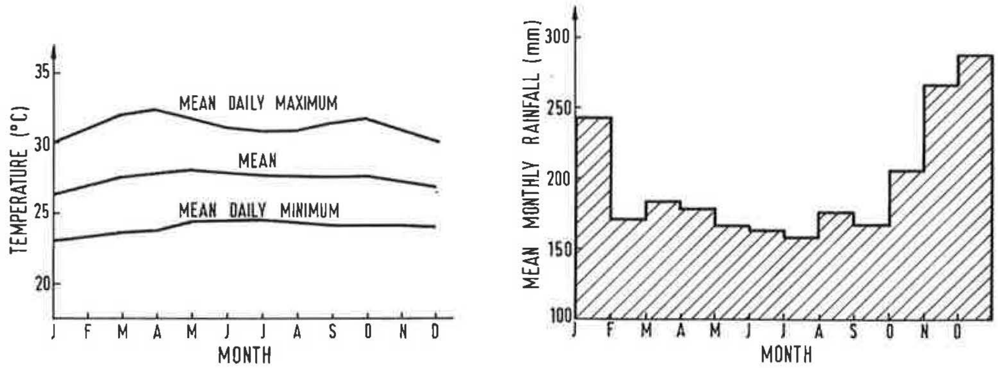

FIGURE 4 Characteristics of Singapore climate based on data from 1967–1986 (*16*).

tion rate of vehicle age was an increase of 1.53 mL/km/year, and that of total mileage traveled was 0.8 mL/km higher for every additional 1000 km—that is, the fuel consumption rate increased as the vehicle aged, or as the total mileage coverage of the vehicle became higher.

Power Steering

The effect of power steering can be included in Equation 23 by using an indicator variable that assumes a value of one if a vehicle has power steering capability and a value of zero if it has no such capability. The effect was found to be significant at 99.9 percent confidence level. The regression analysis showed that power steering on the average caused a vehicle to consume 11.3 mL/km more fuel than one without power steering.

Automatic Transmission

The effect of automatic transmission was studied by representing it with an indicator variable of the same nature as that of power steering. Regression analyses that included this variable in Equation 23 indicated that the effect of automatic transmission was also statistically significant, though at the lower confidence level of 95 percent. Vehicles with automatic transmission were found to consume about 0.78 mL/km more fuel than vehicles with manual transmission.

Climatic Effects

Studies conducted in temperate countries have shown that seasonal variations of ambient temperature affect fuel consumption rates of vehicles (*6,11,14*). Wet road surfaces on rainy days or in winter are also known to lead to higher fuel consumption because of increased rolling resistance (*6,15*). In the present study, average daily temperature and rainfall were computed for each survey cycle of each participant from daily climatic records provided by the Meteorological Service of Singapore. Correlation analyses of the survey data indicated that the temperature and rainfall variables were highly correlated. Either of the two variables would therefore be adequate to represent the climatic conditions of Singapore. The effect of either temperature or rainfall, when included in the model of Equation 23, was found to be insignificant at a confidence level of 95 percent. This is not surprising because a uniform climate prevails throughout the Singapore island during the whole year. Figure 4 shows the mean monthly variations of temperature and rainfall based on meteorological data collected over 20 years. This finding suggests that no seasonal correction factors are needed for the fuel consumption models derived in this study.

CONCLUSIONS

The analyses performed in this study indicate that the average-speed fuel consumption model (i.e., Equation 8 and Equation 9) applicable to the general automobile traffic in Singapore is comparable to those developed elsewhere for small cars. This finding is consistent with the predominantly small engine capacity of the car population of Singapore. The relationship established is region-specific. It is applicable to the traffic flow, road network layout, driver behavior, and road pavement and climatic conditions in Singapore. This aggregate fuel consumption model is useful for evaluating the energy impact of transportation policies or measures that affect automobile ownership and use.

The survey data also allow the effects of vehicle-specific parameters and climatic conditions to be evaluated. This was achieved by extending the basic average-speed model to include variables representing the parameters of interest. Statistical analyses showed that, in addition to the average traveling speed, factors that had significant impacts on fuel consumption included vehicle weight, engine capacity, vehicle age, and total accumulated odometer mileage, as well as whether the vehicle was equipped with power steering and automatic transmission. Seasonal variations of climatic conditions in Singapore were found to be too small to have any significant effect on fuel consumption rates.

The test results have shown that the survey approach adopted in this study can be used to calibrate the basic average-speed fuel consumption model. In addition, the survey data could be used effectively to identify the effects of various factors on fuel consumption. This approach is cost-effective, and it provides a comprehensive picture of vehicle fuel consumption in actual traffic conditions. A survey of this nature can be conducted easily by any highway agency without having to resort to special measuring equipment. However, this approach cannot be used to study the effects of some factors at the microscopic level, such as speed changes, driver behavior, road geometry, and pavement surface quality.

REFERENCES

1. *Energy Savings and Road Traffic Management*. Organization for Economic Cooperation and Development, Paris, France, 1985.
2. D. C. Biggs and R. Akcelik. Models for Estimation of Car Fuel Consumption in Urban Traffic. *ITE Journal*, Vol. 56, No. 7, 1986, pp. 29–32.
3. L. Evans. Urban Traffic, Fuel Economy and Emissions—Consistency of Various Measurements. *SAE Progress in Technology Series*, No. 18, Warrendale, Pa., 1979, pp. 65–75.
4. D. C. Biggs and R. Akcelik. Estimating Effect of Vehicle Characteristics on Fuel Consumption. *Journal of Transportation Engineering*, ASCE, Vol. 113, No. 1, 1987, pp. 101–106.
5. T. N. Lam. Estimating Effect of Vehicle Characteristics on Fuel Consumption. *Journal of Transportation Engineering*, ASCE, Vol. 111, No. 4, 1985, pp. 339–357.
6. *Automobile Fuel Consumption in Actual Traffic Conditions*. Organization for Economic Cooperation and Development, Paris, France, 1982.
7. D. C. Biggs and R. Akcelik. An Interpretation of the Parameters in the Simple Average Travel Speed Model of Fuel Consumption. *Australian Road Research*, Vol. 15, No. 1, 1985, pp. 46–49.
8. P. F. Everall. *The Effect of Road and Traffic Conditions on Fuel Consumption*. Report LR 226. U.K. Transport and Road Research Laboratory, Crowthorne, Berkshire, England, 1968.
9. *ROV Annual Report*. Singapore Registry of Vehicles, 1990.
10. A Method for Estimating Fuel Consumption and Vehicle Emissions on Urban Arterials and Networks. Report FHWA-TS-81-210. FHWA, U.S. Department of Transportation, April 1981.
11. M. F. Chang, L. Evans, R. Herman, and P. Wasielewski. Gasoline Consumption in Urban Traffic. In *Transportation Research Record 599*, TRB, National Research Council, Washington, D.C., 1976, pp. 25–30.
12. L. Evans and R. Herman. Automobile Fuel Economy on Fixed Driving Schedules. *Transportation Science*, Vol. 12, No. 2, 1978.
13. M. H. L. Waters and I. B. Laker. Research on Fuel Conservations for Cars. Report LR 921. U.K. Transport and Road Research Laboratory, Crowthorne, Berkshire, England, 1980.
14. B. H. Eccleston and R. W. Hurn. Ambient Temperature and Trip Length—Influence on Automobile Fuel Economy and Emissions. *SAE Progress in Technology Series*, No. 18, Warrendale, Pa., 1979, pp. 55–64.
15. D. C. Biggs. Comparison of Observed Heavy Vehicle Fuel Consumption in Canada with Estimates Produced by ARFCOM. *Australian Road Research*, Vol. 20, No. 3, 1990, pp. 11–17.
16. *Summary of Observations*. Singapore Meteorological Service, 1967–1986.

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Home Recharging and Household Electric Vehicle Market: A Near-Term Constraints Analysis

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Market-potential studies based on household travel behavior and consumer preferences show that electric vehicles may capture a share of the household motor vehicle market. However, most studies have ignored the implications of home refueling requirements on the market potential of electric vehicles. Using data from the 1985 American Housing Survey, we estimate the number of households that constitute the potential near-term private market for battery-powered electric vehicles. This estimate is based on housing characteristics and general vehicle usage patterns that are conducive, and probably necessary, to owning an electric vehicle. Foremost among these is the ability to recharge the vehicle at home. From these criteria we estimate that the potential private market for current-technology electric vehicles is approximately 28 million households—28 percent of the 1985 housing stock. Sensitivity analyses of household income and daily commute distances suggest that purchase price may have a greater impact on the marketability of electric vehicles than driving range.

Several studies have attempted to estimate the household market potential for electric vehicles (EVs). Various approaches have been used in these studies. A common methodology relies on models that project vehicle purchase behavior based on household characteristics and vehicle use patterns (*1,2*). Discrete choice modes base their estimates of the potential EV market on hedonic utility functions (i.e., one's willingness to pay for specific vehicle characteristics) (*3–7*). Another approach estimates the largest EV market potential based on travel behavior—how many household trips could be made in a limited-range EV (*8–11*). The common assumption among these approaches is that all households (at least all multicar households) belong to the universe of potential EV owners.

In this study we argue that not all households belong to the universe of prospective early-market EV purchasers. Instead, we estimate this total universe on the basis of four criteria not linked to hedonic preferences or economic trade-offs between familiar vehicle characteristics. The resulting subset of U.S. households includes only those that have the necessary infrastructure to recharge an EV and work commute demands that do not preclude using an EV. Analytical approaches that define EV markets in terms of travel behavior and consumer choices are appropriate and necessary for estimating the initial market for EVs. However, analyses of hedonic preferences and economic trade-offs should more appropriately be applied only to that subset of the total population that have household infrastructure and travel characteristics amenable to EV use.

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We use the following criteria to measure the largest possible initial market for battery-powered electric vehicles:

1. Potential EV purchasers must own their primary place of residence,
2. They must have a carport or garage at their primary residence,
3. In addition to the EV, they must have at least one vehicle capable of long-distance trips, and
4. There must be at least one household vehicle that is not used to commute more than 80 mi round-trip to work on a daily basis.

We use these criteria as proxies for determining how many households can recharge an EV at home and do not have commuting demands that would prevent them from using an EV. The resulting subset of American households is a good approximation of the largest near-term potential EV market and represents the target market for EVs. As mentioned, these households should serve as the starting point for market analyses based on economic considerations, overall travel demand, and consumer acceptance.

We confine this analysis to the near term because the most recent data set available that contains all the information required for our analysis is the 1985 American Housing Survey (AHS) (several important questions were omitted from subsequent AHSs). We define the "initial" or "near-term" potential EV market as the largest EV market that could be developed by the year 2000 given our proposed constraints. Hence, the EV performance and recharging capabilities that we assume are compatible with existing and likely near-term EV technology. Advances in EV technology will significantly affect our results only if those advances free EVs from home recharging requirements. In the long run, changes in the American housing stock could have a profound effect on our potential-market estimate.

This study is not a market development strategy. The target market as we define it excludes some households that may well be among the early buyers of EVs. For example, one can imagine scenarios in which renters or single-car households that are extremely motivated by air quality concerns or a desire to experiment with EV technology would buy an electric vehicle (*12*). However, we believe the number of such households will be small in comparison with the total number of potential EV-owning households identified in the following analysis. In any event, we test the effects of our assumptions through a sensitivity analysis.

Two previous constraints analyses of household EV markets use AHS data. Hamilton used 1974 data to estimate potential noncommercial applications of EVs (*13*). Kaiser and Graver used 1976 AHS data to estimate the number of likely EV-owning households as part of their analysis of electrical infrastructure (*14*). Our analysis differs from these studies in three ways. First, we update the data to 1985—the latest AHS data set that contains all the variables required for our analysis. Second, we use a different set of criteria to define the potential market for EVs. Finally, unlike our analysis, both previous studies assume that the EV ownership criteria are independent. We show the magnitude of error resulting from this assumption using the 1985 data.

We limit our market-potential analysis to privately owned passenger vehicles. The fleet market that has already been analyzed in detail will probably constitute a large part of the overall initial EV market (*15,16*). However, even if commercial fleets provide the first EV market, recently passed legislation gives reason to believe that widespread penetration of EVs in the private sector will not lag far behind.

LEGISLATIVE PROGRAMS DRIVING EV MARKETS

Several recent government actions encourage the use of electric vehicles because EVs provide an excellent prospect for mitigating urban air-quality problems. Electric vehicles essentially eliminate transportation-generated emissions of carbon monoxide (CO) and hydrocarbons (HC)—a primary ozone precursor (ozone is typically the main component of urban smog). Emission of nitrogen oxides (NO_x) will also be sharply reduced with the implementation of EVs compared with gasoline vehicles (*17,18*).

At the national level, EV implementation will most likely be facilitated by the recently passed amendments to the Clean Air Act that require certain vehicle fleets (e.g., delivery vans and taxis) in ozone and CO nonattainment areas be composed partially of clean-fueled vehicles. The law requires that 30 percent of all light-duty vehicles added to these fleets be clean-fuel vehicles starting in 1998. This requirement increases to 70 percent by 2000 (50 percent for heavy-duty trucks for every year after 1997) (*19*). The same legislation established a California clean-fueled vehicle pilot program. This program requires that 150,000 clean-fueled vehicles be sold in California each year for model years 1996 through 1998 and 300,000 a year thereafter.

California has adopted a program more specifically targeted at promoting EVs. In 1998 the state will require that 2 percent of the light-duty vehicles that each automobile manufacturer sells in California emit essentially no pollutants at all. This will rise to 10 percent of unit sales (about 200,000 vehicles a year) by 2003. Electric vehicles represent the only automobile technology currently under development that can meet these rigid zero-emission standards. Other states are expected to follow California's lead. Already, 12 northeastern states and the District of Columbia have committed to proposing regulations based on California's stringent automobile emission standards.

In Los Angeles an initiative has been passed that calls for the local sale of at least 10,000 electric and electric-gasoline hybrid vehicles by 1995. Three companies were awarded contracts to produce the 10,000 vehicles, which will consist of 1-ton vans, ¼-tone microvans, minivans, pickups, and four-passenger sedans.

The effectiveness of EVs at fulfilling the air quality mandates in government legislation depends on the total number of vehicle miles of travel that these vehicles can satisfy. This in turn depends on the marketability of EVs. The objective of our analysis is to determine the largest possible near-term EV household market on the basis of home refueling requirements and compatible commuting behavior.

STATUS OF TECHNOLOGY

The current capability and short-term prospects for improvements in EV technology and performance affect the possible size of the EV market. Technological variables such as recharging efficiency, battery performance, and total battery capacity along with the electrical capacity of a housing unit determine the necessary recharging requirements and thus the suitability of the structure for recharging an EV. In this section we briefly describe the status of EVs and state the EV technology assumptions used in our subsequent analysis.

We assume the first commercially available EVs will exhibit performance characteristics similar to existing prototypes. More specifically, they will have a driving range of 100 to 150 mi and an efficiency of 0.33 kWh/mi from the outlet. Without specifying a battery type, we assume that charging can be accomplished at nearly a constant rate (i.e., there is essentially no trickle charge), there are no gases emitted during charging, there is no significant "memory effect" from cyclic shallow discharges, and deep discharges are not detrimental to battery life. In general, lead acid batteries, the only battery type commercially available for EVs today, do not exhibit all the characteristics assumed in our analysis. However, several battery types—including advanced lead acid batteries—are being designed and developed specifically to overcome these problems (*20*).

Most prototype EVs are equipped with an onboard charger integrated into the vehicle electronics. We assume that near-term EVs will be equipped with onboard battery chargers, and thus can simply be plugged into a wall outlet.

We analyze battery-powered EVs because they are the focus of most ongoing EV research and are likely to be the first commercially available EVs. However, there are other types of EV. One alternative to the battery-powered EV is the hybrid EV (HEV). In one type of hybrid, a gasoline tank and a small internal-combustion engine are added to the electric drive to extend the range of the vehicle. However, regulations and incentives encouraging the use of EVs are motivated primarily by air quality concerns. If HEVs are to provide environmental benefits similar to those of dedicated EVs, they will have to run primarily on battery power—the gasoline or diesel engine would have to be used sparingly, perhaps only as an emergency backup. In this case home recharging is still an essential requirement for HEVs, and the market for hybrid vehicles will be similar to that of battery-powered EVs.

Framework for Analyzing Potential EV Market

To estimate the number of potential EV households, we determine which housing unit conditions are needed to accommodate an EV and identify household travel patterns that would preclude using an EV. Our conditions and requirements are based on insights gained from the literature. Much of what we wish to measure is not well understood (such as the amount of driving-range reserve that prospective EV buyers will demand) or lacks supporting data for a more robust analysis (the only measures of household travel in our data set are daily work commute distances and modes). In this section we lay out the logic for each of our conditions and test the sensitivity of the logic to different assumptions.

Our analysis is based on household and commuting data from the 1985 AHS. The AHS is conducted jointly by the U.S. Departments of Commerce and Housing and Urban Development. The AHS, formerly the Annual Housing Survey, is conducted every 2 years. It contains approximately 47,000 randomly sampled housing units enumerated in the 1980 census and approximately 6,500 units built since 1980 (*21*). The sampling unit is the structure, not the occupants.

Criteria Definition

We limit the potential market to those households that can recharge an EV at home, even though theoretically there are several means by which EVs can be recharged away from home. However, these methods are expensive, difficult to implement, and will not be developed until there are enough EVs to justify a large investment in the necessary recharging infrastructure. Furthermore, even if away-from-home recharging becomes available, it is likely that EV owners will do most of their recharging at home; certainly, enough so that they will not buy an EV if they cannot recharge at home. Battery-powered EVs will not become ubiquitous without home recharging.

Restricting EV Purchases to Homeowners

Analyses of the EV market seldom distinguish between homeowners and renters even though renters will probably be unable to recharge an EV at home. Relatively few rental units have the necessary facilities for recharging a personally owned EV, and it would be expensive to equip parking spaces with separately metered recharging outlets.

Cost estimates for installing multivehicle recharging facilities with a 240-V 50-amp electrical capacity range from $100 to $900 (1991 dollars) per stall, depending primarily on the number of recharging stalls, whether the recharging facility is installed at a new or existing housing structure, where the facility is located relative to the electrical source, and whether the charging stalls are opened or covered (*14,22,23*). Moreover, the landlord would have to agree to put in the recharging unit (outlet, wiring, etc.).

An investment in recharging facilities would be risky for landlords. They are unlikely to provide recharging facilities for EV owners unless installation and operation are safe and trouble-free, and only if they are certain of recouping their investment within a reasonable period of time. The only way to guarantee this would be to charge the EV-owning tenant enough per month to ensure complete payback within the current lease period because subsequent tenants may not have use for the recharging station. In most cases, the renter who owns an EV would probably be obligated to essentially buy the recharging station outright.

We expect that renters will not assume the installation cost of the recharging infrastructure unless they anticipate remaining at the residence long enough to justify the investment. Although this condition applies to homeowners too, the AHS data show that renters generally have a much shorter tenure at their residences than do homeowners.

Furthermore, many apartment complexes do not have the parking capacity for EV. Multifamily structures often have parking lot capacities designed for one parking space per unit; renters who own EVs will require a parking space for their second vehicle (as per Criterion 3) in addition to a designated space for EV recharging.

For these reasons we exclude all renters from the initial target market. However, it is possible that local utilities may assume the cost of recharging facilities on rented properties, in which case renters could play an important role in EV markets. We perform a sensitivity analysis to show the potential market gain when renters are included as prospective initial EV purchasers.

Need for Garage or Carport

We anticipate that EV owners will want a safe and secure covered place close to their houses to charge their vehicles. We assume this place is an existing garage or carport. A homeowner with no garage or carport could set up a recharging station, but this would involve additional costs. There would probably be a relatively long underground run from the service panel to the outlet, a recharging pole, weatherproofing for the outlet and pole, and some kind of security device (to prevent unauthorized plugging and unplugging). All these are extra-cost requirements compared with recharging in a garage. We assume that there would be relatively few people willing to set up an outdoor recharging station under these circumstances.

Furthermore, many residences without a garage or carport must park their vehicles on the street. EV owners who park on the street can not be guaranteed a vacant parking space near their outlet even if they could set up a curbside recharging station.

Some homeowners who do have garages or carports currently use them for purposes other than parking vehicles (workshops, converted bedrooms, storage, etc.). The AHS data specifically exclude garages that have been rendered permanently unusable for parking vehicles. However, AHS data do not identify garages that have not been permanently converted for other uses but that nevertheless are not being used for vehicle storage.

Owning a garage or carport does not guarantee the necessary electrical infrastructure to recharge an EV. Unfortunately, there are no data that enable us to determine how many households have an outlet in their garage or carport or how much electrical capacity is available within a specific dwelling unit. In the absence of these data, we assume that households meeting all our criteria either have the necessary electrical infrastructure to recharge an EV adequately for its intended use, or the occupants would be willing to install an electrical outlet in their garage or carport that is capable of sufficiently recharging an EV. We summarize the likely costs homeowners will incur if they do not already possess the outlet needed for EV recharging.

Cost estimates for installing a 240-V, 50-amp receptacle in a carport or garage range from approximately $100 to $675 (1991 dollars), depending primarily on how far the outlet is from the electrical source and whether it is installed in a new or existing structure (*14,22,23*; estimates from local electricians, unpublished data, 1991). A 240-V, 50-amp circuit would be capable of recharging an EV with our assumed performance characteristics at the rate of 28.5 mi of driving range per hour of charging (36.5 mi of driving range per hour of charging if charging is limited to 3 hr or less as per the "continuous load" restrictions stated in the National Electrical Code).

The lower-bound cost estimate represents the best installation scenario (not having to install an outlet at all would be the best overall scenario) for which there is sufficient service entrance panel capacity and a 240-V, 50-amp receptacle is installed close to the panel. The high cost estimate represents the opposite situation, in which the panel must be replaced and a 240-V, 50-amp receptacle is installed far from the panel. Generally, costs for replacing a circuit with one of higher amperage and voltage will typically be closer to the lower end of the cost range, and installing a receptacle in a new home during construction is much less expensive than retrofitting an existing house.

Although the cost of a recharging station in a rental unit could be as low as the cost in an owned unit, it is not likely. It is more likely that renters would incur costs near the high end of the range (as much as $900) to install recharging capability because most rental units are not equipped at all to handle vehicle recharging (this is further justification for excluding renters). On the other hand, many people who own their residence will probably face few or no additional costs to upgrade their electrical infrastructure because they already have adequate electrical capacity in their garage.

All single-family homes with an attached garage built since 1974 are required by the National Electrical Code to have at least one electrical receptacle in the garage that is not dedicated to a "permanent" appliance. Furthermore, on the basis of an informal survey of electricians nationwide, we believe it is likely that most garages and carports have at least a 110-V, 15-amp outlet that perhaps could be used for recharging an EV. Therefore, our assumption that electrical infrastructure requirements in owner-occupied homes will not alone prohibit the purchase of an EV seems reasonable.

Need at Least Two Vehicles

Because the first-generation mass-produced EVs will require several hours to recharge and will have a significantly shorter driving range than conventional gasoline vehicles, we assume the EV-owning household will keep at least one vehicle that is capable of long-distance trips. We posit that every household will want the option of making long trips without having to rent or borrow a vehicle (although we acknowledge that occasionally renting a vehicle may be the most economical solution). Therefore, we assume that the EV will replace an existing household vehicle or be added to the household's stock of vehicles, but that it will not be the only vehicle in the household.

Given this assumption, the number of potential EV-owning households at any given time (assuming they meet the other three criteria) is the number of multivehicle households that might add or replace a vehicle, plus the number of single-vehicle households that are in the market for a second vehicle. However, we do not have data on the number of single-vehicle households that are in the market for a second vehicle. Moreover, we do not know to what extent the introduction of EVs would affect the rate at which single-vehicle households become multivehicle households, or the rate at which multivehicle households would become one- or zero-vehicle households. Instead of attempting to estimate these effects, we use the number of current (1985) multivehicle households as a proxy for the number of potential EV-owning households. (We recognize that the percentage of multivehicle households—and hence the potential EV market—appears to be growing. Thus, our estimate may be considered a lower bound.)

It is also possible that some households would be willing to rely on an EV as its only vehicle. For example, a household that makes long trips only infrequently may be willing to rent or borrow a gasoline vehicle to fulfill their long-distance travel demands. We do not attempt to estimate the number of households that would be content with an just an EV, but we do relax the two-vehicle constraint in a sensitivity analysis.

Commute Distance Requirement

The regularity in timing and distance of work commute trips allows us to exclude households in which commute distances and characteristics of the household's current stock of vehicles would prohibit EV ownership regardless of other household travel. However, we do not require that the EV be used only for commuting purposes.

Households are excluded from the potential EV market only if the number of household members who commute more than 40 mi one way to work in a car, van, or truck, or report a variable work-trip distance (and hence might travel more than 40 mi), is equal to or greater than the number of vehicles available to the household.

This criterion implies that an EV could be used by any commuter with a round-trip distance to work of less than 80 mi. Given an EV with a range of 100 to 150 mi, an 80-mi commute would allow a buffer of 20 to 70 mi. The precise buffer required depends on what length of reserve the consumer will demand to ensure against running out of "fuel" and whether daily recharging is viewed as a significant inconvenience. Focus group participants have expressed concern about running out of "fuel" while driving an EV (*24,25*). However, some EV purchasers may be willing to accept more frequent recharging or a smaller buffer in exchange for a lower purchase price.

It is likely that the required reserve range of an EV will be a function of daily travel demands. Therefore, people with shorter commutes—say, less than 20 mi round trip—may be willing to opt for an EV with less than 50 mi of range if they bought it primarily as a commute vehicle. As the role of personal automobiles is reevaluated and expectations about multipurpose usage of vehicles change, EVs could become the most-used vehicles in many households.

To summarize, we define the potential near-term private EV market to be the subset of households that are occupied by the owner, have a garage or carport, have two or more vehicles, and do not have commute patterns that preclude the use of an EV. Although the criteria we propose are not mandatory for EV ownership, we posit that households not meeting all of the criteria will generally be unable to use an EV in the near term because of prohibitive travel demands or the inability to recharge at home. We believe households not meeting these criteria are very unlikely to buy an EV and therefore should be excluded from the universe of potential near-term EV-owning households for the purpose of further analyses.

RESULTS

According to our analysis, 27.9 percent of the 1985 U.S. housing stock (27.89 million out of 99.93 million U.S. households) has the potential to own an electric vehicle. These housing units are owner-occupied and have a carport or garage. The occupants have at least two vehicles and do not have prohibitive daily commute demands. This estimate is subject to a sampling error of ± 0.36 million households at the 95 percent confidence level. The geographical representation of potential EV households is generally the same as the total housing distribution.

If, like previous constraint analyses, we had assumed that all the criteria were independent (i.e., the same household could be erroneously excluded more than once from the potential EV market if it failed to meet more than one criterion), our potential near-term EV household market estimate would be 15 million households. This is 46 percent less than our actual estimate of 27.9 million households.

A comparison of AHS data from 1985 and 1987 suggests an upward trend in the percentage of American households that can accommodate an electric vehicle. Whereas the housing stock increased only 2 percent between 1985 and 1987, the percentage of households that are potential EV purchasers, according to our criteria except for household commute distances, rose approximately 6 percent in that same period. (We did not use the 1987 AHS data set for the complete analysis because neither it nor any AHS data set since 1985 contains information on household commute distances.)

Sensitivity Analysis

In this section we examine the effects of relaxing, in turn, each of our assumptions about which households can recharge and use an EV. The results are summarized in Table 1.

Case 1: Rental Units

If we relax the criterion that excludes renters, the total potential EV market increases by 15.85 percent, to 32.31 million households (Case 1 in Table 1). However, for reasons presented previously, it is highly improbable that a significant number of renters would be able to accommodate an EV until well after EVs are commonplace.

TABLE 1 Sensitivity Analysis Results

	Scenario	% of Households
0	Base case -- all criteria	27.89
1	Base case plus rental units that otherwise qualify	32.31
2	Base case plus rental units that otherwise qualify minus all households with at least one non-relative	30.67
3a	Base case minus condominiums and cooperatives	27.42
3b	Base case minus condominiums and cooperatives and households with at least one non-relative	26.60
4a	Base case plus households with only one vehicle that otherwise qualify	37.85
4b	Base case plus rental units that otherwise qualify with garage/carport constraint relaxed for all households	47.05

Case 2: Nonrelated Household Members

Furthermore, we are inclined to exclude renters for another reason: many renters share a residence with a nonrelative who may own at least one of the household vehicles. Prospective EV buyers living in households in which all vehicles are owned by unrelated individuals do not meet our two-vehicle criteria.

Because of the nature of the data, we were unable to determine who in the household owns each vehicle. However, examination of the rented housing unit composition shows that of the 4.42 million rented units that meet the other three criteria for EV ownership, 19.0 percent have at least one person who is not related to the head of the household. If we assume that the owner of each of these housing units has only one vehicle and the rest belong to unrelated individuals (an extreme case), our potential renters market estimate decreases from 4.42 million to 3.58 million. Consequently, the number of rented housing units that could accommodate an EV (as per the other three criteria) is more appropriately given as a range from 3.58 million to 4.42 million households.

The same type of analysis of owner-occupied households revealed that a maximum of only 0.80 million households (less than 3 percent of the estimated potential EV household market) could possibly be excluded given this nonrelative constraint. Case 2 in Table 1 shows the effect of excluding households with at least one nonrelative.

Case 3: Condominium and Cooperative Owners

A strong argument could also be made to exclude owner-occupied condominiums and cooperatives from the potential EV market for the same reasons that we exclude renters. Basically, condominiums and cooperatives are less likely than single-family detached homes to have reserved parking next to an outlet. Furthermore, condos and coops that do not have an accessible recharging receptacle may be less capable of installing one because condo and coop owners are often subject to stringent homeowner association guidelines on permissible changes to the building. If we exclude condos and coops, the market potential decreases 1.69 percent to 27.42 million households (Case 3a). If we further exclude households with nonrelated members (Case 3b), the potential market drops to 26.60 million households. This represents the lower-bound market potential estimate in this analysis.

Case 4: One-Vehicle Households

Finally, our assumption that one-vehicle households will not purchase an EV excludes 9.96 million households from the estimated market. Rejecting this hypothesis would increase the total EV household market potential 35.71 percent to 37.85 million housing units (Case 4a). We expect that there are relatively few single-vehicle households willing to replace their sole petroleum vehicle with an EV unless car rental agencies move to exploit this potential market by serving residential areas. If we relax the homeownership requirement in addition to including single-vehicle households, the potential market estimate increases to 47.05 million households (Case 4b). This represents the upper-bound market potential estimate in this analysis.

Varying EV Range Requirements

Another important consideration is the effect of household commute distances on EV market size. So far we have used commute distances to exclude households from our market estimate. Now we will show more specifically how commute distances affect our potential market estimate.

Although we chose an 80-mi round-trip commute distance as the cutoff point above which a current-technology EV is not likely to be useful for commuting, we recognize that actual EVs may have driving ranges that are shorter or longer. It is likely that manufacturers will offer EVs that have a range of more than 100 mi—already several prototype EVs have achieved ranges exceeding 100 mi. On the other hand, some individuals may wish to lower their vehicle purchase costs by purchasing an EV with a shorter range.

In Figure 1 we show how varying the shortest household commute distance (as per Criterion 4) affects the size of our potential market estimate. The x-axis represents the daily work-trip distance of the household member with the shortest commute. Each column is cumulative—that is, it includes all shorter commute distances. Note that the y-axis, the number of households, begins at 23 million rather than zero so that changes are visible on the graph.

The greatest change is between households for which the shortest round-trip commute is 10 mi or less and those 50 mi or less. There is virtually no change in the size of the potential market above a 50-mi round-trip commute, which suggests that current-technology EVs have two to three times the range needed to meet the minimum daily commute demands of most potential EV households. The potential EV market increases only 0.78 percent (216,334 households) when the shortest household commute is increased from at most 50 mi round trip to at most 100 mi round trip (this is essentially the same as increasing the effective range of an EV commute-vehicle from 50 to 100 mi).

Other studies that estimate EV market potential on the basis of vehicle performance characteristics and household travel demand indicate that EV range limitations may not be critical if a household's desired vehicle range is based on satisfying daily travel demands (*8,9,11*). Yet focus group studies and market estimates based on consumer choice models using revealed or hypothetical preferences suggest that the limited range of an EV is perceived as a significant problem (*3,7,24,25*). One can infer from these studies that, regardless of how frequently they actually travel long distances, people desire a vehicle versatile enough to make long trips.

Effect of Income on Market Size

We did not use income as a variable in identifying the initial target market, but it clearly will play an important role, and thus we examine the income distribution of our estimated potential market.

There is reason to believe that the life-cycle cost of EVs may eventually be less than those of conventional gasoline vehicles (*18,26–29*). However, EVs will probably still have higher purchase prices than gasoline vehicles. Batteries are currently very expensive and will continue to be so. Furthermore, there will be very few inexpensive used EVs available until EVs become widespread. The larger initial cost will most likely deter households with lower incomes. Not only do lower-income households have less money to spend on new vehicles, but there is evidence that suggests that these households use a much higher implicit discount rate when making automobile purchase decisions (*6,30*). Because of this, they may be less willing to wait for life-cycle cost savings to amortize the higher initial cost of an EV.

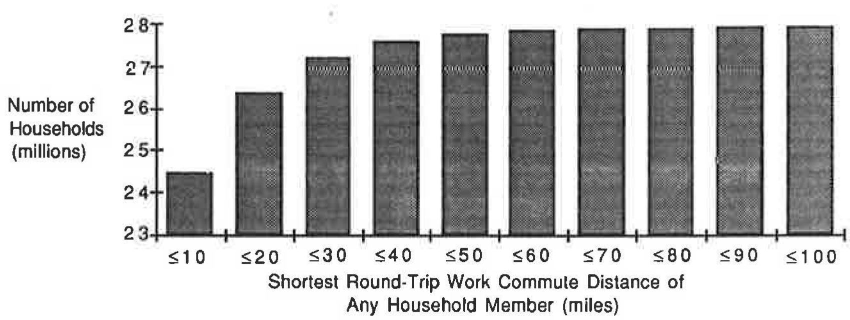

FIGURE 1 Potential EV market versus shortest household work commute.

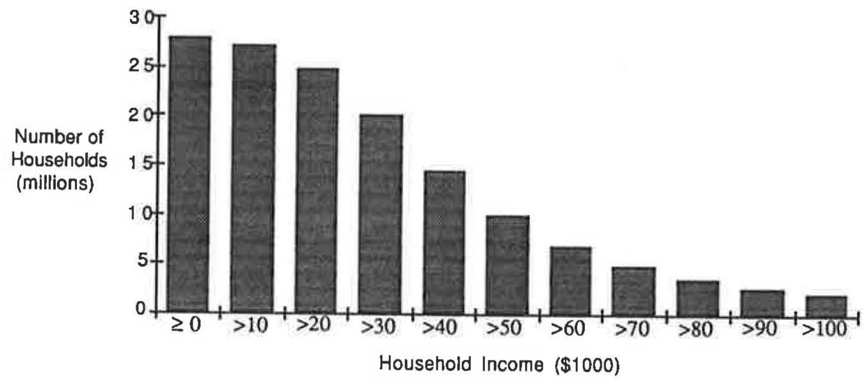

FIGURE 2 Potential EV market by household income.

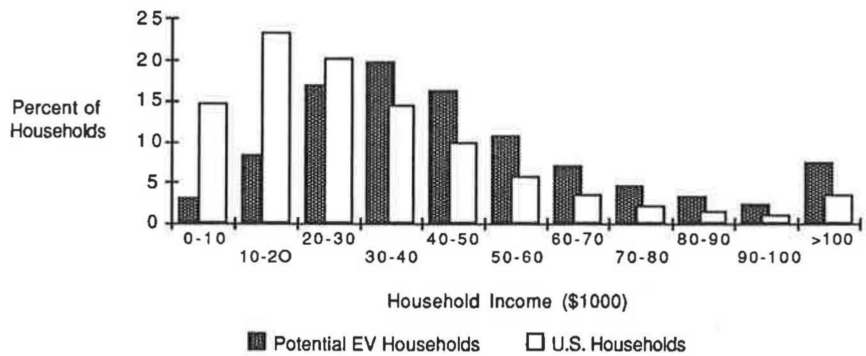

FIGURE 3 Household income distributions.

Without specifying a cutoff income level below which households would not buy an EV, we show in Figure 2 the distribution of household income levels for our estimated potential EV market. The income measure we use is the total pretax income (from all sources) for the reference person and all household members related to the reference person. (The reference person is the housing unit's owner, co-owner, or the owner's spouse.) Income is measured in 1985 dollars.

The number of potential EV-owning households drops markedly as the hypothetical income constraint is raised. The number of households that meet our four criteria and have household incomes greater than $20,000 is 24.70 million; the number meeting the criteria and earning more than $100,000 is only 2.10 million. These numbers are presented only as examples, not as conditions or predictions.

Income and Age of Potential EV Market Versus General Population

Our potential EV market is wealthier than the general population of American households (potential market included). The respective income distributions are illustrated in Figure 3. This comes as no surprise, given that our potential EV market is made up of households that own their residence and have more than one vehicle available.

Although we make no attempt to correlate income and EV purchase behavior, our results show how income may affect the potential EV market if it proves to be a critical factor in the decision to purchase. If EV purchases are highly dependent on income, our criteria will more effectively define the near-term potential EV market. Households not meeting our criteria will be even less likely to purchase an EV than previously argued because they generally have lower incomes.

Senior citizens and retired individuals are often cited as good candidates for EVs because they typically drive less than the general population. However, households composed entirely of retirement-age individuals (at least 65 years old) are underrepresented in our market estimate. Households in which all members are of retirement age make up 14.14 percent of all U.S. households but only 7.0 percent of the potential EV market. This does not mean that senior citizens will not be among the first to buy an EV; instead, it means that households consisting entirely of retirement-age individuals are less likely than the general population to meet all four of our proposed criteria.

CONCLUSIONS

Summary

Our assessment of the 1985 housing stock reveals that approximately 28 million households in the United States were owner-occupied, had a garage or carport, possessed two or more vehicles, and had daily commute demands that did not preclude EV utilization. We believe, as per the arguments presented in this paper, that given current and likely near-term EV technology, each of these households is a candidate for EV ownership. Projections of EV market penetration based on consumer choice theory, hedonic models, travel demand surveys, or other methods should use this subset of households as the starting point for their analysis. Market penetration studies that include households that face significant physical barriers to EV recharging will most likely provide erroneous results.

The findings of this paper agree with other analyses of potential EV markets that conclude that driving range, beyond a relatively short distance, is largely irrelevant to whether people could use an EV on a daily basis (*8,9,11*). However, our analysis says nothing about whether those who *could* use an EV, as per our criteria, *would* actually buy one.

Our analysis does suggest that EV costs could have profound consequences on the success of EVs. The market potential rapidly diminishes if lower-income households are excluded, which suggests that reducing EV cost could be more effective at enticing a larger market share than increasing EV driving range. EV research and development efforts should consider this possibility and focus attention on reducing the cost of EVs.

Implications

In this analysis, the near-term potential EV market is defined by four constraints that are used to indicate if a housing unit has the capability to recharge an EV and whether or not a household's commuting demands would preclude the use of an EV. However, these constraints could eventually be overcome by institutional, technological, and behavioral changes.

Institutional changes could overcome some of the market-limiting effects of our selected criteria. One problem discussed in this study is that investments in recharging stations are risky for landlords and renters. However, if utilities are allowed to include recharging station costs in rates charged for electricity (rate-basing), they may be more than willing to pay for recharging facilities in rental units. The cost of this part of the recharging infrastructure could be spread over a larger population that would enjoy the air quality benefits of EVs. Alternatively, utilities could earn emissions credits for subsidizing recharging stations.

Technological advances could also mitigate one or more of the constraints used in this study. The most significant increase in market potential may come from freeing EVs of the home recharging requirement. One possibility is to recharge EV batteries away from home: however, we do not believe that away-from-home recharging will be practicable until battery-powered EVs are ubiquitous. Another technological alternative is to replace or complement the battery with a fuel cell—an electrochemical device that converts stored methanol or hydrogen into electricity and that can be refueled in a few minutes. Should fuel cell technology progress and a hydrogen or methanol retail network for the fuel cell vehicle develop, then our home recharging constraints may not be applicable. However, any solution to the constraint of home recharging should be cognizant of the fact that those who *can* recharge at home may consider the convenience a significant advantage of battery-powered EVs (*12,25*).

Finally, changes in travel behavior may provide a means of overcoming the constraints presented in this analysis. The marketability of EVs is dependent not only on how close their performance characteristics are to conventional vehicles, but also on the willingness of households to adapt their travel behavior to EVs. Scientists and engineers continue to strive for greater driving ranges, but some households will be learning to adapt to shorter-range vehicles. Incentives that motivate changes in travel behavior may expand the potential EV market more effectively than technological advances.

REFERENCES

1. K. Train. The Potential Market for Non-Gasoline-Powered Automobiles. *Transportation Research*, Vol. A, No. 14A, 1980, pp. 405–414.
2. E. P. Marfisi, C. W. Upton, and C. E. Agnew. *The Impact of Electric Passenger Automobiles on Utility System Loads, 1985–2000*. EPRI EA-623. Electric Power Research Institute, Palo Alto, Calif., July 1978.
3. J. E. Calfee. Estimating the Demand for Electric Automobiles Using Fully Disaggregated Probabilistic Choice Analysis. *Transportation Research*, Vol. B, No. 19B, 1985, pp. 287–301.
4. W. Hamilton. *Electric Vehicle Design for Maximum Market Potential*. SAE Technical Paper 850226. SAE, Warrendale, Pa., 1985.
5. S. D. Beggs and N. S. Cardell. Choice of Smallest Car By Multi-Vehicle Households and the Demand for Electric Vehicles. *Transportation Research*, Vol. 14A, 1980, pp. 389–404.
6. S. D. Beggs, N. S. Cardell, and J. Hausman. Assessing the Potential Demand for Electric Cars. *Journal of Econometrics*, No. 17, 1981, pp. 1–19.
7. W. L. Garrison, J. E. Calfee, and H. W. Bruck. *Consumer Preferences for Electric Vehicles*. EPRI EM-4634. Electric Power Research Institute, Palo Alto, Calif., June 1986.
8. G. K. Despande. *Development of Driving Schedules for Advanced Vehicle Assessment*. SAE Technical Paper 840360. SAE, Warrendale, Pa., 1984.
9. S. J. Kiselewich and W. F. Hamilton, *Electrification of Household Travel by Electric and Hybrid Vehicles*. SAE Technical Paper 820452. SAE, Warrendale, Pa., 1982.
10. A. Horowitz and N. P. Hummon. Exploring Potential Electric Vehicle Utilization: A Computer Simulation. *Transportation Research*, Vol. 21A, 1987, pp. 17–26.
11. D. L. Greene. Estimating the Daily Vehicle Usage Distributions and the Implications for Limited-Range Vehicles. *Transportation Research*, Vol. 19B, 1985, pp. 347–358.
12. T. Turrentine and D. Sperling. The Development of the Alternative Fueled Vehicles Market: Its Impact on Consumer Decision Processes. In *Conference Proc., 6th International Conference on Travel Behaviour*, Quebec, Canada, 1991.
13. W. Hamilton, *Electric and Hybrid Vehicles*. Technical Background Report, Flexible and Alternative Fuels Study. U.S. Department of Energy, May 1988.
14. R. Kaiser and C. Graver. *Analysis of the Infrastructure for Recharging Electric Vehicles*. SAE Technical Paper 800112. SAE, Warrendale, Pa., 1980.
15. M. Berg, M. Converse and D. Hill. *Electric Vehicles in Commercial Sector Applications: A Study of Market Potential and Vehicle Requirements*. Detroit Edison Project 1569-3. Institute for Social Research, University of Michigan, Ann Arbor, May 1984.
16. J. Brunner and D. Wood. Assessment of Potential U.S. Market of Electric Vans. *Proc., 9th IEVS*, Toronto, Ontario, Canada, Nov. 1988.
17. Q. Wang, M. A. DeLuchi, and D. Sperling. Emission Impacts of Electric Vehicles. *Journal of Air and Waste Management Association*, Vol. 40, Sept. 1990, pp. 1275–1284.
18. M. A. DeLuchi, Q. Wang, and D. Sperling. Electric Vehicles: Performance, Life-Cycle Costs, Emissions, and Recharging Re-

quirements. In *Transportation Research*, Vol. 23A, No. 3, 1989, pp. 255–278.
19. *Congressional Record*. Vol. 136, No. 149, pt. 4, Oct. 26, 1990.
20. W. H. DeLuca, K. R. Gillie, J. E. Kulaga, J. A. Smaga, A. F. Tummillo, and C. E. Webster. *Performance and Life Evaluation of Advanced Battery Technologies for Electric Vehicle Applications*. Chemical Technology Division, Argonne National Laboratory, Argonne, Ill., 1991.
21. U.S. Department of Commerce. *American Housing Survey 1985: National File*. Inter-University Consortium for Political and Social Research, Ann Arbor, Mich.
22. W. C. Harshbarger. *Installation Costs for Home Recharge of Electric Vehicles*. IM-2291. General Research Corp., Santa Barbara, Calif., Jan. 1980.
23. *Assessment of Costs and Benefits of Flexible and Alternative Fuel Use in the U.S. Transportation Sector*. Technical Report 4: Vehicle and Fuel Distribution Requirements. U.S. Department of Energy, Aug. 1990.
24. D. O. Nelson. Assessing the Consumer Market Potential for Electric Vehicles: A Focus Group Approach. Presented at the 61st Annual Meeting of the Transportation Research Board, Washington, D.C., Jan. 1982.
25. D. Sperling, T. Turrentine, and K. S. Kurani. *Market Potential of Electric and Natural Gas Vehicles*. Report UCD-ITS-RR91-17. California Institute for Transportation Studies, Dec. 1991.
26. W. W. Marr, W. J. Walsh, and J. F. Miller, *Analysis of Life-Cycle Costs for Electric Vans with Advanced Battery Systems*. SAE Technical Paper 890919. SAE, Warrendale, Pa., 1989.
27. W. Hamilton. *Electric Van Performance Projections*. Report RP2882-1. Electric Power Research Institute, Palo Alto, Calif., Oct. 1988.
28. S. K. Takagishi. Life-Cycle Cost Comparison. *Proc., 9th International Electric Vehicle Symposium*, Toronto, Ontario, Canada, 1988.
29. J. W. Brunner, W. Hamilton, and O. Bevilacqua. *Estimated Life-Cycle Costs for Electric and Conventional Vans*. Electric Vehicle Development Corp., Cupertino, Calif., 1987.
30. K. Train. Discount Rates in Consumers' Energy-Related Decisions: A Review of the Literature. *Energy*, Vol. 10, No. 2, 1985, pp. 1243–1253.

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Economic Evaluation of Compressed Natural Gas Fleet Conversion and Operation

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Increased public concern about energy efficiency and air quality has led to a number of state and federal initiatives that examine the use of alternative fuels for motor vehicles. Texas instituted an alternative fuels program for public fleet operations beginning in FY 1991–1992. A life-cycle benefit/cost model for evaluating the economic implications of fleet conversion and operation on compressed natural gas (CNG) is presented. The principal benefit in a CNG-fleet operation is the fuel cost savings resulting from the price difference between gasoline/diesel and natural gas. The costs are classified according to capital infrastructure costs, capital vehicle costs, and operating costs. The benefits and costs are driven by fleet-specific demand parameters, including number and type of vehicles, annual mileage, fuel consumption, and fueling procedures. Sample fleets similar to those of the Texas Department of Transportation are analyzed to identify critical benefit/cost elements in the model. The sample analysis confirms that fuel prices, fueling infrastructure, and vehicle conversion costs are the key factors in the life-cycle economic evaluation.

During the 1980s it became increasingly apparent that transportation professionals would have to respond to new environmental mandates. At the forefront of these mandates was the recognition that motor vehicle fuels are a great source of undesirable emissions (*1*). A number of states and the federal government took action to investigate alternatives to gasoline and diesel fuels. Inherent in these policy directives were not only air quality issues but also national security concerns about U.S. dependence on foreign oil. Consequently, there has been a growing volume of research and demonstration projects on the use of alternative fuels.

Texas, a state rich in natural gas, adopted alternative fuels legislation (*2*) requiring all school districts with more than 50 buses, state agencies with more than 15 vehicles (excluding emergency vehicles), and metropolitan transit authorities to buy new vehicles that operate on natural gas, propane, methanol, or electricity. Affected agencies can receive a waiver of this requirement if they can demonstrate that (a) operation of an alternatively fueled fleet is more expensive than operation of a gasoline/diesel fleet or (b) alternative fuels are not available in sufficient supply. This paper analyzes the first area for natural gas. As the analysis demonstrates, it is difficult to show cost-effectiveness for compressed natural gas (CNG) as an alternative fuel when excluding externalities. This is a serious limitation to an otherwise progressive legislative action.

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On the basis of research at the Center for Transportation Research for the Texas Department of Transportation (TxDOT), the authors have developed a model for analyzing the cost-effectiveness of CNG as an alternative fuel for fleet operations. Basically, the model examines the benefits and costs of a CNG-fueled operation over the life cycle of a CNG fast-fill station. It is important to note that in this paper the model is used only for fleet analysis, not general public policy analysis.

CONCEPTUAL COSTS AND BENEFITS

As already noted, several positive social impacts result from the use of alternative fuels for motor vehicles. Although the focus of the benefit/cost analysis is on fleets, it is still important to consider the larger social impacts even if they are not dealt with in financial terms for the fleet analysis. In the long run, all benefits and costs must be considered in evaluating alternative fuel policies and their consistency with broader societal issues. However, it is important to determine what agencies or segments of society incur particular costs or benefits, as an input to public policy and budgetary allocation.

Societal benefits from natural gas may accrue in the following areas: urban air pollution, global warming, national energy security, regional economic stimulus, fuel toxicity, land and water pollution, vehicle safety, and transitions to future vehicular fuels, such as hydrogen. These benefits are difficult to quantify and incorporate into a fleet-level benefit/cost analysis. Rather than attempt to place a monetary value on these benefits, one can determine the minimum value that the broader social benefits must assume in order to overcome costs. This value could be used as a basis for developing a tax or fee to accommodate externalities that typically are not included in economic analysis.

In evaluating the economic feasibility or implications of operating a fleet of vehicles on natural gas, a life-cycle benefit/cost analysis is necessary. The main focus of this analysis is from the fleet operator's viewpoint, in particular on cost-effectiveness. Therefore, the narrower monetary benefits and costs (listed in Figure 1) to the fleet are analyzed. The benefits associated with other important policy goals are not included.

Benefits

Monetary fleet benefits are derived from the fuel price difference between natural gas and gasoline (or diesel) and from

BENEFITS
- A. Fuel cost savings
- B. Maintenance cost savings

COSTS
- A. Capital infrastructure
  1. Compressor
  2. Storage
  3. Dispenser
  4. Dryer
  5. Setup
  6. Land
- B. Capital vehicle
  1. If converted
     - a. Conversion kit equipment
     - b. Storage tank(s)
     - c. Labor
  2. If OEM
     - a. Cost differential
- C. Operating
  1. Station maintenance
  2. Power to drive compressor
  3. Cylinder recertification
  4. Driver and mechanic training
  5. Labor losses from fueling
  6. Texas state natural gas vehicle fuel tax

FIGURE 1 Summary of principal monetary fleet costs and benefits.

potential maintenance savings. The former is the primary source of monetary benefits, since natural gas is currently cheaper on an energy-equivalent basis. Adjusting for possible differences in fuel efficiencies between natural gas and gasoline or diesel vehicles, savings are accrued on the basis of the differential in price between the fuels. Maintenance savings (increased oil and spark-plug life are two possibilities) is the other potential monetary benefit. Documented proof of maintenance savings or of its magnitude is currently lacking, though anecdotal and theoretical evidence suggest the possibility of some savings. It is assumed that the fleet already has gasoline or diesel fueling capabilities on-site. These facilities will be used less while dual-fuel converted vehicles are used and may be eliminated if dedicated original equipment manufacturer (OEM) vehicles are fully phased in, but no benefit is given in this analysis for reduced operating and maintenance costs or for the possible elimination of those facilities.

Costs

Monetary fleet costs can be categorized into capital infrastructure costs, capital vehicle costs, and operating costs.

Capital Infrastructure Costs

Capital infrastructure costs represent the initial investment for an on-site natural gas fueling station and future additions for increased capacity. The station design could be slow-fill, nurse truck, fast-fill, or a combination. If fast-fill, the station design will vary according to the particular fueling scenario for a given fleet (for instance, whether all vehicles fill daily, in one session, or several sessions, etc.). The station itself has six cost components: compressor, storage, dispenser, dryer, setup, and land. Setup costs include all miscellaneous costs, such as those for priority and sequencer panels, piping, installation labor, and managerial soft costs.

Capital Vehicle Costs

Capital vehicle costs are those above what would be spent on a comparable gasoline or diesel vehicle. If the vehicle is converted from a gasoline or diesel vehicle, these differential costs are divided into three categories: conversion kit equipment, storage tank, and labor. The conversion kit costs include those for all "under the hood" parts such as air and fuel mixers, regulators, and piping. Storage tank costs include the cost of on-board tanks and mounting equipment. Labor costs are incurred in performing the conversion. If the vehicle is replaced with an OEM natural gas vehicle, then the capital vehicle cost is the price difference between the comparable OEM natural gas and gasoline (or diesel) vehicle.

Operating Costs

Operating costs include station maintenance, which is performed mainly on the compressor; power to drive the compressor; costs to recertify cylinders to conform with U.S. Department of Transportation (DOT) regulations; additional training for drivers and mechanics; labor losses from fueling; and the Texas state natural gas vehicle fuel tax.

Labor losses are incurred with fast-fill fueling, because of longer and more frequent fills (a result of current technology). If slow-fill is used, labor time savings may result, because time is required only to connect and disconnect the fueling probe, which is minimal compared with the downtime resulting from filling and switching (driving the vehicle up to and away from the fueling probe and getting in and out of the vehicle). Moreover, the fueling occurs during idle periods, so no person hours are lost due to waiting.

Texas law requires some state fleets to pay a fuel tax on vehicular use of natural gas. TxDOT vehicles are not exempt from this tax, and they must also pay state gasoline and diesel taxes. Currently, TxDOT vehicles are exempt from federal gasoline and diesel taxes, and there is no federal tax on natural gas use for vehicles.

Fleet conversions also generate some nonmonetary costs and benefits. Because of the difficulty in quantifying them, they are not included in the main economic analysis. Possible benefits include safer vehicles and improved public relations from capitalizing on the clean air benefits of natural gas. Possible costs include the risk involved in investing in a new technology (although there are more than 700,000 natural gas vehicles operating worldwide, there are only about 30,000 in the United States) and the negative impact from perceived safety problems.

The costs and benefits discussed represent the significant factor for evaluating the economic feasibility of a CNG-fueled fleet. Additional work is needed in valuing broader social impacts. These issues, although critical from a policy perspective, are often excluded in more limited applications.

FRAMEWORK DEVELOPMENT

This section presents an overview of the cost-effectiveness analysis framework and discusses the underlying assumptions and required input data. The analysis applies at the fleet level. A fleet is composed of different types of vehicles, each with a given set of attributes reflecting performance characteristics and use, both of which influence fuel consumption. Most of the cost and benefit items are incurred at the individual vehicle level, independent of other fleet characteristics. The major exceptions are infrastructure capital costs, for which some fixed costs are incurred regardless of actual fleet size.

The detailed expressions for each cost time are not presented here. These are mostly straightforward, and they would be too tedious and space-consuming; they have been implemented in spreadsheet format and documented elsewhere (*3*). Instead, this section focuses on the principal conceptual relations and assumptions, as well as on the input data required and the manner in which the various data items affect the calculations. Of particular interest is the approach devised in this study to estimate the fueling infrastructure requirements of the fleet under consideration; these requirements are translated into approximate sizes for the various station components on the basis of fundamental engineering principles (*4*).

The discussion in this section follows the order in which the principal benefit and cost elements are presented in the previous section. The principal input data requirements and assumptions are then discussed.

Benefit and Cost Calculations

The monetary cost/benefit fleet analysis uses a net present value (NPV) approach whereby all future costs and benefits over the time horizon of interest are discounted to the present using a rate that reflects the opportunity cost of capital for the particular fleet operating agency. In addition, the cost (or saving) per vehicle per year is computed by dividing the annualized NPV by the fleet size, in order to compare cost-effectiveness for different fleet sizes and to assist in identifying the offsetting level of societal benefits.

As explained in the previous section, monetary benefits derive primarily from fuel cost savings under CNG operation relative to gasoline and diesel. At the fleet level, then, savings depend on fleet size and composition (in terms of the different vehicle categories described). For a given vehicle type, the annual fuel cost savings are given by

$$\text{savings} = [\eta_{GAS,U}p_{GAS} - \alpha_{CNG}\eta_{CNG,C}p_{CNG} - (1 - \alpha_{CNG})\,\eta_{GAS,C}p_{GAS}] \times \text{miles} \quad (1)$$

where

α_{CNG} = fraction of total annual miles driven on CNG, $0 \leq \alpha_{CNG} \leq 1$;
p_{CNG}, p_{GAS} = respective prices of CNG and gasoline (per gasoline gallon equivalent), for the year under consideration;
$\eta_{CNG,C}, \eta_{GAS,C}$ = respective CNG and gasoline fuel consumption characteristics (in gasoline gallon equivalents per mile) of the vehicle after conversion to dual-fuel operation;
$\eta_{GAS,U}$ = gasoline fuel consumption for the vehicle before conversion; and
miles = annual mileage of the vehicle.

The given expression is modified appropriately to consider conversions of diesel-fueled vehicles as well as OEM vehicles. It is applied to each year separately over the time horizon of interest, allowing increased reliance on CNG over time as users become more familiar with converted vehicles and as the reliability of the technology is established. This can be reflected by increasing the value of α_{CNG} over time, or simply by using a lower value for the first few years.

In developing fleet-level estimates, average values (for each vehicle type) are used for the vehicle use and consumption characteristics. Letting the subscript k denote a particular vehicle type, the total fuel cost savings are given by

$$\sum_k (\text{savings})_k N_k \quad (2)$$

where N_k is the number of fleet vehicles of type k.

The other source of cost savings is maintenance costs savings. As noted earlier, these may or may not materialize. No particular methodology has been developed here to estimate such savings, given the absence of factual evidence to support such calculations. Such savings can be input directly in the spreadsheet as a per-vehicle amount for each type, allowing the analyst to conduct related sensitivity studies.

Three major cost items were described in the previous section: fueling (capital) infrastructure costs, vehicle conversion (capital) costs, and operating costs. The most challenging to estimate are the fueling infrastructure costs, as the literature contains little guidance in this regard. A new cost estimation methodology was developed for this application.

This analysis assumes that fleets will provide their own fueling infrastructure. Even if this is not the case, and the fleet is assumed to fuel at a public CNG filling station, this framework can still be used. The CNG fuel prices would then be adjusted to reflect public station prices, and all capital infrastructure, station maintenance, and station power costs would be removed, because they are now incurred by the public station and passed on to the fleet in the fuel price. The fleet can provide its own fueling infrastructure in several ways: (a) slow-fill from pipeline-supplied gas, (b) nurse truck–supplied gas/slow-fill, (c) nurse truck/fast-fill, (d) fast-fill from pipeline-supplied gas, and (e) combination slow-fill and fast-fill. Lower costs to the fleet may be possible with the slow-fill option, though one would have to change the fueling operation for the fleet. Such a change may not always be detrimental, as pointed out in the earlier discussion of possible gains in person-hour productivity associated with slow-fill. Though this analysis can be performed for any of the natural gas fueling options, the rest of this paper deals with the option that most closely replicates the service a fleet now receives with its own on-site gasoline or diesel stations, namely, continuous fast-fill with pipeline-supplied natural gas.

The most cost-effective fast-fill (with pipeline gas) fueling station design requires compression of natural gas into cascade storage [usually in three banks at about 3,600 psi gauge (psig)]. Vehicles are filled (nominally to 3,000 psig) from the storage in cascade fashion to get the maximum amount of gas out of storage while still retaining sufficient flow rates to fill vehicles in times comparable to those for gasoline and diesel. The size of the compressor and the size of the storage are chosen so that the storage is depleted when the last vehicle fuels. With depleted storage, another vehicle could still fuel, but it would take longer than the required maximum time allowed for fueling. It has often been suggested that minimizing the compressor size and maximizing the amount of storage will always be most cost-effective. Although we have not seen sufficient proof of this claim to generalize for all fleets, we assume it here for three reasons: (a) if the assumption is incorrect, costs are not significantly higher; (b) minimizing the compressor size minimizes the peak power required, which benefits electrical-rate-setting purposes; and (c) the assumption offers computation convenience.

This analysis features a new cost estimation approach that relies on a fueling station design methodology based on underlying engineering relationships (*4*). The compressor and storage sizes directly affect their costs according to cost-size relationships empirically calibrated using data reported in the literature and received from manufacturers and vendors (*5*; unpublished data, Christy Park, Inc.; Cherco Compressors, Inc.; Tri-Fuels, Inc., 1991). Following are some of the more important assumptions affecting these sizes and, therefore, costs.

The compressor cost-size relationship holds only for compressors designed to operate at input gas line (suction) pressures of 5 to 7 psig. Significant capital compressor and operating (power) costs savings are possible if the fleet has access to higher pipeline gas pressures. In fact, it has been reported that in Italy cost-effective natural gas filling stations require suction pressures of 150 psig (*6*). This analysis also assumes continuous filling of vehicles in one session per day. This maximizes the required storage. If it were assumed instead that vehicles fueled in two or three continuous sessions, with storage recharge time in between, then the storage size and cost would be less. The minimum storage cost occurs if vehicle fueling is uniformly distributed throughout the work day. The amount of fuel remaining in the vehicle's storage tank when fueling is performed must also be assumed. Another factor to consider is that these estimates are based on average daily fuel needs. In reality, a fleet may want to buy a compressor and storage that are slightly larger than estimated here (and therefore more expensive) in order to handle their worst-case days.

Finally, the calibrated relation for storage implies that storage is available in continuous increments, and this is not so. In reality, the fleet will need to purchase an amount of storage that is commercially available. This will probably result in a slightly higher cost than predicted here. The same is true for compressors, because individual companies may offer specific compressors at a price lower than predicted here on the basis of average patterns.

Some elements in this methodology tend to underpredict and others tend to overpredict station costs. On balance, the resulting estimate should be sufficiently close to actual costs for the purpose of this analysis. In fact, it produces predictions that are similar to other reported natural gas fueling station costs (*5,7,8*). It also provides the fleet operator with an approximate station design (i.e., size of compressor and storage) and indications of how conversion to natural gas (fueling aspects only) will affect fleet operation, through comparison of fueling session times, number of vehicles fueling daily, and labor fueling losses between natural gas and gasoline/diesel.

The fueling station design methodology used in this analysis breaks each fueling cycle into two distinct time periods: the time of the continuous fueling session ($T_{session}$) and the time for storage recharge ($T_{recharge}$) before the next session. The minimum compressor size (C_{min}) is then computed from

$$C_{min} = D_{session}/(T_{session} + T_{recharge}) \quad (3)$$

where $D_{session}$ is the fleet demand per session.

The maximum storage size (S_{max}) is computed from

$$S_{max} = D_{session}/[U_{storage} \times (1 + T_{session}/T_{recharge})] \quad (4)$$

where $U_{storage}$ is usable storage or the proportion of storage deliverable to vehicles from cascade operation.

Equation 4 is derived from Equation 3 and from the fact that the amount of natural gas used from storage during the session must be replaced by the compressor during recharge, as shown here:

$$S_{max} \times U_{storage} = C_{min} \times T_{recharge} \quad (5)$$

The underlying assumption in each of these equations is that the compressor is running continuously in order to minimize its size and maximize its productivity. One must have values for $U_{storage}$, $T_{recharge}$, $T_{session}$, and $D_{session}$ in order to calculate compressor and storage sizes.

$U_{storage}$ is a function of desired flow rate and the initial vehicle tank pressures (*4*). Therefore, values for $U_{storage}$ and flow rate per dispenser hose (F_{hose}) must be assumed. $T_{recharge}$ can be found by subtracting $T_{session}$ from the fleet fueling cycle time, which is normally 24 hr, since fleets typically operate on daily cycles.

$T_{session}$ is computed by assuming that queues of vehicles (with vehicles uniformly distributed by type) form at each available dispenser hose and that each vehicle type requires a certain total fill time ($T_{vehicle}$), which consists of a transition time between vehicles (T_{switch}) and an actual filling time (T_{fill}). The latter is simply calculated as $D_{vehicle}/F_{hose}$, where $D_{vehicle}$ is the natural gas demand per fill.

The average number of vehicles of each type fueling daily and $D_{session}$ can be derived, if one knows the on-board storage capacity and average annual miles traveled for each. The average number of vehicles of each type fueling daily and the number of dispenser hoses then gives the number and type of vehicles in each fueling queue.

Dispenser and dryer costs are input directly by the analyst, and the station setup cost is considered to be equivalent to a percentage of the combined cost of the compressor, storage, and dispenser (*7,8*).

The other major capital costs are vehicle conversion costs. No particular calculations are required here, because the various cost items are supplied directly by the analyst, as will be discussed later.

As reported in the previous section, six operating cost components are included in the analysis.

Station Maintenance Costs

Costs for station maintenance are incurred primarily by the compressor and are taken to be directly proportional to the fuel consumed. The unit cost per gasoline gallon equivalent is an input to the procedure.

Power Costs

Power costs, a significant operating cost component, are a function of the cost of electricity per kilowatthour and the energy required by the compressor. The cost per kilowatthour is an input for the procedure. The energy required by the compressor is a function of its motor horsepower (HP), its duty-cycle, and the number of hours of operation (obtained from the station design methodology). The compressor HP is computed from an equation empirically calibrated from published data and data obtained directly from manufacturers and vendors (*3,5*). Note that in years when tank recertification is required for a given vehicle, the consumption of natural gas (and therefore the compressor operating hours and fuel price savings) is reduced accordingly to account for the number of days that the vehicle cannot be operated on CNG, as current methods require that the tank be removed from the vehicle and taken off-site for hydrostatic testing.

Cylinder Recertification Costs

Costs for cylinder recertification are incurred periodically (every 3 years for composite cylinders and every 5 years for steel). They are computed on a per-cylinder basis and include costs for labor (to remove and replace the cylinder on the vehicle), for transportation (to the testing facility), and for the test itself. The total cost per cylinder is an input to the procedure. Recertification is required by DOT regulations.

Additional Training

Additional training, encompassing both driver and mechanic training, is directly entered by the analyst in the appropriate year it is incurred, if applicable.

Fueling Labor Lost Time

The natural gas fueling process is more time-consuming because of its slower fuel dispensing rate and lower on-board fuel capacity that requires these vehicles to fuel more frequently than gasoline and diesel vehicles (and thus incur the switching time between vehicles). The additional CNG fueling time relative to gasoline is multiplied by an hourly labor rate to obtain the corresponding labor costs. Any differences in queue waiting times between CNG and gasoline/diesel are ignored.

Texas State Natural Gas Vehicle Fuel Tax

This is a tax required for TxDOT and many other state fleets by Texas law. The tax is based on the annual mileage driven on natural gas and the weight of the vehicle.

As noted, these calculations have been implemented in spreadsheet format. The calculations require fleet data and several assumed values that must be supplied by the analyst. These are discussed next.

Input Data Requirements

The input data can be broken into five categories: vehicle data, fuel prices, fueling station data, fueling labor loss data, and miscellaneous factors.

Vehicle Data

Four vehicle types are considered in this framework: automobile, light truck (pickups and vans), heavy-duty gasoline, and heavy-duty diesel. Each type is characterized by different attributes that affect the costs and benefits of CNG conversion and operation. The data for a specific fleet consists of its composition (number of vehicles, year they are converted or an OEM natural gas vehicle replacement is purchased, and current gasoline fuel efficiency) and vehicle utilization (average annual miles traveled and percentage of this mileage using natural gas) by type. Factors to adjust fuel efficiency for comparable converted and OEM natural gas vehicles are also included here, as are the costs of conversion kit equipment, tanks, and labor for conversion and an OEM price differential. Other vehicle data include on-board gasoline storage capacity, maintenance cost differential, tank recertification cost, number of CNG tanks per vehicle, and salvage value differentials.

Fuel Prices

Fuel prices are used to calculate the major monetary fleet benefit. The pipeline price of natural gas to the fleet in dollars per thousand cubic feet (mcf) is used along with the natural gas-to-gasoline and natural gas-to-diesel energy conversion factors (in the miscellaneous factors section) to compute the price of natural gas per gasoline and diesel gallon equivalents. These prices are for an amount of natural gas with the energy equivalence of a gallon of gasoline or diesel. Also needed are the gasoline and diesel prices per gallon.

Because of the uncertainty involved in predicting natural gas, gasoline, and diesel prices over the next year—much less over the next 30 years—this paper does not present any elaborate future predictions. Because natural gas price trends have tracked gasoline price trend fairly closely over the past 20 years (see Figure 2), it is not unreasonable to assume that they will continue to do so in the future. This assumption might be incorrect if natural gas vehicles take over a significant share of the gasoline and/or diesel vehicle market. For flexibility and sensitivity analysis purposes, the analysis framework permits the consideration of any forecast profile and the comparison of different macroeconomic scenario forecasts, thereby allowing an assessment of the robustness of a particular fleet conversion decision.

Fueling Station Data

The principal parameters introduced in the station cost estimation procedure must be supplied by the analyst. In partic-

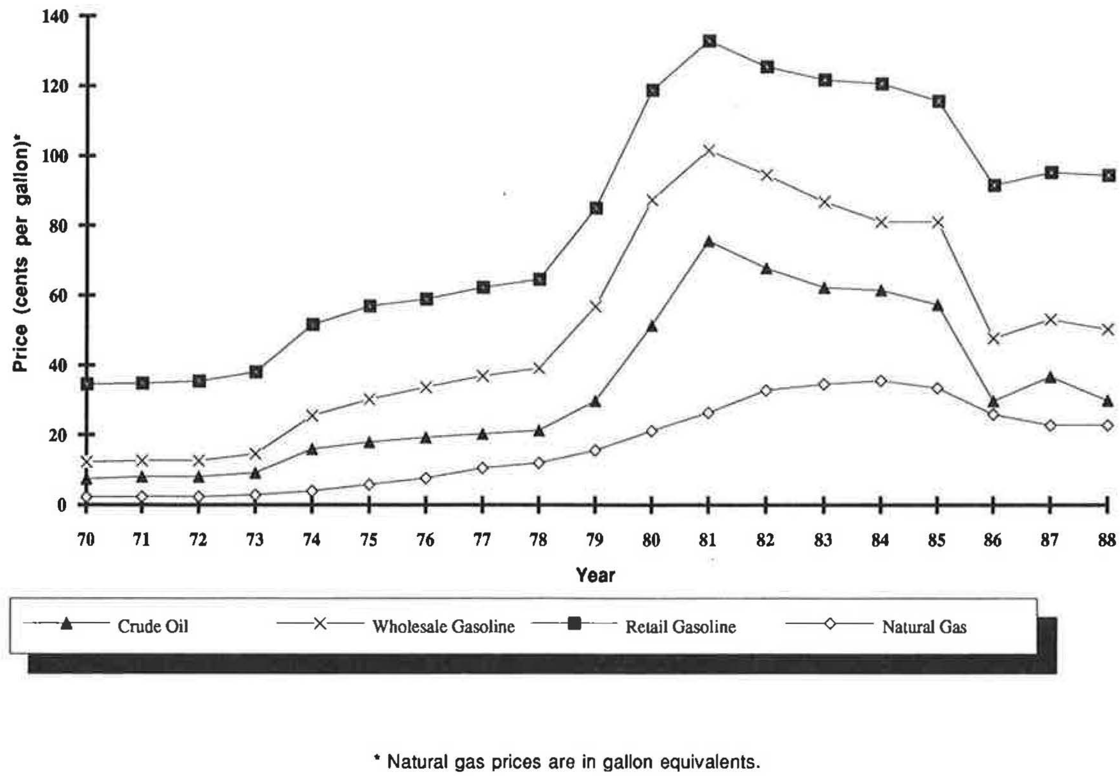

FIGURE 2 Fuel price comparison.

ular, values for the dispenser cost, dryer cost, switch time between vehicles, cycle time (i.e., session plus recharge time), number of dispenser hoses, station setup cost factor, usable storage, and average flow rate per dispenser hose over the whole session must be provided.

Fueling Labor Losses

The input data required for this calculation are very similar to those necessary to calculate the fueling session time for natural gas. In particular, values for gasoline and diesel flow rates, number of gasoline and diesel hoses, the gasoline/diesel switch time between vehicles, and the average hourly labor rate must be provided.

Miscellaneous Factors

Included here are the number of fleet work days per year, the cost of station maintenance per gasoline gallon equivalent, and the percentage of natural gas stored in a vehicle tank after the tank temperature stabilizes to around 70°F. During a fast-fill, increased tank temperatures effectively reduce the capacity of the tank (*9*). Compression factors allowing the calculation of the amount of natural gas in the vehicle when it is ready to be filled are also given, as are the volumes of natural gas in cubic feet at standard pressure and temperature (standard cubic feet, or scf) that have the energy equivalence of a gallon of gasoline or diesel.

The cost of electricity per kilowatthour is the price to the fleet under analysis. The national average is about $0.07/kWh (*13*). Also input is the number of days that tanks will be off a converted vehicle for DOT recertification. It is assumed that by the time OEM natural gas vehicles are widely available, tank recertification will be a part of ordinary state vehicle inspection and maintenance programs. Costs for this additional testing during inspection will be spread over all vehicle types, gasoline, diesel, natural gas, and others, so at that time there will be no incremental difference in cost for recertification. Finally, the discount rate or opportunity cost of capital, used to compute the present values of future monetary costs and benefits, is also an input.

SAMPLE FLEET APPLICATION AND SENSITIVITY ANALYSIS

This section illustrates the use of the previously described net present value and sensitivity analyses. First, a hypothetical fleet with characteristics favorable to cost-effective operation on CNG is analyzed, as an illustration of the type of fleets that may be cost-effective. Such favorable characteristics include a large number of vehicles to share in the fixed fueling infrastructure costs and a high average annual mileage, generating great fuel price savings per year. Next, fleets more representative of TxDOT are analyzed and compared with the "favorable" fleet.

Sample Fleet

To facilitate comparison, characteristics of the favorable fleet are based on TxDOT fleets, with the main differences being higher average annual mileage and larger-than-average fleet size. The characteristics of this fleet are shown in Table 1.

TABLE 1 Characteristics of Favorable Fleet

Vehicle Type	Number of Vehicles	Average Fuel Efficiency	Average Annual Mileage
Automobile	10	19.0 mpg	22,500
Light Truck	120	14.0 mpg	22,500
Heavy-Duty Gasoline	10	5.5 mpg	22,500

Heavy-duty diesel vehicles are not considered, because their conversion is much less cost-effective than gasoline vehicles. This is due to higher vehicle costs, both conversion and OEM; reductions in fuel efficiencies for CNG over diesel (for dedicated CNG vehicles); the greater energy density of diesel relative to gasoline; and the lower price of diesel to TxDOT fleets relative to the price of gasoline ($0.04/gal less).

Vehicles are assumed to be used for 90,000 mi (i.e., 4 years for this fleet). For the first 10 years, OEM gasoline vehicles are purchased and converted to dual-fuel CNG operation. In Year 11, OEM-dedicated natural gas vehicles are assumed available for all vehicle types.

Other important input variables are fuel prices, conversion costs, and OEM vehicle price differentials. Fuel prices are obviously highly uncertain, and conversion and OEM costs are somewhat negotiable and subject to change owing to technological advances and economies available with mass production and market competition, among other things. In this example, constant fuel prices (1991 dollars) are used over the entire 30-year analysis period. A gasoline price of $0.89/gal (including tax) is assumed, based on the prices paid by TxDOT in 1991. Conversion costs and OEM cost differentials are drawn from several sources (*5,7,14*; Natural Gas Resources, Inc., unpublished data, 1991) and shown in Figure 3, along with all other major input data assumptions.

Conversion costs	
Automobile	$1,950
Light Truck	$2,200
Heavy-Duty Gasoline	$3,300
OEM vehicle cost differential	$900
Gasoline fuel price (cents/gallon) (constant over entire analysis period)	89.0
Station maintenance cost (cents/gallon)[a]	4.5
Electricity cost (cents/kWh)	6.3
Vehicle life (miles)	90,000
CNG in a gallon of gasoline (scf)	122.7
Vehicle tank pressure before fill (psig)	100
Year OEM vehicles available	11
Cylinder recertification cycle (years)	3
Analysis period (years)	30
Discount rate	10%
Fuel efficiency decrease for conversions	5%
Fuel efficiency increase for OEM	15%
Usable cascade storage	40%
Average flow rate per hose (scfm)	300
Number of dispenser hoses	2
Vehicle maintenance cost savings	$0
Land cost for fueling station	$0
Additional training cost	$0
Labor rate per hour	$15

[a] Values of this factor ranging from 3 to 10 cents have been reported (6, 7, 9–12).

FIGURE 3 Input data assumptions.

Figure 4 shows a summary of the analysis for the favorable fleet with a natural gas price of $1.65/mcf. Under the base assumptions of the model, this price is required for operation of this fleet to be cost-effective (i.e., for the 30-year NPV of savings minus costs to be non-negative). Because actual natural gas prices are quite variable for different fleet locations, the break-even price of natural gas (i.e., the price required for cost-effectiveness) is found by performing a sensitivity analysis. One can then compare the break-even price with the price to any particular location or—as done in these analyses—compare the break-even price with plausible natural gas prices. Herein, $2.50/mcf is considered to be the lowest plausible pipeline-delivered natural gas cost to TxDOT fleets (*15,16*). Thus, conversion of this hypothetical fleet is not cost-effective under the base model assumptions.

It is interesting to note the relative magnitudes of the cost items. The 30-year NPV of fueling station infrastructure costs

SAVINGS	30 year NPV
Gasoline Price Diff.	$1,569,297
Automobiles	$75,664
Light Trucks	$1,232,247
Heavy Duty Trucks	$261,386
Diesel Price Diff.	$0
Maintenance	$0
Total Savings	**$1,569,297**
COSTS	
Infrastructure	
Land	$0
Station setup	($85,153)
Compressor	($65,829)
Storage Vessels	($240,395)
Dispenser	($24,857)
Dryer	($9,943)
Subtotal	**($426,176)**
Vehicle	
Conversion Kit	($89,889)
Tanks	($132,500)
Labor	($175,872)
OEM	($82,748)
Subtotal	**($481,009)**
Operating	
Station Maint.	($102,971)
Cylinder Recert.	($25,228)
Power	($127,855)
Labor - fuel time loss	($239,386)
NG Fuel Tax	($165,160)
Additional training	$0
Subtotal	**($660,599)**
Total Costs	**($1,567,784)**
Savings - Cost	**$1,512**

FIGURE 4 Favorable fleet analysis summary ($1.65/mcf natural gas price).

($426,176) and vehicle costs ($481,009) are of the highest magnitude, followed by labor-fuel time losses ($239,386), Texas state natural gas vehicle fuel tax ($165,160), power ($127,855), and station maintenance ($102,971). It should be noted that power and station maintenance costs accumulate on a per-gallon basis, and as such directly reduce the savings from the fuel price differential. There are no economies of scale for these costs, as more fuel is consumed through either annual mileage increases or changes in fuel economy. The sensitivity of the results to the assumptions used in computing the four highest cost items is examined next, along with the sensitivity to fleet size, average annual miles traveled per vehicle, and discount rate.

Sensitivity Analyses

For the sample fleet described, sensitivity to the following three relaxations of the base model assumptions are analyzed first:

- Relaxation 1—Eliminate Texas state natural gas vehicle fuel tax;
- Relaxation 2—Ignore labor-fuel time losses; and
- Relaxation 3—Reduce fueling station infrastructure costs by one-third.

Relaxation 1 is appropriate as a policy instrument for encouraging greater natural gas use. Relaxation 2 is important in order to highlight the value of both fueling station and on-board storage technology improvements. Finally, Relaxation 3 is used as an approximation of the maximum potential cost reductions associated with other fueling scenarios and technologies. The results are shown in Table 2 for the favorable fleet. Under relaxations 1 and 2 (jointly), this fleet's conversion becomes cost-effective at low—but plausible—natural gas prices.

Sensitivity to the price of natural gas can be examined by considering the base case above, where cost-effectiveness occurred at a price of $1.65/mcf ($0.20/gasoline-gal equivalent). As natural gas price increases to $7.25/mcf (equivalent to the gasoline price of $0.89/gal), fuel price savings approach zero (and become slightly negative owing to fuel efficiency losses with CNG), resulting in a very high cumulative NPV (−$1,567,784). Thus, cost-effectiveness is very sensitive to fuel price, since natural gas prices in the middle and at the high end of this range are quite plausible (*15,16*).

Vehicles in TxDOT fleets are driven in the range of 15,000 mi year. So, under the 90,000-mi vehicle life assumption used in this analysis, they are kept for 6 years. Because there are approximately 300 TxDOT locations at which vehicles fuel, and because TxDOT has about 6,000 gasoline vehicles statewide (mostly light trucks), the average fleet size is only about 20 vehicles, as opposed to 140 vehicles in the fleet analyzed earlier. Yet fleet variability is such that there are a few locations as large as 140 vehicles. Therefore, sensitivity analyses to average annual miles per vehicle and fleet size are performed. Fleet size is adjusted by changing the number of light trucks and leaving both the number of automobiles and the number of heavy-duty gasoline vehicles at 10. Three fleet sizes are analyzed. They contain 10, 60, and 120 light trucks, respectively, in addition to the 10 automobiles and 10 heavy-duty gasoline vehicles. The results for 15,000 mi/vehicle fleets are shown in Table 3.

The case with 120 light trucks differs from the one previously analyzed only in that the average annual mileage per vehicle is assumed to be 15,000 instead of 22,500 mi. The results are quite sensitive to this change. The break-even natural gas price is reduced by an amount ranging from $0.71 to $1.08/mcf. One must relax all three assumptions for the 15,000 mi/vehicle fleet to become cost-effective for a low—but plausible—natural gas price.

Results are also fairly sensitive to fleet size. The break-even natural gas price increases as the fleet size increases, mainly because of economies of scale in the fueling infrastructure costs. The break-even natural gas price is about $0.20 less for the 60 light-truck fleet than for the 120 light-truck fleet and drops by about another $0.40 for the 10 light-truck fleet. Since most of the TxDOT locations are best represented by the 10 light-truck fleet, even relaxation of all three assumptions does not quite yield a plausibly low break-even price for natural gas. Any other combination of relaxations yields implausibly low break-even prices. One can therefore conclude that it will not be cost-effective to convert most TxDOT locations to natural gas, unless more of the base assumptions of this analysis can be relaxed or natural gas is available at prices less than $2.50/mcf.

Sensitivity to the discount rate is reported in Table 4 for the 10 light-truck fleet with 15,000 average annual miles per

TABLE 2 Sensitivity Analysis, Favorable Fleet

Relaxations	Break-Even NG Price (per mcf)
None	$1.65
1	$2.24
1 & 2	$3.09
1, 2, & 3	$3.60

Note: NG = natural gas.

TABLE 3 Sensitivity Analysis, 15,000-mi Fleet

Relaxations	Break-Even NG Price (per mcf) for 10 LTs	60 LTs	120 LTs
None	-$0.03	$0.37	$0.57
1	$0.75	$1.22	$1.46
1 & 2	$1.55	$2.06	$2.31
1, 2, & 3	$2.49	$2.75	$2.89

Note: NG = natural gas, LT = light truck.

TABLE 4 Sensitivity to Discount Rate, Fleet with 10 Light Trucks and Annual Mileage of 15,000 mi

Discount Rate	Break-Even NG Price (per mcf)
10 %	-$0.03
8 %	$0.68
6 %	$1.37
4 %	$2.04
2 %	$2.69
0 %	$3.30

Note: NG = natural gas.

TABLE 5 Analysis for Immediate Availability of OEM Vehicles, 15,000-mi Fleet

Fleet Size	Break-Even NG Price (per mcf)
10 LTs	$1.85
60 LTs	$2.37
120 LTs	$2.61

Note: NG = natural gas.

vehicle, assuming no other relaxations of the base assumptions. The appropriate discount rate would have to be very low (0 or 2 percent) for the majority of TxDOT fleets to be cost-effective, and then only with fairly low natural gas prices.

The final sensitivity analysis reported here is for conversion costs. Assumed conversion costs (see Figure 3), which include kit, tank(s), and installation labor, are about 30 percent less than TxDOT is currently paying, as our analysis assumed a more mature natural gas vehicle market in Texas. Nevertheless, because of claims that conversions can and will be performed even cheaper, the limiting case of immediate availability of dedicated CNG OEM vehicles was analyzed for the three fleet sizes for 15,000 average annual miles per vehicle. This is the best case possible for vehicle costs, because OEMs cost less than conversions, tank recertification is not necessary, and greater benefits accrue from the increased fuel efficiencies of OEM-dedicated CNG vehicles. The analysis results are reported in Table 5. As expected, the break-even natural gas prices are much higher than those when conversions are used for the first 10 years. This further confirms that the introduction of OEM vehicles is very important, as is the reduction of conversion costs until that time.

Sensitivity to other factors (e.g., maintenance savings, vehicle fuel efficiencies, labor costs, electricity costs, power costs, station maintenance costs, and cylinder recertification costs) can also be investigated using this model.

CONCLUSIONS

The analysis has illustrated the primary significance of fuel price differential, conversion cost, and fueling infrastructure cost in the trade-offs underlying CNG fleet operation decisions. This analysis confirms that the actions of the natural gas industry and others to push for OEM vehicles, improved and lower-cost on-board storage technologies, and improved and lower-cost fueling infrastructure represent a good near-term strategy for achieving greater market penetration of natural gas vehicles.

The analysis has shown that the Texas state natural gas fuel tax is a significant cost item. Its removal should be investigated as a possible policy measure for improving the effectiveness of the Texas alternative fuels legislation.

The model presented in this paper is a decision support tool that allows one to deal with uncertain energy and technological futures through alternative scenarios and sensitivity analyses. It allows the proper accounting for the costs and benefits to fleets versus society at large, which has implications for the budget-setting process. For example, Texas has recently approved legislation that mandates some fleet conversion to natural gas unless it is not cost-effective for the fleet to do so. From the analysis herein, it appears that it will not be cost-effective for most TxDOT fleets to convert to natural gas operation with fuel prices, conversion costs, fueling infrastructure costs, and such comparable to current prices. Yet, if the societal benefits are considered to be great enough, the required additional funds may be provided to these fleets to achieve those objectives. Public policy in this regard could be guided by the use of this approach to compute the valuation of societal benefits that would make fleet conversion cost-effective.

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REFERENCES

1. R. E. Paaswell. Air Quality and the Transportation Community. *TR News*, TRB, National Research Council, Washington, D.C., May–June 1990.
2. Tex. S. 740, Aug. 28, 1989.
3. D. Taylor, M. Euritt, and H. Mahmassani. *Documentation for CNG Fleet Conversion Cost-Effectiveness Model*. Research Report 983-1. Center for Transportation Research, University of Texas, Austin, 1992.
4. D. Taylor, H. Mahmassani, and M. Euritt. *Fast-Fill CNG Fueling Stations*. Working Memorandum 3. Center for Transportation Research, University of Texas, Austin, April 1991.
5. *Analysis of the Economic and Environmental Effects of Compressed Natural Gas as a Vehicle Fuel: Volume II, Heavy Duty Vehicles*. Environmental Protection Agency, April 1990.
6. M. A. DeLuchi, R. A. Johnston, and D. Sperling. *Methanol vs. Natural Gas Vehicles: A Comparison of Resource Supply, Performance, Emissions, Fuel Storage, Safety, Costs, and Transitions*. SAE Paper 881656. SAE, Warrendale, Pa., 1988.
7. *Analysis of the Economic and Environmental Effects of Compressed Natural Gas as a Vehicle Fuel: Volume I, Passenger Cars and Light Trucks*. Environmental Protection Agency, April 1990.
8. *Assessment of Costs and Benefits of Flexible and Alternative Fuel Use in the U.S. Transportation Sector*. Technical Report 4: Vehicle and Fuel Distribution Requirements. U.S. Department of Energy, Aug. 1990.
9. *Natural Gas Vehicles 1990*. International Association for Natural Gas Vehicles. Jan. 1990.
10. *An Analysis of the Economic and Environmental Effects of Natural Gas as an Alternative Fuel*. EA 1989-10. American Gas Association, Dec. 1989.
11. T. L. Moran and V. B. Fiore, R&D Opportunities to Enhance the Utilization of Natural Gas as a Vehicle Fuel. Presented at Gaseous Fuels for Transport I Conference, Aug. 1986.
12. *Emissions, Fuel Economy, and Performance of Light-Duty CNG and Dual-Fuel Vehicles*. Technical Report EPA-AA-CTAB-88-05. Environmental Protection Agency, June 1988.
13. *Economic Analysis of Low-Pressure Natural Gas Vehicle Storage Technology*. Task 3 Topical Report GRI-90/0139. Gas Research Institute, April 1990.
14. Natural Gas Vehicles Research Contract Status Reports. In *Power Generation Subprogram Status Report 1988–1989*, Gas Research Institute.
15. *Natural Gas Prices for the Vehicle Market*. Issue Brief 89-19. American Gas Association, 1989.
16. *Monthly Energy Review*. Energy Information Administration, 1984–1990.

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PART 2

Environmental Analysis

Washington State Department of Transportation Wetland Monitoring Program

JAMES A. SCHAFER AND MARY C. OSSINGER

The Washington State Department of Transportation (WSDOT) is subject to federal, state, and internal directives that require the protection of wetlands. The department has responded to these requirements by examining alternatives to construction in wetlands and by following standard mitigation sequencing. For times when affecting wetland is unavoidable, WSDOT has created replacement wetlands. Compensatory mitigation sites are constructed to develop wetland characteristics and provide designated functions such as food chain support, ecosystem diversity, and wildlife habitat. Documenting the development of wetland characteristics and the performance of designated functions is the best way to assess the success of a mitigation project. A monitoring program was established to evaluate wetland mitigation sites and examine their progress toward achieving stated objectives. These objectives include the successful development of hydrologic characteristics of a wetland, hydric soils, and wetland vegetation. The monitoring program began with the development of the *Guide for Wetland Mitigation Project Monitoring*. This document details several monitoring tasks designed to assess important wetland characteristics. Currently, 10 wetlands are being monitored. Sites are monitored for 5 consecutive years, and some are now in their 4th monitoring year. The monitoring results have confirmed that WSDOT's created wetlands are developing wetland characteristics and are performing some wetland functions. Knowledge gained through monitoring has also helped to refine and improve site design and construction techniques.

The Washington State Department of Transportation (WSDOT) is subject to federal, state, and internal directives that require the protection of wetlands. These include the Presidential Executive Order 11990, *Protection of Wetlands*, issued on May 24, 1977; Section 404 of the Clean Water Act, which stresses wetland avoidance along with compensatory mitigation for unavoidable impacts; and the Washington State Governor's Executive Orders of December 1989 and April 1990, relating to the protection of wetlands. By asking all state agencies of Washington to use their "substantive authority" in enforcing a no-net-loss policy, the governor's orders have added even more impetus to the department's work in wetland avoidance and mitigation for nearly all projects.

WSDOT has tried to carry out the state and federal mandates during project development by examining alternatives to construction in wetlands. A standard five-step mitigation sequence followed on all projects with potential wetland involvement is to avoid, minimize, restore, compensate, and monitor results in cases in which affecting wetland is unavoidable. WSDOT has created, enhanced, and restored wetlands on many projects as a result.

Compensatory mitigation sites are intended to develop into functioning, sustainable wetland systems. Permits have been approved on the basis of detailed mitigation plans and the trust that WSDOT will create a fully functional replacement wetland. The purpose of this report is to describe a methodology used by WSDOT to attempt to establish and maintain an atmosphere of trust among the resource agencies, as well as to fulfill regulatory obligations.

PROCESS

When there is a project that has unavoidable wetland impacts, WSDOT proceeds with developing substitute wetland resources. Substitute resources can be the creation of a new wetland or the restoration or enhancement of an existing wetland. The emphasis in WSDOT's wetland mitigation project design is to develop a site that will have wetland hydrology and develop recognized and measurable wetland characteristics such as hydrophytic vegetation (plants adapted to saturated conditions) and hydric soils.

The primary objective for most projects is to provide food chain support, ecosystem diversity, wildlife habitat, and water quality benefits—things a little more difficult to measure, but nevertheless essential to the success of a site. These functions are expected to develop over a period of time that is agreed upon, in advance, with resource agencies in the development of a wetland mitigation plan. This document is prepared after the highway design is complete; it provides the details of the proposed mitigation plan. It is coordinated with and approved by agencies before the project permit approvals.

Maintaining positive expectations at this point in the permit approval process is very important to the timely issuance of permits that allow WSDOT to proceed with the highway project.

WSDOT has developed a systematic wetland monitoring technique that is task-oriented and that gives the department an opportunity to examine past performance along with the current status of the mitigation sites. A status report on past mitigation projects provides a feedback mechanism for reviewing agencies and is a valuable tool for developing a positive attitude on new projects.

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WETLAND MONITORING

The definition of wetland monitoring is as follows: the periodic evaluation of a wetland mitigation site to assess the progress toward achieving established objectives relative to the development of wetland characteristics and functions. WSDOT anticipates this to normally be a 5-year process.

Monitoring Manual

WSDOT's monitoring manual, entitled *Guide for Wetland Mitigation Project Monitoring* (*2*), was the result of a 2-year cooperative research project between WSDOT, FHWA, and the University of Washington. It was completed in October 1989 as an operational draft document.

In this manual, wetland monitoring is divided into five major areas that set up the framework for the monitoring task:

- Area A: mapping and hydrologic,
- Area B: water quality,
- Area C: soil and sediment,
- Area D: primary producer, and
- Area E: consumer monitoring.

For each section, an introduction and description provides background information and lists all of the equipment, supplies, and the procedural steps needed to obtain, record, and interpret data.

For example, Task A1 is the first task in Area A; Task A1 is wetland mapping. It is divided into objective, background, equipment and supplies, and data interpretation. This is typical of most of the tasks. The objective of the mapping task is to produce a map that can be used to locate sampling transects and plant communities and to assist with the overall evaluation of the mitigation plan. The background section states that the purpose is to quantify the areal extent of the created wetland and coverage by the different wetland classes (i.e., open water, emergent, scrub-shrub, etc.), to note changes that have occurred over time, and to use the data for comparison with the wetland mitigation plan.

Task A2 in this section is called transect establishment, and the methodology is given for determining the number and location of permanent transects at the wetland mitigation site. Transects are used as points of reference in several monitoring tasks. Both field and office procedures are described for laying out these reference points.

Task A3, photographic record, has the objective of producing a visual record of the development of the created wetland over time in a logical and meaningful fashion. The appropriate methodology is described.

Task A4 is water level gauging; its objective is to define the hydrologic fluctuation of the wetland over time by installing a staff gauge. Equipment and supplies, equipment installation, sampling program design, and data interpretation are included in the write-up. Related to this task is Task A5, which has to do with crest stage gauging, which amplifies the information that was obtained from the previous task.

The manual continues in this way through the other four sections—water quality monitoring tasks, soil and sediment, primary producer, and consumer monitoring tasks. Task D, primary producer, involves looking at the plant community. Several plant sampling techniques are described to evaluate the occurrence and influence of the plants on the mitigation site. Task E, consumer monitoring, deals with fish, wildlife, and invertebrate surveys, and a considerable amount of detailed information is given on types of sampling equipment, analytical procedures, and data interpretation. Samples of data collection sheets are provided for most procedures.

Not all tasks are done on every site; the monitoring protocol will be based on the wetland mitigation plan objectives and the determination of what is appropriate for the site.

Task Sequence

The appropriate sequence of monitoring tasks is described in the context of a typical field season (May through September).

Initial Site Visit and Installation of Transects—May

After construction and planting are completed at a new mitigation site, WSDOT visits the area during the first spring and permanent markers (steel fenceposts) are installed to establish sampling transects. The site is roughly mapped, showing the major features of topography, hydrology, and vegetation. A staff water gauge is installed in an appropriate area of the site.

Plant Community Mapping and Photography—Once Each Field Season

A rough map of plant communities is drawn, allowing biologists to note major shifts in vegetation patterns over several years. Color photos are taken from established points, including at least one panoramic series of the site.

Vegetation Sampling—July 15–August 31

Plants are sampled systematically along permanent transects. A typical sampling strategy uses quadrat sampling for the herbaceous layer and a line-intercept for trees and shrubs. Quadrats are sampled at an interval of every 3 or 6 m depending on baseline length.

Wildlife Surveys—June 1–July 15

Each site is visited three times to count wildlife (primarily birds) seen or heard from designated sampling points. Usually four stations are established per site with 5-min observation bouts at each station.

Water Quality Measurements—May–September

During every site visit throughout the field season, water temperature, pH, and dissolved oxygen are measured at designated points (e.g., inflow and outflow). This can be expanded considerably to meet special needs.

Invertebrate Sampling—June 15–July 15

Aquatic invertebrate samples are collected at several pre-established points at each site. Sample composition is analyzed later in the lab. A minimum of three stations per site are sampled, and a variety of equipment may be used. Results are expressed in grams per square meter for an estimate of productivity.

Soil Description—Once Each Field Season

The soil profile near the water's edge is characterized. Also, soil texture and organic content are analyzed during the 1st and 5th monitoring years.

Annual Monitoring Report—September–December

After all data are analyzed and interpreted, an annual monitoring report is written. The report discusses each site separately, comparing the current year's data with those from previous years (when available). The progress of each site is discussed. Special note is given to the development of wetland characteristics and to evidence of any problems. This report is sent to all pertinent resource agencies and satisfies wetland monitoring requirements that were part of the project approval.

DISCUSSION OF RESULTS

In the past 6 years, WSDOT has created about 25 acres of wetland to compensate for unavoidable wetland losses due to highway projects. These mitigation sites include freshwater emergent marshes, stream channel relocations, and estuarine wetlands. Monitoring of some of these sites began in 1988; as of 1991, 11 sites were being monitored. They are required to be monitored for at least 5 years.

The monitoring results demonstrate that the mitigation sites are developing into functioning wetlands. All monitored sites have met standards of wetland vegetative cover. Good invertebrate species productivity has been shown to occur at most sites. Wildlife that uses wetlands as primary habitat has been observed at all sites.

In addition to documenting WSDOT's overall progress with compensatory mitigation, wetland monitoring has also provided a means to evaluate the success of specific concepts and techniques used in the creation of mitigation sites and determine if and where improvements need to be made. Some lessons learned follow.

Need for Biological Expertise During Site Construction

Sometimes design plans are subject to interpretation by construction personnel. They may not understand the concepts and purposes behind the project design. To have sufficient wetland hydrology, which is the driving force, site contours and drainage control features must be correct. A wetland biologist should visit the site during construction to ensure that the end product will possess adequate wetland hydrology and appropriate substrate to support hydrophytic vegetation.

Substrate-Soils

Vegetation takes longer to establish on soils compacted by heavy machinery or previous fills. Soils lacking sufficient fine sediments and organic material are also undesirable. The best success comes when hydric soils are stockpiled from the affected wetland and later spread on the surface of newly constructed wetlands.

Vegetation Establishment

The success rate for planting is relatively low for many wetland species. For trees and shrubs, wattling (or brush layering) is an effective technique for rapidly establishing some shrub species (e.g., willows) along wetland edges or stream channels. Natural recolonization by adjacent vegetation communities is an effective method of vegetating new sites that are near existing wetlands if one can create good growing conditions. Also, stockpiled wetland soils can supply a ready seed source of wetland plants. Natural recolonization will be slow if the substrate is poor, and weed species may become established if a seed source is located nearby.

Potential for Disturbance

Urban and suburban sites are prone to disturbance by people and domestic animals. Since this will decrease the desired function of providing wildlife habitat, such sites may best be fenced to limit access.

CONCLUSION

The WSDOT wetland monitoring program has provided useful information about creating wetlands. Using that information, the department has been able to improve wetland design and construction techniques. The results of monitoring have also demonstrated to resource agencies that WSDOT is complying with their permit requirements.

In the past, wetland replacement was viewed simplistically in terms of the size, type, and location of a mitigation site. Increasing emphasis is being placed on the importance of a variety of wetland functions as attributes that should be evaluated independently. In the future, WSDOT expects to develop a larger number of specific goals to be met by compensatory mitigation sites. To measure success in meeting these goals, new monitoring techniques must be developed, and monitoring may have to be extended to cover a longer period of time at each site. With the foundation of the present program, these adaptations should be readily incorporated.

REFERENCES

1. Federal Interagency Committee for Wetland Delineation. *Federal Manual for Identifying and Delineating Jurisdictional Wetlands.* U.S. Army Corps of Engineers, Environmental Protection Agency, U.S. Fish and Wildlife Service, and USDA Soil Conservation Service. Cooperative Technical Publication, Washington, D.C., 1989.
2. R. R. Horner and K. J. Raedeke. *Guide for Wetland Mitigation Project Monitoring.* Operational Draft, Report WA-RD 195.1. Washington State Department of Transportation, Olympia, 1989.

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Method To Identify, Inventory, and Map Wetlands Using Aerial Photography and Geographic Information Systems

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The Washington State Department of Transportation (WSDOT) has a need to inventory wetlands along highway rights-of-way. Aerial photo interpretation was determined to provide a reasonable compromise between accuracy and cost, so several forms of aerial photography were tested. Two test areas were photographed with both true color and color infrared film in three scales. Photo interpreters classified wetlands and delineated their boundaries, and these interpretations were compared with data from field delineations performed by a wetland biologist. A method using aerial videography was also analyzed. On the basis of the test results and other factors, the preferred inventory method will use color infrared film at a 1:12,000 scale. WSDOT devised techniques to plot wetland boundaries on existing base maps and developed an Oracle data base that will be linked with the map files. When this is completed, it will be possible to print maps that depict wetland boundaries and classifications. A variety of modeling tasks and data analyses will also be possible. The inventory will cost approximately $658,000 for 7,030 mi of WSDOT right-of-way. The end product should improve early project planning, eliminate problems resulting from late discovery of wetlands within project boundaries, and reduce biologist field time.

In recent years, the state of Washington, along with other regional and national governments, has begun to realize the value of wetlands. Washington's wetlands benefit the state in many ways: they desynchronize peak runoff events, moderating surface flows and groundwater supplies; they detain floodwaters, helping to reduce flood damage; they trap sediments and pollutants, improving water quality in associated watersheds; and they provide vegetation diversity and crucial fish and wildlife habitat.

Recognizing the value of wetlands, state and federal legislatures have established regulations to preserve them. The state of Washington Governor's Executive Order EO 89-10 proclaimed a state goal of "no overall net loss" of wetlands. To meet this goal, development projects must avoid wetland impacts, or, if unavoidable, replace any lost wetlands. This effort has been hampered by a lack of information about the locations, types, and sizes of all the state's wetland resources.

RESEARCH APPROACH

Washington State is working toward better understanding and management of these resources. The Washington State Department of Transportation (WSDOT) has a particularly pressing need for wetland management because state and federal regulations demand that highway projects must avoid wetlands when possible, or mitigate when impacts are unavoidable. The WSDOT Protection of Wetlands Action Plan D31-12 (August 1990) directs that WSDOT shall complete a statewide inventory of wetlands within highway rights-of-way. This pioneering project will identify, classify, and map wetlands along all 7,030 mi of paved state highway.

Such an inventory will allow more precision and fewer false starts during initial planning stages. During project scoping and in early stages of project planning, this inventory will provide valuable information to aid in those processes. Field study time will be reduced. And long-term planning exercises could, for the first time, include reasonable estimates of impact on the state's wetland resources.

It is hoped that data from a WSDOT inventory can be linked with data from other state agencies to compile a detailed and comprehensive inventory of all of Washington's wetlands. Such information will provide an important management tool and will allow tracking of the state's resources over many years. Also, WSDOT will be better able to evaluate its own contributions to the goal of no net loss and consider remedial action if necessary.

This research is an attempt to determine the best method to inventory wetlands along all state transportation corridors. The study includes an overview of current methodology along with pilot projects to test the most promising methods. The final report, with appendexes, is available from WSDOT in Olympia (*1*).

State of the Art

Regional mapping of wetlands is being done on national and local scales. Interpretation of aerial photography is the most commonly used method because it can be employed over large areas with reasonable speed and accuracy. Brown, Howland, and others have found aerial photo interpretation to be a very effective method, provided that the interpreters are skilled in aerial photo interpretation and have a sound understanding of wetland ecology (*2,3*). Several types of film have been used, including normal black and white, black and white infrared, true color, and color infrared. Carter stated that color infrared was the best overall choice for wetland identification but added that true color was a good second choice (*4*). Austin et al. found that color infrared was best for surface vegetation, whereas true color was best for submerged vegetation (*5*). At

Washington State Department of Transportation, Environmental Branch, Transportation Building, Olympia, Wash. 98504.

present, color infrared is most frequently used for regional wetland inventories.

In large regions, small-scale (high-altitude) photos are generally used as the only feasible way to cover large areas in a reasonable amount of time. The best-known work has been done nationally as part of the U.S. Fish and Wildlife Service National Wetland Inventory (NWI). This survey has mostly used 1:80,000 (1 in. = 6,666 ft) and 1:58,000 (1 in. = 4,833 ft) color infrared photos. After the photos are interpreted by experienced persons, wetlands are plotted on 1:24,000 (1 in. = 2,000 ft) U.S. Geological Survey (USGS) topographic quadrangles, and these are made available to the public. The NWI maps work well to define large wetlands, but the scale is not well-suited to handling small areas such as those WSDOT deals with on highway rights-of-way. The NWI also purposely omitted agricultural land from the mapping exercise.

Several states have conducted, or are now working on, statewide wetland inventories. These are generally on a scale similar to the NWI work, although the resolution is often much lower. For example, the inventory in Michigan includes only wetlands that are at least 10 acres, whereas New York maps freshwater wetlands only larger than 12 acres. On the other end of the spectrum, some local governments have inventoried wetlands very precisely using a combination of large-scale (low-altitude) aerial photos and ground checks by wetland biologists. Unfortunately, these time-intensive and expensive surveys are not practicable for large areas.

To determine the experience and current thinking of other state departments of transportation (DOTs) in the subject of wetland inventories, wetland specialists from 15 state DOTs were contacted. States selected for the survey were those that seemed the most likely to be pursuing wetland inventories of their own. However, as of January 1991, none of the DOTs of surveyed states had endeavored to develop wetland inventories. Maine has come closer than any other state surveyed. The Maine DOT is working in cooperation with the Maine Department of Natural Resources to inventory the state's wetlands. The Maine DOT has funded some of the aerial photography, and the two agencies are sharing the photos and the results of their interpretations.

In several states, wetland regulation has prompted statewide wetland inventories by one or more state environmental agencies. The resulting information is generally made available to other state agencies, including the state DOT. However, this information is usually at a smaller scale than that required for highway planning, so the information may be useful only as a general guideline. When better data are unavailable, most DOT environmental sections use NWI maps for initial predictions about the presence of wetlands, although these maps are generally considered to be rough indicators. In later planning stages, field biologists are deployed to find wetlands and delineate the boundaries. The field work is very time-consuming and is probably part of the reason that most state DOT biologists profess that they are always struggling to keep up with the workload.

Research Goals

Given the usefulness of an inventory of wetlands along WSDOT rights-of-way, it remained for WSDOT to choose a method of performing such a task. There are no established procedures for inventorying wetlands along transportation corridors, although aerial photo interpretation can be considered as a starting point. The purpose of this study, then, was to determine the best method of using aerial photos to identify, classify, and map wetlands. Along with standard aerial photo techniques, a new method of computer-enhanced aerial videography was also evaluated.

Selection of the best method was based on a combination of factors, especially accuracy and cost. Once the best method was established, techniques were explored for entering data into a geographical information system (GIS). It is hoped that such a data base will make wetland data easily accessible in a variety of formats, including maps and summary reports. It will also allow WSDOT to share data with other state agencies to produce a more comprehensive picture of the state's wetland resources, thereby providing better management opportunities.

Research Approach

Aerial Photo Interpretation

Literature review and discussions with experts provided an understanding of the current state of the art in performing regional wetland inventories. This understanding led to a choice of techniques to be tested during the pilot study. Moderate-scale color aerial photography was deemed the most promising for WSDOT needs. Two small study areas were selected: 1.7 mi of SR-395, an agricultural area in the Colville River valley, and 3.7 mi along SR-18, a forested region with several stream crossings (Figure 1).

Aerial photos were taken in true color and color infrared using three scales: 1:24,000, 1:12,000 (1 in. = 1,000 ft), and 1:6,000 (1 in. = 500 ft). The photography was completed during the first week of April 1990. Two sets of prints were made so that two interpreters, working independently, could identify the wetlands. The two interpreters had very different background experience, which allowed some comparison of accuracy. Interpreter 1 had 4 years of photo interpretation experience, all of which focused on wetland delineation and included considerable field work in various wetland types. Interpreter 2 had more years of experience in photo interpretation, including 4 years interpreting hydrologic features, but no actual wetland delineation experience.

The interpreters used magnifying stereo lenses with stereo photo pairs to identify wetlands. Interpretation standards were set as follows:

1. NWI maps and USGS soil maps could be used for collateral data.
2. Wetlands within 250 ft of the road edge were to be classified according to the Cowardin et al. system and their boundaries drawn on mylar overlays (*6*).
3. All wetlands of at least 0.25 acres were to be included.

To determine the accuracy of the various interpretations, the study areas were assessed in detail in the field by a WSDOT wetland biologist. The field biologist produced a list of wetlands bordering the highway, documenting the Cowardin clas-

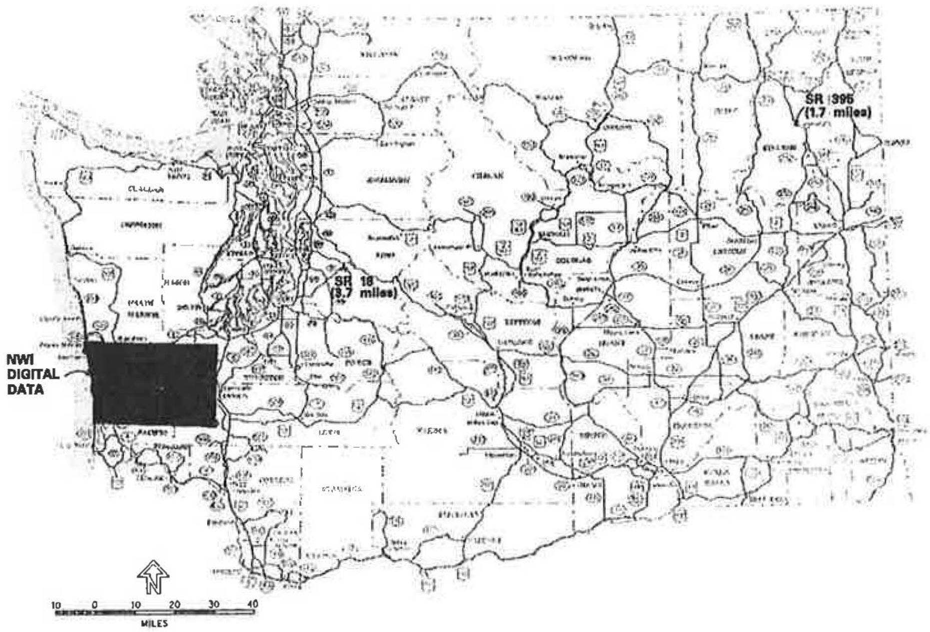

FIGURE 1 Location map of test areas.

sification of each wetland and its linear extent along the highway (*6*). These data are called the ground-truth in the discussion that follows.

A different WSDOT biologist compared the photo interpretations with the ground-truth. The biologist who did the comparisons checked some of the discrepancies in the field. In all cases that were checked, the ground-truth was found to be correct and the aerial photo delineation was in error. This supports the premise that the ground-truth accurately represents reality.

Comparison Methods

To evaluate the accuracy of the aerial photo interpretations, a WSDOT biologist compared the photo interpretations with the ground-truth. The first step was to record the extent and classification of wetlands on the aerial photo interpretations. Then the amount of overlap with ground-truth wetlands was determined at a rough level of comparison. There was no expectation of 100 percent agreement because geographic referencing was not accurate enough to translate wetland positions with complete precision. However, all photo interpretations received the same treatment, so different interpretations could be compared with one another.

The comparison between aerial and ground-truth wetlands determined the number of aerial-mapped wetlands that reasonably matched ground-truth wetlands. Wetlands that were incorrectly classified as to type and those more than 50 percent underestimated or overestimated were considered partially correct, as long as the general location had been correctly identified. Upland areas incorrectly classified as wetlands were counted as errors. Ground-truth wetlands that were missed in the photo interpretation were rated as worst-case errors.

Once these rough comparisons were completed, the results were used to produce a weighted score for each photo type. The scoring formula was based on the following criteria:

1. It is most important that existing wetlands be found by the aerial interpretation, so this criterion was most heavily weighted.
2. It is also important, though less so, that the interpretations should not show wetlands where there are none.
3. Errors in determining the vegetation type or size of a wetland will have a negative effect on the usefulness of the data, so such inaccuracies reduced the score, although these errors carried the least weight.

Computer-Enhanced Videography

As a demonstration project, WSDOT engaged EnviroScan, Inc., to use its aerial video imaging technique to delineate wetlands in the same two test areas that were used for the aerial photo study. Aerial videotapes were produced using four spectral filters: narrow-band chlorophyll a, narrow-band carotene, wide-band infrared, and wide-band ultraviolet. Approximately 1,000 ft along one side of the road was taped on each pass. In addition, one lower-altitude pass was flown to tape a strip 250 ft wide. A computer program was used to find and color-code regions on the videotape that had similar reflectance values (as represented by 256 shades of gray), supposedly areas of similar vegetation. The videos were interpreted by a wetland expert just as aerial photos would be.

The WSDOT research biologist intended to compare the EnviroScan wetland delineations with the ground-truth data in the same way that the aerial photo interpretations were compared. However, the EnviroScan report lacked the information required to do a detailed comparison. Therefore, this method was evaluated on the basis of a live demonstration of the product along with the written report.

Cartography and GIS

WSDOT Geographic Services installed new computer equipment and software shortly before beginning this research project. The new system consisted of an Intergraph 6240 Series Modular GIS Environment (MGE). Before any attempts to handle the data produced by the aerial photo interpreters, a sample data set of NWI maps in digitized form were obtained from the U.S. Fish and Wildlife Service (Figure 1). Techniques and software were developed to input the NWI data, in DLG-3 format on 8-mm tape, into the Intergraph system.

After the NWI data were successfully entered and test maps were printed, the cartographers moved on to processing the wetland information produced by the WSDOT interpreters. Aerial photo overlays for the true color photos in all three scales were used in the trial. The work of only one interpreter was used.

There were several steps in processing the wetland data. First, an existing 1:24,000 base map in digital form was scaled to fit the scale of the aerial photography, then printed. Second, wetland boundaries and labels were traced onto the fitted base map. Third, the annotated paper map was attached to a digitizing table, and standard techniques were used to fit the paper map mathematically to the digital base map and to trace the wetland boundaries. This produced a new digital layer for the base map.

Once the spatial data were entered, maps could be printed in a variety of scales, regardless of the original scale of the aerial photography. To deal with the wetland classification data, additional work is required to set up an Oracle data base to be linked with the base map files. The data base portion of the system will allow data analysis and modeling using the wetland data. For example, users will be able to find out how many acres of palustrine forested wetland are present along a specified milepost range on a given highway. As of this writing, WSDOT Management Information Services and Geographic Services are working together to develop this data base application.

FINDINGS

Aerial Photography

Accuracy

The calculated scores for each of six photo types are shown in Table 1. In theory, the higher the score, the more accurate the aerial photo interpretation. A perfect match between the ground-truth and an aerial interpretation would score 200. A negative score results from a large percentage of errors. Although these scores provide some method of comparison, they are the result of a highly subjective rating system. Therefore, it is not possible to use statistical methods to determine a percentage error or a significant difference between scores.

Two photo interpreters worked independently to delineate and classify wetlands on the aerial photos. Interpreter 1 was experienced in wetland delineation, but Interpreter 2 was not. Despite several years of experience with photo interpretation and acknowledged expertise, Interpreter 2's lack of actual wetland experience was probably responsible for the lower level of accuracy of those interpretations. All scores were considered when selecting the best method, but the scores of Interpreter 1 were probably more representative of photo interpretation work that would be done by trained and experienced wetland biologists.

Scores for both test sections are combined here, although the scores for the eastern Washington section were notably lower than those for the section west of the Cascades. The agricultural land in the eastern test area was particularly difficult to delineate, both in the field and by photo interpretation.

The aerial photos at 1:12,000 produced the most accurate results. At the larger scale of 1:6,000, there were more errors in which upland areas were incorrectly designated as wetlands. At the smaller scale of 1:24,000, accuracy was only slightly less than at 1:12,000, but the interpreters expressed a high level of uncertainty in defining very small wetlands on these photos. Because the goal of the inventory is to find wetlands as small as 0.25 acre, the 1:24,000 scale was rejected.

Although the 1:6,000 scale provides the most detail, both interpreters believed that the 1:12,000 photos had a definite advantage over the 1:6,000—the broader scope of the photos

TABLE 1 Wetland Interpretation Scores for Two Interpreters and Six Combinations of Scale and Film Type

	Scores		
Scale and Film Type	Interpreter One	Interpreter Two	Combined Data
1:6,000 True Color	72	10	34
1:6,000 Color Infrared	18	8	-26
1:12,000 True Color	103	38	54
1:12,000 Color Infrared	104	72	86
1:24,000 True Color	98	43	63
1:24,000 Color Infrared	61	15	35

made overall drainage patterns and ecological relationships more visible. The 1:12,000 scale seemed to provide the best compromise between a detail view (as in 1:6,000) and an overview (as in 1:24,000). The middle scale provided a reasonable level of resolution: quarter-acre wetlands were barely distinguishable without magnification, although magnification was used in the interpretation process.

There was no definite trend showing one film type, true color or color infrared, to be better than another. The higher-scoring film varied with the interpreter, the scale, and the test area. When asked about their preferences in working with true color versus color infrared, both interpreters marginally preferred the true color. These results are surprising, because historically, most wetland photo interpretation has used infrared photography. However, the test photos were taken in early April, somewhat early in the growing season for good plant definition on infrared film.

Estimated Costs

A comparison of costs is presented in Table 2. Aerial photo costs are for 7,030 mi of highway and are the same for color and color infrared. The number of photos required at each scale, and thus the cost for materials, doubles with each increase in scale. In our limited trial, the interpretation and mapping costs were roughly the same for each scale, despite the difference in the number of photos, because the lower resolution at smaller scales made some aspects of the work more difficult and time-consuming.

Computer-Enhanced Videography

For our pilot study, the consultants presented a demonstration of computer-enhanced images of the SR-395 study area, followed by a written report with a more detailed analysis of the wetlands in the area. The SR-18 study area was not analyzed because the aerial videotaping produced inadequate images.

After careful consideration, this method was rejected because of several shortcomings. The videotapes provided black and white images with relatively low resolution. The low-altitude trial showed more detail but also exhibited severe vertical distortion, making the landscape appear to undulate and obscuring the actual topography. When the videos were digitized for computer enhancement, even more resolution was lost.

Although the computer enhancement could highlight areas of similar reflectance, the demonstration showed that these areas were not necessarily areas of similar vegetation. And not all areas of similar vegetation would always have the same reflectance, since reflectance depended on several factors, including phenology and environmental conditions. Therefore, despite the computer enhancement of the images, an interpreter was still required to look carefully through the entire tape. Overall, the computer enhancement required a significant amount of manipulation and interpretation by the operator, and this interpretation appeared to be more difficult than what could be done using color stereo photographs. Reflectance signatures are just one of many clues used in aerial photo interpretation, and with the EnviroScan method the other clues, such as texture and topography, were more obscure than with color aerial photography. The two aerial photo interpreters involved in this study attended the EnviroScan demonstration, and both indicated that they could do much better with standard aerial photos.

Other factors also reduced the feasibility of this alternative. Selecting the best spectral filter would be very difficult because none of the filters was perfectly suited to discriminate a wide variety of wetland vegetation such as we expect to encounter in a statewide inventory. Problems with geographic referencing and map production may be dealt with in future implementations of the system, but at present the techniques are untried. The EnviroScan system may have some potential for wetland inventory use, but the system has never been used for regional wetland identification and mapping, and it seems that several bugs remain.

Cartography and GIS

Techniques were developed to enter interpreted wetland boundaries and classifications as a separate layer of the WSDOT Intergraph system. Some additional work will be needed to quantify and attempt to minimize inaccuracies inherent in the techniques. The available base maps are at 1:24,000 scale, and when these are enlarged to fit the larger aerial photo scales of 1:12,000 and 1:6,000, any errors are magnified. The errors would be greater using the 1:6,000 than the 1:12,000 photo scales. Some additional error may be introduced when the annotated paper maps are fit to the digital base maps. Nonetheless, the maps produced in this trial were deemed to be accurate enough for their intended purpose.

Costs for this portion of the project were almost entirely labor expenses. It was determined that computer costs were insignificant because this project used equipment and materials already in-house for other WSDOT needs. Labor costs did not vary significantly with the different aerial photo scales

TABLE 2 Estimated Costs of Wetland Inventory of 7,030 mi of State Highway

	Estimated Costs[a]			
Method	Materials	Salaries	Travel	Total
Aerial photos, 1:6,000	$428,000	$420,000	$24,000	$872,000
Aerial photos, 1:12,000	$214,000	$420,000	$24,000	$658,000
Aerial photos, 1:24,000	$107,000	$420,000	$24,000	$551,000

[a] Costs include aerial photography, wetland delineation, and entry of wetland boundaries and associated data into the GIS computer system.

used. The task of entering wetland inventory data for the entire state highway system would require one full-time Cartographer 2 working over a 4-year period.

CONCLUSIONS AND RECOMMENDATIONS

The WSDOT statewide inventory of highway rights-of-way should use color infrared aerial photography at 1:12,000. Results of the pilot project and other considerations show that the moderate scale of 1:12,000 is at least as accurate as 1:6,000 and is much less expensive. Although the timing of photography was not evaluated, all consulted experts agreed that the optimum time would be early in the growing season when water levels are high and when differences in vegetative reflectance are maximized.

The pilot project did not clearly distinguish between the accuracy of true color and color infrared photos. The costs are the same, so other factors must be considered. Color infrared has traditionally been the film of choice in the field of wetland delineation from aerial photos. There are some disadvantages to using color infrared photos. Infrared film requires special shipping and storage procedures that make it harder to supply on short notice. Accurate evaluation of color infrared photos requires that the interpreter be experienced in reading infrared photos for the particular area under consideration, since certain reflectance signatures are relied on to distinguish wetland vegetation.

Despite these drawbacks, there is strong support for color infrared from the community of professionals who have been interpreting aerial photos over the years. Virginia Carter, photo interpretation expert with USGS, and Dennis Peters, Northwest Regional Coordinator of the NWI, both claimed to have had very good results delineating wetlands from color infrared photos and recommended that WSDOT should use that film type (personal communications). Considering the close results of our pilot study, these recommendations provide sufficient evidence to suggest that we select color infrared film as the preferred medium.

The selected method will cost approximately $93/highway-mi, including the GIS implementation. This investment will give WSDOT a valuable planning tool. Designers will be able to refer to the inventory and note the presence of wetlands during the development of a project prospectus. Biologists, planners, and maintenance personnel are expected to make frequent use of the maps and other wetland data that will ultimately be available. Field delineations will still be required for projects that affect wetlands, but these will take less time with the inventory data available as a starting place. It will also be easier to find suitable mitigation sites. Finally, by sharing data with other state and local agencies, the WSDOT wetland inventory will contribute to a better understanding of Washington's invaluable wetland resources.

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REFERENCES

1. M. C. Ossinger, J. A. Schafer, and R. F. Cihon. *A Method To Identify, Inventory, and Map Wetlands Using Aerial Photography and Geographic Information Systems.* Technical Report WA-RD 250.1 Washington State Department of Transportation, Olympia, 1991.
2. W. W. Brown. Wetland Mapping in New Jersey and New York. *Photogrammetric Engineering and Remote Sensing*, Vol. 44, No. 3, pp. 303–314.
3. W. G. Howland. Multispectral Aerial Photography for Wetland Vegetation Mapping. *Photogrammetric Engineering and Remote Sensing*, Vol. 46, No. 1, 1980, pp. 87–99.
4. V. Carter. Applications of Remote Sensing to Wetlands. In *Remote Sensing for Resource Management* (C. J. Johannsen and J. L. Sanders, eds.). Soil Conservation Society of America, Ankeny, Iowa, 1982, pp. 284–300.
5. A. Austin and R. Adams. Aerial Color and Color Survey of Marine Plant Resources. *Photogrammetric Engineering and Remote Sensing*, Vol. 44, No. 4, 1978, pp. 469–480.
6. L. M. Cowardin, V. Carter, F. C. Golet, and E. T. LaRoe. *Classification of Wetlands and Deepwater Habitats of the United States.* FWS/OBS-79/31. U.S. Fish and Wildlife Service, 1979.

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Calcium Magnesium Acetate Degradation in Roadside Soil: Acetate Microcosms

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Aseptic soil samples from the loam cover of a state highway shoulder in southeastern Massachusetts were placed in sterile serum bottles, forming a series of aerobic soil microcosms. The samples were dosed with reagent grade acetate solutions without acclimation, then sacrificed at various time intervals and analyzed by gas chromatography in a laboratory determination of the aerobic microbial degradation kinetics. The acetate degraded rapidly in the loam layer, demonstrating that the shoulder has the potential to reduce oxygen demand by acetate on groundwater under the highway.

The aerobic degradation of acetate in roadside soils was measured in soil microcosms in this study. The work provides data on the fate and transport of the deicing agent calcium magnesium acetate (CMA) in the subsurface environment as it migrates from plowed snow on the highway shoulders through the unsaturated zone to the water table. In this regard, the aerobic decomposition of the acetate consumes oxygen in the subsurface environment, so that there is concern for potential oxygen depletion if an appreciable amount of acetate reaches the underlying unconfined aquifer (*1*).

FHWA, in an effort to weigh this possibly adverse impact against the many benefits associated with CMA use, has funded a series of research projects (*2*), including a thorough environmental study of a pilot scale test runoff plot in Washington State (*3*). The FHWA initiative, administered by the New England Transportation Consortium (NETC), extends the effort to the field by studying a highway in southeastern Massachusetts that has been deiced with CMA since it opened to traffic in August 1987. The laboratory work reported here is the first phase of the NETC project, which will feature additional microcosms, field measurements of acetate profiles, and comparative analyses of the Washington pilot scale and Massachusetts field scale data.

FIELD SITE AND SAMPLING METHODS

The research site is in southeastern Massachusetts along State Route 25. The test area is on the north-sloping highway shoulder that was constructed with native sandy fill covered with a nominal 0.20-m layer of loam. The road was opened in August 1987 under environmental constraints requiring the use of nonchloride deicing chemicals in the survey area, which receives plowed snow, airborne drift, and breakdown lane runoff (but no travel lane runoff) from the three 3.66-m-wide westbound lanes of this divided highway. About 25,000 vehicles travel on State Route 25 each day, and CMA has been applied at an annual rate of about 0.71 kg/m^2 pavement in response to 0.94 m of annual snowfall, on the basis of 1987–1988 data. This loading implies a representative surface CMA concentration of 700 mg/L, on the basis of 50 percent of the applied agent distributed over 20 m of the northern highway shoulder as drift, breakdown lane runoff, and plowed snow.

Environmental Engineering Program personnel sampled the northern slope on June 27, 1991, from the pared wall of a soil pit 0.80 m deep located 2.44 m from the edge of the breakdown lane. Soil samples of about 0.01 kg were taken from the two depths cited in Table 1 with the deepest location sampled first, using autoclaved 10-mL Becton Dickinson disposable syringe barrels and acid-washed, milliQ-rinsed, autoclaved 20-mL serum bottles with aerated headspace, Teflon closures, and plastic screw caps. The serum bottles were iced and sent back to the laboratory for storage and testing in a Fisher Scientific Mini Lo-Temp incubator at 5°C. Soil moisture, pH, and grain size distributions were also measured at the two depths using conventional methodology; the results are given in Table 1.

MICROCOSMS AND CHROMATOGRAPHY

The serum bottles served as aerobic soil microcosms; they were dosed without prior exposure to acetate (no acclimation). The spiking liquid was a nominal 1000 mg/L solution of reagent grade glacial acetic acid, buffered to its nonvolatile (ionic) acetate state with sodium bicarbonate to a pH of 6. The spiking solution was added while the bottles were held sideways using a Becton Dickinson 3-mL disposable sterile syringe with a bevel-tipped 20-gauge needle 0.064 m long in five separate injections spaced uniformly over the exposed surface of the soil. The soil was gently shaken to mix the spiking solution into the matrix with sufficient liquid added to bring the soil to about half saturation, facilitating sample extraction and analysis without unduly displacing oxygen from the porespace. Autoclaved microcosms were similarly spiked

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TABLE 1 Soil Characteristics and Calibrated Kinetics

Depth m	Mean Grain Size, mm[a]	Fine Fraction[b]	Moisture Content kg water / kg wet soil	pH	V mg/l-hr	K mg/l
0.025	0.80	0.152	0.0258	6.5	16.4	148
0.153	0.82	0.126	0.0496	5.5	5.29	68

[a]As fit with a Van Genuchten (4) distribution.

[b]Less than 0.15 mm.

to check for abiotic losses, and ample headspace existed in the microcosms to provide abundant oxygen.

Replicate bottles and abiotic controls were sacrificed at varying time intervals by the addition of 6 mL of distilled water to dissolve the microcosm fluid, followed by extraction (agitation, centrifugation, and filtration). The filtrate (0.9 μL) was then injected with a 0.1-μL plug of oxalic acid through a flash vaporization injector at 200°C into a Perkin Elmer Sigma 1 gas chromatograph (GC) using a 2-μL Hamilton 7000 series syringe with a Chaney adaptor. The oxalic acid plug lowered the pH to below 2, converting essentially all the acetate into its volatile acetic acid form for GC analysis in a 1.83-m-long, 2 mm ID packed glass column in a constant oven temperature of 165°C. A 25-mL/min zero-grade nitrogen gas flow carried the separated sample to a flame ionization detector set at 250°C, and daily calibration curves related instrument response to acetate concentration. Blanks and replicate injections were run routinely to verify quality control and instrument performance. Instrument sensitivity was about 1 mg/L, some three orders of magnitude below the maximum extract concentrations observed in the experiments.

DATA AND REACTION KINETICS

Figure 1 summarizes the degradation data observed at the two depths, along with calibrated Michaelis-Menton kinetic curves fit to the observations. Ambient concentrations were less than 1 mg/L at the site. The recovery efficiency from the abiotic controls varied between 91 and 118 percent and was used to adjust the biotic data as part of the calibration. In the assumed abundance of oxygen and nutrients, the substrate limited reaction is governed by a half-saturation constant K and a maximum reaction rate V in accordance with Alexander and Skow (5).

$$\frac{dc}{dt} = \frac{Vc}{c + K} \tag{1}$$

$$c = \frac{\text{mass acetate}}{\text{volume soil water}} \tag{2}$$

with concentration c and time t since spiking began. Oxygen limitations would impose an additional multiplying factor on the Michaelis-Menton term with its own half-saturation constant (6). In this absence of this effect, the theoretical degradation rate yields a temporal decay of observed concentra-

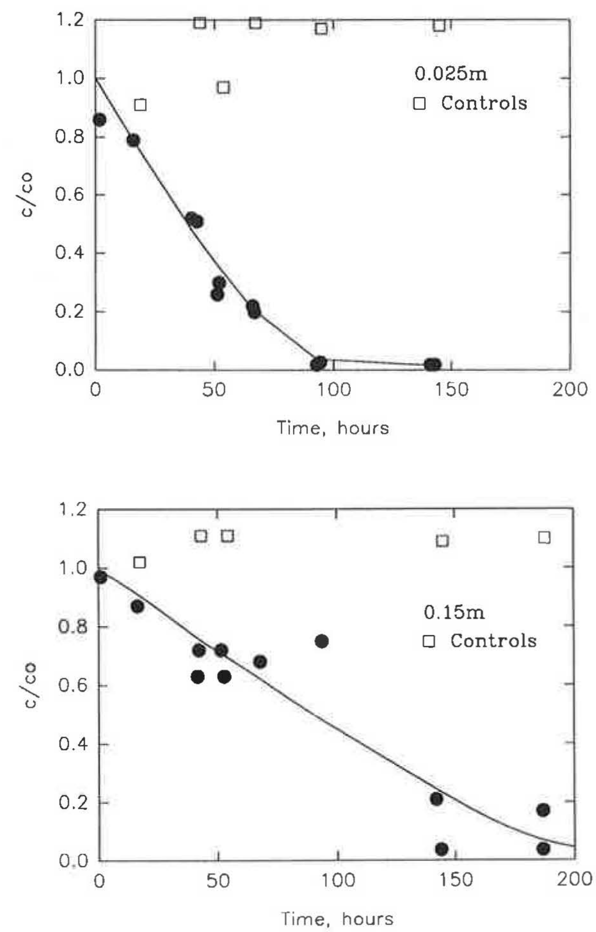

FIGURE 1 Observed (circles) and predicted (curves) acetate concentration in soil microcosms as function of time for loam cover of road shoulder 0.025 m deep (*top*) and 0.15 m deep (*bottom*), adjusted for abiotic losses (squares); predictions reflect *V*- and *K*-values of Table 1.

tion when Equation 1 is integrated from its initial concentration c_0 to any subsequent condition (7)

$$t = \frac{K}{V}\ln\left(\frac{c_0}{c}\right) + \frac{c_0 - c}{V} \tag{3}$$

The half-saturation constant and maximum reaction rate values cited in Table 1 were used to zero the error mean and standard deviation through a nested Fibonacci curve-fitting subroutine (*8*). The half-saturation constant varied from 70 to 150 mg/L, and the maximum reaction rate decreased from a high near surface loam value of 16.4 mg/L-hr to a lower value of 5.3 mg/L-hr near the sand interface. The respective one- and two-day half-lives implied by Figure 1 represent relatively rapid aerobic degradation in a natural setting.

DISCUSSION OF RESULTS

The experiments were run without acclimation to prior doses of acetate substrate and as such represent the initial response the roadside soil to CMA applications early in the salting season. Typically, the microbial population would grow in response to repeated exposures to the substrate (*5*), thereby increasing the speed of reaction so long as nutrients and oxygen are present in abundance. In the latter regard however, the high solubility of acetate and relative insolubility of oxygen (roughly 10 mg/L) suggest that oxygen may control the speed of the reaction in water saturated soil of low permeability. Thus the loam cover, which is likely to be saturated near the surface with deiced runoff due to its low permeability, may not perform at the rapid rates of Table 1 under field conditions. The sandy fill is much more permeable than the loam and is considerably drier (samples collected in February 1991 contained a moisture content of 0.035 kg water/kg wet soil in the sand compared with 0.17 in the loam). Oxygen should not limit the degradation rate in the sand as a consequence; a comparable set of microcosms is currently being run on the sandy fill to quantify its kinetics.

Approximate calculations suggest that there is enough oxygen in the unsaturated zone to satisfy the estimated 30 000 kg of acetate that annually infiltrates the shoulders and median of the highway in the survey area, which is 1930 m long and 100 m wide. The stoichiometry for complete aerobic mineralization of acetate is given by

$$2C_2H_3O_2^- + 3.5O_2 \rightarrow 3H_2O + 4CO_2 \tag{4}$$

Thus 0.95 kg of oxygen are required to consume 1 kg of acetate, corresponding to an annual demand of about 28 000 kg oxygen imposed by CMA application in the study area. Allowing for a 30 percent reduction of oxygen through the root zone due to ambient plant and soil activity (*9*), the 5-m-deep, 0.20–air porosity unsaturated zone would contain 37 000 kg of O_2 in the absence of CMA loading. Thus aerobic conditions are expected to prevail in the unsaturated zone.

If the microbes are rapid enough (and the infiltration rate is slow), the acetate will degrade before the percolation reaches the water table and the deficit of oxygen will not extend to the underlying unconfined aquifer. One would then expect a winter deficit of oxygen in the unsaturated zone due to CMA degradation, followed by diffusive reaeration in drier summer and fall months. Rapid infiltration and slow reactions, on the other hand, will permit acetate penetration to the water table with a resulting oxygen demand on the groundwater. The interaction of reaction and infiltration is currently being modeled mathematically in order to quantify the transport processes and potential oxygen demand at the research site.

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REFERENCES

1. I. L. Stevenson and G. H. Katznelson. The Oxidation of Ethanol and Acetate in Soils. *Canadian Journal of Microbiology*, Vol. 4, 1958, pp. 73–79.
2. B. H. Chollar. Federal Highway Administration Research on Calcium Magnesium Acetate—An Alternative Deicer. *Public Roads*, Vol. 47, 1984, pp. 113–118.
3. R. R. Horner. *NCHRP Report 305: Environmental Monitoring and Evaluation of CMA*. TRB, National Research Council, Washington, D.C., 1988.
4. M. T. Van Genuchten. A Closed Form Solution for Predicting the Hydraulic Conductivity of Unsaturated Soils. *Soil Science Society of America Journal*, Vol. 44, 1980, pp. 892–898.
5. M. Alexander and K. M. Skow. Kinetics of Biodegradation in Soil. *Reactions and Movement of Organic Chemicals in Soils*. SSSA, Madison, Wis; 1989, p. 243–269.
6. F. J. Molz, M. A. Widdowson, and L. D. Benefield. Simulation of Microbial Growth Dynamics Coupled to Nutrient and Oxygen Transport in Porous Media. *Water Resources Research*, Vol. 22, 1986, pp. 1207–1216.
7. D. W. Ostendorf and D. H. Kampbell. Bioremediated Soil Venting of Light Hydrocarbons. *Hazardous Waste and Hazardous Materials*, Vol. 7, 1990, pp. 319–334.
8. G. S. G. Beveridge and R. S. Schechter. *Optimization: Theory and Practice*. McGraw-Hill, New York, N.Y., 1970.
9. D. W. Ostendorf and D. H. Kampbell. Biodegradation of Hydrocarbon Vapors in the Unsaturated Zone. *Water Resources Research*, Vol. 27, 1991, pp. 453–462.

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Total Petroleum Hydrocarbons in Highway Maintenance Waste

CRAIG A. MARTIN, ERVIN HINDIN, AND ED HANNUS

Highway maintenance wastes consisting of road sweepings, vactor sludges, and ditch spoil were contaminated by semivolatile and nonvolatile hydrocarbons. The total petroleum hydrocarbon (TPH) content in road sweepings ranged from 2 to 16 966 mg TPH/kg; that in vactor sludges and ditch spoils ranged from 251 to 7690 and 214 to 2541 mg TPH/kg, respectively. The TPH content of the fine particle–size fraction of the road sweeping was significantly higher than that present in the coarser fractions. Weathered road sweepings had a lower TPH concentration than did the fresh sweepings.

The disposal of road sweepings, vactor sludges, and ditch spoils from highway maintenance operations is coming under closer environmental scrutiny by federal, state, and local regulatory agencies. These wastes have long been recognized, in themselves, as being potentially hazardous materials or serving as surfaces onto which hazardous substances can sorb.

Because of increasing liability and monitoring costs, municipal landfills are reluctant to receive the materials because they may contain hazardous substances. Consequently, accumulation and storage of these materials occur at highway department maintenance sites at which they have become a disposal problem. As a result, highway departments are concerned with the proper management of these materials. Many states have established guidelines for disposal of waste materials. Many states have established guidelines for disposal of waste materials emphasizing the protection against contamination of surface and ground waters. Because there is little information about the contaminants within the waste materials, it is difficult to assess if the current waste management practices are effective.

The use of motor vehicles on roadways generates wastes containing hazardous heavy metals and organic compounds. Such heavy metals as lead, cadmium, and zinc are found, in a relatively immobile form, in road sweepings, vactor sludge, and in spoils from ditches adjacent to heavily traveled roads. These wastes contain a wide variety of organic compounds from the condensed gases and particulates from the internal combustion engine as well as oils and greases lost from the vehicles lubricating system.

The identity of specific organic compounds in the highway environment is only fragmentary. Because of the many sources of the contaminants, the types of compounds present cover a wide spectrum of hydrocarbons. Uncombusted gasoline contributes volatile hydrocarbons with low molecular weight such as short-chained alkanes and single-ring aromatics. Heavier semi- and nonvolatile hydrocarbons, such as long-chained straight and branched alkanes and polycyclic aromatics, are found in oil, greases, engine exhaust, asphalt deposits, and tire wear.

Early research of street sweepings is monitored by the Chemical Oxygen Demand (COD) and the Biochemical Oxygen Demand (BOD) (*1,2*). These studies focused on the impact of biodegradable organic contaminants on receiving waters.

Polycyclic aromatic hydrocarbons are a group of compounds that have been widely documented to be present in vehicle exhaust and street dust (*3–6*). These compounds are present in gasoline and oils and are also formed in combustion processes. Several members of the polycyclic aromatic hydrocarbon group are known carcinogens and are on the Environmental Protection Agency's (EPA's) Priority Pollutant and Washington State's Dangerous Waste lists (*7,8*).

The objective of this paper is to document information about the characterization and fate of organic contaminants, as measured by total petroleum hydrocarbons (TPHs), in wastes generated from highway maintenance operations.

RESEARCH PLAN

Site Selection

Samples were collected from waste piles at Washington State Department of Transportation (WSDOT) maintenance sites and along state highways. A map of the state of Washington, showing the sample site locations, is shown in Figure 1. Sample-site locations are indicated by filled circles. The number in parentheses following the sample-site name represents the number of waste piles sampled at that location. The numbers shown in large print (1 through 6) represent the six highway districts within WSDOT.

Samples were taken from three types of waste material: road sweepings, vactor sludges, and ditch spoils. Piles of varying ages were sampled to investigate the weathering effect on TPH concentrations. They ranged from samples of recently collected roadside piles to well-weathered piles at highway department maintenance sites. The waste piles ranged in size from a few to several thousand cubic yards. The type of waste material, origin of waste, and age of waste pile were among the data recorded.

Sample Collection

Samples were obtained by digging into the sides of waste piles at five or six locations and removing subsamples. Samples

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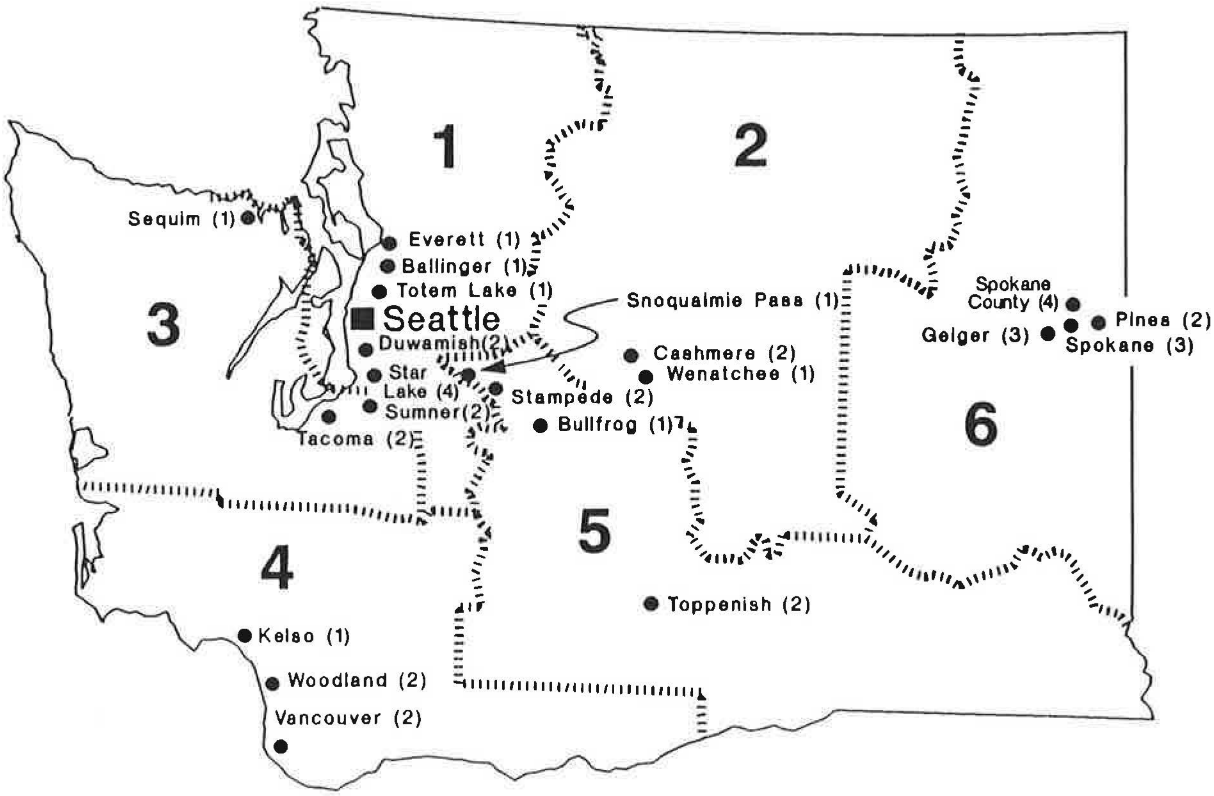

FIGURE 1 Sample-site locations.

were taken below the pile surface to a depth of about 2 ft. Subsamples were then mixed, and a composite sample was taken. The samples were placed in glass containers, iced, and transported back to the laboratory. Samples were then refrigerated at 4° C until extracted.

Total Petroleum Hydrocarbon

Since a wide range of organic compounds are present in highway maintenance wastes, a parameter was chosen that would measure a broad spectrum of compounds in a single analysis. The TPH analysis measures volatile, semivolatile, and nonvolatile hydrocarbon compounds, exception for the aromatic compounds present in the waste materials. The analysis does not identify specific hazardous substances in samples, but it is an effective screening method. Because there is no established method for measuring TPH in soils, a combination of Standard Method 5520E and EPA Method 418.1 was used (*9,10*). This procedure is valid for a TPH concentration range of 5 mg/kg to approximately 10,000 mg/kg (*11*). Several states use this procedure to analyzing petroleum-contaminated soils resulting from leaking underground storage tanks (*12*).

A Foxboro Co. MIRAN 1A CVF infrared analyzer was used in the analysis. The TPH parameter being a relative parameter is dependent upon the type of standard used. An EPA-recommended standard consisting of a mixture of n-hexadecane, isooctane, and chlorobenzene was used (*10*). Benzene, hexane, heptene, used motor oil, and lube oils were used as comparative standards. TPH concentrations given in this paper were based on the EPA standard.

Particle Size Distribution

An experiment was conducted to determine the particle size distribution of the waste materials and the concentration of TPH in the particle size fractions. The samples were air-dried and separated into three particle size fractions by mechanical sieving. The three particle sizes used in this study were as follows:

Particle Size	*Description*
> 2 mm	Coarse fraction consisting of gravel and large debris
< 2 mm > 250 μm	Medium fraction consisting mostly of small gravel, sand, silt, and vegetative matter
< 2 mm	Fine fraction consisting of loam and clays

Samples from each of the categories were analyzed for TPH.

RESULTS AND DISCUSSION

Sample Collection

A total of 39 composite samples were collected. The number of samples from each of the three waste categories is shown in Table 1.

Quality Control Analysis

Quality control analysis was performed using blanks, spiked blanks, spiked samples, and background samples to determine the accuracy and sensitivity of the analytical procedures.

TABLE 1 Summary of Samples by Waste Category

Type of Waste Pile	Number of Samples
Road sweepings	26
Vactor sludge	8
Ditch Spoils	5

Several samples were spiked with the EPA-recommended standard to further demonstrate the effectiveness of the extraction and analytical procedures. Table 2 shows the percentage recovery of the spiked samples. Low recovery was attributed in part to the heterogeneity of the sample and the difficulty in determining the "true" TPH concentration of the sample before spiking.

TPH Concentration of Waste Categories

Samples were analyzed in duplicate; selected samples were analyzed in greater replication. The results for each sample were averaged. Table 3 shows the range and mean values of TPH concentration for samples of each waste type. Both arithmetic and geometric means were determined. This analysis was conducted on a wet-weight basis, and moisture content was not accounted for. Wet-weight analysis was used because drying would result in the loss of volatile substances.

The results indicated that vactor sludges and road sweepings were higher in TPH than ditch spoils. It was apparent from the results that TPH concentrations of the waste materials vary greatly. Since a wide range of TPH concentrations was found in the waste materials, the geometric mean is a more accurate estimate of the average value.

Effects of Waste Pile Weathering on TPH Concentrations

During sample collection, information was obtained that allowed categorization by relative waste pile age. Waste pile age was determined using information obtained from local WSDOT personnel as well as by visual inspection of the waste piles. Although this was a qualitative method of estimating age, the sampled materials were separated into three road sweeping categories and two vactor sludge groups.

The freshest road sweepings were those that were found in recently swept small piles along the highway. These piles were typically less than a few weeks old. The second category of road sweepings were piles that had recently been deposited at highway maintenance sites. These piles, although generally older than the piles found along the highway, were not more than a few months old. The last category was for well-weathered road sweeping piles found at the highway maintenance sites. These piles were typically more than 6 months old.

The vactor sludges were classified as either wet (fresh) or dry (aged) waste piles. Wet sludges contained free water, and dry sludges had been allowed to drain and dry. Ditch spoils were not classified by age since only five samples were collected. Table 4 is a tabulation of the mean and range of concentration values for the weathered road sweeping categories and vactor sludges.

The results indicate that weathering of waste piles of road sweepings reduces the concentrations of TPH. These results were expected, because natural processes would reduce the concentration of contaminants over time. However, the effect of weathering was not obvious in vactor sludge samples.

Particle Size Analysis

Figure 2 shows the particle size distribution of typical samples from fresh sweepings and well weathered sweepings piles. The percentage of the fine fraction (<250 μm fraction) in the fresh road sweepings was found to be consistently higher than the weathered sweepings samples. This was attributed to the loss of fine fractions in the older piles by wind or runoff.

Figure 3 shows the concentrations of TPH in the three particle size fractions for the two samples indicated above. This analysis was conducted on a dry sample weight basis.

TABLE 2 Quality Control-Spike Sample Recovery

Spike Blank Number	Spike (mg TPH)	Sample TPH no spike (mg TPH/kg)	Sample TPH plus spike (mls)	% Recovery (of spike)
43	100	8082	12184	72
44	100	16966	21408	79
1	107	4157	8095	66
29	116	2467	7204	48

TABLE 3 TPH Concentrations in Waste Categories

Waste Type	TPH Concentration (mg (TPH/kg)		
	Arithmetic Mean	Geometric Mean	Range
Road sweepings	2524	1054	2-16966
Vactor sludge	2884	1788	251-7690
Ditch Spoils	954	664	214-2541

TABLE 4 Effect of Weathering on TPH Concentrations

Waste Type	Number of Samples	TPH Concentration (mg/kg) Arithmetic Mean	Geometric Mean	Range
Fresh roadside sweepings	5	3307	3215	2410-4157
Fresh sweepings at maintenance site	9	4560	2870	825-16966
Well weathered sweepings	12	671	312	2-2009
Wet vactor sludge	5	2503	1604	251-5787
Dry vactor sludge	3	1070	2412	553-7690

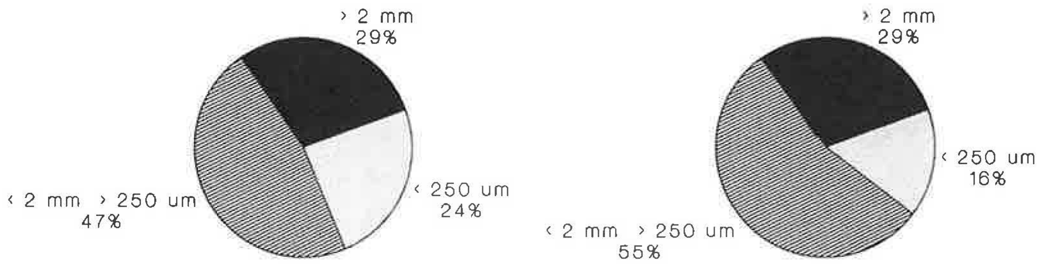

FIGURE 2 Particle size distribution, weight percent of particle size fractions: *left*, fresh roadside sweepings (Sample 29); *right*, well-weathered sweepings (Sample 6).

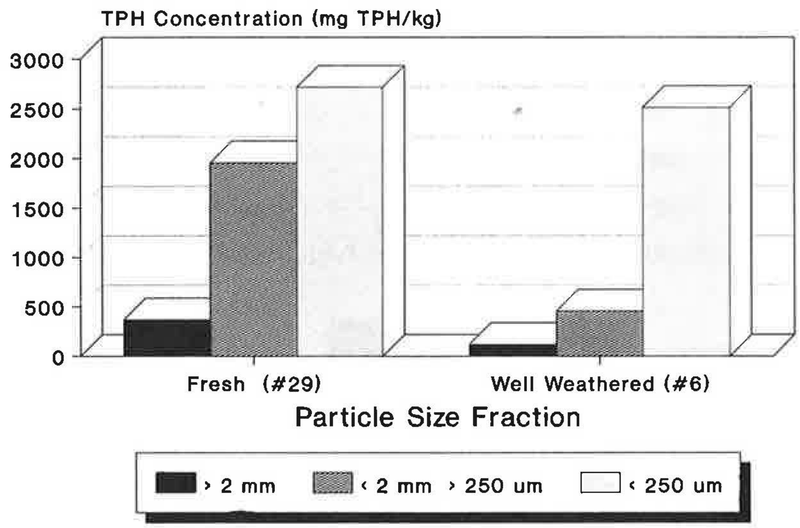

FIGURE 3 TPH concentration of particle size fractions.

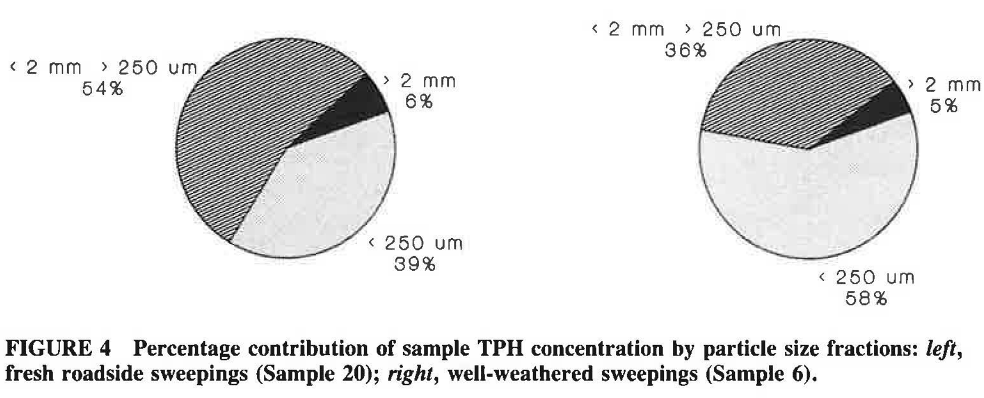

FIGURE 4 Percentage contribution of sample TPH concentration by particle size fractions: *left*, fresh roadside sweepings (Sample 20); *right*, well-weathered sweepings (Sample 6).

Figure 4 illustrates that TPH is more concentrated in the finer fractions. This is a common finding with contaminated soils. Sorption of organic compounds is highest by clay mineral particles that are included in the <250 μm fraction.

Figure 3 further emphasizes the importance of particle size on TPH concentration. In the two samples shown, the coarse fraction (>2 mm) makes up almost a third of the sample (by weight) and the TPH contribution is only 5 and 6 percent of the total sample.

SUMMARY AND CONCLUSIONS

This study has shown that highway maintenance wastes contained a wide concentration range of TPH. Road sweepings and vactor sludges generally were found to have higher concentrations of TPH than ditch spoils were.

Weathering of waste piles was found to reduce TPH concentration. Fresh piles of road sweepings along highways and at maintenance sites had much higher TPH concentrations than did well-weathered piles. Fresh road sweepings were found to have the highest concentrations of TPH compounds. Older waste piles were found to have lower concentrations. A correlation between waste pile weathering and reduction of TPH concentration was not found for the vactor sludge piles. The effect of weathering of ditch spoil piles was not determined.

TPHs were found to be concentrated in the smaller particle size fractions of the waste materials. The highest TPH concentrations were associated with the <250 μm fraction. The TPH concentration in the >2 mm fraction was generally found to be insignificant as compared to the total sample TPH.

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REFERENCES

1. *Water Pollution Aspects of Street Surface Contaminants*. EPA-R2-72-081. Environmental Protection Agency, Nov. 1972.
2. *Analysis of Street Sweepings, Portland, Oregon*. Open File Report. U.S. Geological Survey, 1973.
3. Takada et al. Determination of Polycyclic Aromatic Hydrocarbons in Urban Street Dusts and Their Source Materials by Capillary Gas Chromatography. *Environmental Science Technology*, Vol. 24, No. 8, 1990, pp. 1179–1185.
4. Blumer et al. Polycyclic Aromatric Hydrocarbons in Soils of a Mountain Alley: Correlation with Highway Traffic and Cancer Incidence. *Environmental Science and Technology*, Vol. 15, 1975, pp. 1082–1084.
5. Giger and Schaffner. Determination of Polycyclic Aromatic Hydrocarbons in the Environment by Gas Capillary Gas Chromatography. *Analytical Chemistry*, Vol. 50, No. 2, Feb. 1987, pp. 243–249.
6. Butler et al. Some Observations on the Polycyclic Aromatic Hydrocarbon (PAH) Content of Surface Soils in Urban Areas. *The Science of the Total Environment*, Vol. 33, 1984, pp. 75–85.
7. Environmental Protection Agency. *National Pollutant Discharge Elimination System*. 40 C.F.R., Part 122, 1988.
8. *WAS Dangerous Waste Regulations*. Washington Department of Ecology, June 1986, Chapter 173–303.
9. *Standard Methods for the Examination of Water and Wastewater*. 1990.
10. *Test Methods for Evaluating Solid Wastes*. Physical/Chemical Methods SW-864. Environmental Protection Agency.
11. *Manual of Sampling and Analytical Methods for Petroleum Hydrocarbons in Groundwater and Soil*. API Publication 4449.
12. TPH State Guidelines. *Soils Contaminated by Petroleum*. Wiley, New York, N.Y., 1988.

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PART 3

Air Quality

Impact of Preaggregation of Highway Network Travel Data on Accuracy of MOBILE4-Based Emissions

W. THOMAS WALKER

Preliminary Environmental Protection Agency guidance for the generation of mobile source emissions inventories indicates that a link-level computation based on simulated network volumes is preferred in urban areas. However, because of the size and complexity of some link-level emissions computations, a streamlined postprocessor based on 5-km grid aggregates of network vehicle miles traveled (VMT) and speed may also be useful to facilitate sensitivity testing and refinement of the MOBILE4 parameters used to generate an emissions inventory and to plan for emissions reductions. The results are presented of a series of accuracy checks made to ensure that the preaggregation of network VMT and speed does not introduce significant errors or biases into the resulting emissions estimates. These comparisons also shed some light on network aggregation errors that result from generating gridded emissions from Highway Performance Monitoring System VMT and speed data on other nonnetwork sources. The results clearly show that a postprocessor methodology is adequate to calculate emissions when the VMT and speed inputs are stratified by highway functional class. This does not mean that traffic simulation networks are not needed for preparing emissions inventories and planning for emissions reductions. When available, travel simulations are the most consistent and cost-effective way to estimate current and forecasted VMT and speed. However, for a given network assignment, using the postprocessor can expedite and streamline the calculation of emissions with little loss of accuracy or consistency.

In support of air quality planning activities required by the Clean Air Act Amendments of 1990, the Delaware Valley Regional Planning Commission (DVRPC) has been asked to prepare an inventory of mobile source emissions of three pollutants—volatile organic compounds (hydrocarbons), carbon monoxide, and oxides of nitrogen—for the Philadelphia ozone nonattainment area. This inventory will be prepared for a 5-k grid system that covers DVRPC's Pennsylvania and New Jersey counties, as well as Cumberland and Salem counties in New Jersey, Kent and New Castle counties in Delaware, and Cecil County, Maryland (see Figure 1).

It is anticipated that developing the emissions inventory will require the use of alternative emissions scenarios using different combinations of MOBILE4 parameters and seasonal travel factors. MOBILE4 parameters incorporate factors such as control device tampering rates, vehicle type and age distributions, inspection and maintenance programs, vehicle refueling practices, ambient temperatures, cold start percentages, and fuel volatility into the emissions calculation. Taken together, these factors have a great impact on the quantity of emissions produced.

Each of the four states in the Philadelphia nonattainment area is likely to use a different set of emissions factors consistent with their air quality programs. Calibrating the urban airshed model will require the reestimation of mobile source emissions inputs that reflect the conditions that occurred on the day of the episode being modeled. Sensitivity analyses will also be useful for preliminary testing and evaluation of emissions reduction strategies.

Figure 2 presents an overview of the proposed emissions inventory process. Inputs are shown in dashed boxes, and outputs are specified in solid-lines boxes. This process contains two emissions calculators shown in the heavy-lined boxes. The base case processor obtains simulated vehicle miles trav-

FIGURE 1 Philadelphia ozone nonattainment area.

Delaware Valley Regional Planning Commission, The Bourse Building, 21 South Fifth Street, Philadelphia, Pa. 19106.

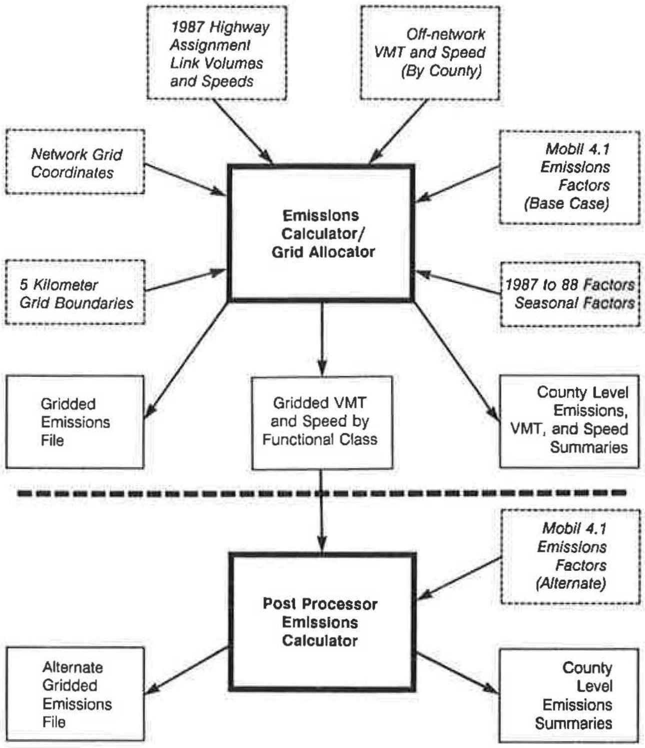

FIGURE 2 MOBILE source emissions inventory process.

eled (VMT) and speed data from a computerized highway network, calculates link-level emissions that reflect the base case set of seasonal and MOBILE4 emissions factors, and then allocates the emissions, VMT, and highway speed to each 5-km grid traversed by the link. This allocation is based on the proportion of the link distance within that grid. VMT that occurs on local streets not included in the regional network will be separately allocated to grids from county-level control totals.

Preliminary guidance from the Environmental Protection Agency (EPA) for the generation of mobile source emissions inventories indicates that a link-level computation based on simulated network volumes is preferred in urban areas (*1*). However, the participating state air quality agencies have requested that streamlined computational procedures also be developed to allow them a hands-on capability to fine-tune the MOBILE4 parameters that are used to generate the emissions inventories for the portions of the air quality region under their jurisdictions. For this reason, the postprocessor is included in the inventory process to recalculate grid-level emissions that reflect alternative sets of emissions factors.

The postprocessor emissions calculator is a simple matrix manipulator that considers MOBILE4 emissions factors by speed range and two highway demand matrices aggregated from link-level data by the base processor. These matrices, one containing VMT and the other average speed, are cross-classified by grid and highway functional class (freeway, arterial, and local/collector). Emissions for a given grid are calculated by multiplying the VMT for each functional class by the emissions factor appropriate for the average speed of that roadway class. Both base and postprocessors produce state- and county-level VMT, speed, and emissions summaries to assist in controlling and interpreting the inventory process.

A postprocessor is required to expedite the development of emissions scenarios because of the large size and diverse nature of the highway travel inputs to the base case processor. Ultimately, this calculation will use data from three highway networks containing about 60,000 links, Highway Performance Monitoring System (HPMS) data for three rural counties, and off-network VMT grid allocations for 14 counties. Although it is shown as a single box, the base case processor involves a number of separate computer programs, some of which must be run on a mainframe computer. Although the computation time and disk storage requirements for the link-level computation are not large for a modern mainframe (about 5 min on an IBM 4361 and 5,000 K of disk storage), the postprocessor reduces this time to about 8 sec and data storage to about 200 K and, more important, allows the downloading of both data and computation to a microcomputer for distribution to state and local air quality agencies.

However, using the postprocessor for this requires that a series of accuracy checks be performed to ensure that the preaggregation of VMT and speed from individual highway links to 5-km grids does not introduce significant errors or biases into the resulting emissions estimates. If speeds on individual links vary widely within a given grid, the nonlinearity of emission factors with respect to speed could cause significant differences. This paper presents the results of a series of accuracy checks that compared network-generated emissions with corresponding results from the postprocessor program. These accuracy comparisons will also shed some light on network aggregation errors that result from generating gridded emissions estimates from HPMS VMT and speed data and other nonnetwork sources. However, one should remember that gridded VMT based on HPMS data will also contain significant statistical errors because of the crude methods used to allocate county totals to grids and the small sample of highway links that are counted as part of this FHWA program.

DVRPC'S REGIONAL TRAVEL SIMULATION

The travel simulation models in use at DVRPC follow the traditional steps of trip generation, trip distribution, modal split, and travel assignment (minimum path, equilibrium capacity restraint for highway). They use computer programs included in the federally sponsored Urban Transportation Planning System (UTPS). The 1987 travel simulation, used as a basis for this analysis, was prepared from traffic zone–level socioeconomic trip generation inputs (households, automobile ownership, employed residents, and employment by place of work) extrapolated from the 1980 census and highway and public transit networks updated to include all facilities opened to traffic in 1987. This 1987 travel simulation is documented in detail in a commission report (*2*). Of particular interest to this exercise is the highway network and simulated traffic volumes and the method used to convert link-level simulated volume/capacity ratio into average operating speed. The 1987 simulated highway volumes were validated with traffic counts using FHWA screenline methods.

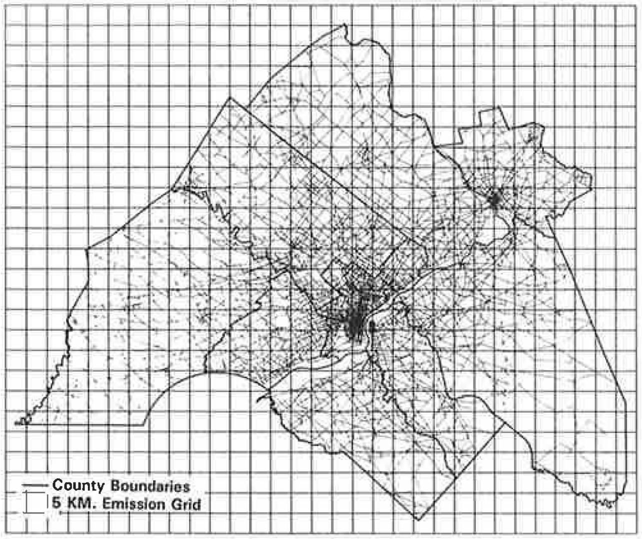

FIGURE 3 DVPRC 1987 highway simulation network.

Figure 3 displays a regional plot of the links included in the 1987 highway network with the 5-km grid system to be used for airshed modeling superimposed. The highway network covers only the DVRPC region shown in crosshatch in Figure 1. This network is very large, covering the 3,580-mi^2 DVRPC region at an average density of about 110 network arcs per 5-km grid.

Overall, the network (see Table 1) contains 1,449 traffic centroids, more than 12,500 nodes, 22,500 two-way links and 2,211 one-way links, which generate 47,227 network arcs for purposes of minimum path building and highway assignment (a two-way link generates two arcs; a one-way link, one arc).

TABLE 1 1987 Highway Simulation Network Statistics

Area Covered	3,846.8 Square Miles
Traffic Centroids	1,449
Nodes	12,533 (including centroids)
Two-way Link Cards	22,508 (including centroid connectors)
One-way Link Cards	2.211
Network Arcs	47,227
Average Arcs/Grid	109.6

Highway Route Miles

Functional Class	Computerized Network	Total Open to Traffic	% in Network
Freeway	739	739	100.0
Arterial	4,348	4,348	100.0
Collector/Local	1,580	14,464	10.9
TOTAL	6,667	19,551	34.1%

This network contains some 6,667 centerline-mi of roadway, which constitutes virtually all freeways and arterial facilities but only 11 percent of the local roads. These local facilities are mostly minor streets within local communities, industrial parks, and residential subdivisions. Local roads usually carry small traffic volumes and, in total, are thought to contribute about 12 percent to regional VMT, despite making up about two-thirds of the region's roadway mileage (*3*)

Emission factors calculated by MOBILE4 vary with vehicle operating speed to a very significant degree. For this reason, the amount and distribution of the mobile source pollutants are influenced by the accuracy and sensitivity of the method used to convert network measures of highway congestion into operating speed. Highway travel time studies conducted by the commission have shown that the so-called FHWA restraining curve has a severe tendency to underestimate operating speeds in the Delaware Valley region (by as much as 50 percent). This speed function is intended to facilitate the simulation of accurate link volumes (via capacity restraint) more than to produce realistic operating speeds. The use of this function to estimate simulated operating speeds would result in severely overestimated emissions. The FHWA restraining curve is given by

$$T = T_0[1.0 + 0.15(V/C)^4]$$

where

T = adjusted link travel time,
T_0 = unloaded link travel time,
V = assigned volume,
C = capacity of the link, and
maximum V/C = 4.0.

For this analysis, a complex but more accurate set of curves was used to estimate simulated operating speeds. These curves were taken from a report prepared by Creighton, Hamburg, Inc., for FHWA (*4*). A separate set of curves was used for freeways and arterials. The freeway curves relate peak-hour link operating speed to the link speed limit, capacity, and peak-hour simulated vehicular volume. The arterial curves relate peak-hour link speed to the speed limit, capacity, traffic signal density (per mile), a free-flow speed, and the peak-hour simulated link volume. The freeway curve required no modification for use in the DVRPC region. The arterial curve, however, required the addition of a minimum speed of 8 to 10 mph (depending on area type) to the Creighton, Hamburg formulation to adequately replicate DVRPC's travel time survey data. The revised curves are determined by the following equations:

Modified Creighton, Hamburg surface arterial equations:

$$S(w_s,n) = \frac{3,600}{\left(\frac{3,600}{w_s}\right) + 12.5n}$$

$$f(n) = -0.336n^3 + 3.905n^2 - 16.116n \qquad \text{if } n < 5.5$$

$$f(n) = 0.69n - 39.14 \qquad \text{if } n \geq 5.5$$

For $d_s \leq 0.8\, c_s$

$$a = S(w_s,n) + d_s f(n)$$

For $1.4c_s > d_s > 0.8c_s$

$$m = \frac{S(w_s,n) + 0.8c_s f(n) - S_m}{-0.6}$$

$$b = 2.333S(w_s,n) + 1.867c_s f(n) - 1.333S_m$$

$$a = \frac{md_s}{c_s} + b$$

For $d_s \geq 1.4c_s$

$$a = S_m$$

where

a = average surface arterial speed (mph),
d_s = surface arterial volume per lane (vehicle/sec),
n = number of signals per mile,
w_s = surface arterial speed limit (mph),
c_s = surface arterial capacity per lane (vehicle/sec),
$S(w_s,n)$ = free-flow speed for a given speed limit and given number of signals per mile,
$f(n)$ = rate of speed change with volume, and
S_m = minimum speed (mph).

Modified Creighton, Hamburg freeway equations:

$$K_2 = \ln\left(\frac{3{,}600}{K_1 S_c} - \frac{3{,}600}{K_1 w_f}\right)$$

For $c_f > d_f$

$$S = \frac{3{,}600}{K_1 \exp\left(\dfrac{K_2 d_f}{c_f}\right) + \dfrac{3{,}600}{w_f}}$$

For $c_f < d_f$

$$S = \frac{3{,}600}{K_1 \exp\left[K_2\left(\dfrac{d_f}{c_f}\right)^{1/2}\right] + \dfrac{3{,}600}{w_f}}$$

where

S = average freeway speed (mph),
w_f = speed limit on freeways (mph),
c_f = freeway capacity per lane (vehicle/hr),
d_f = freeway volume per lane (vehicle/hr),
K_1 = 0.4,
K_2 = intermediate value, and
S_c = speed at capacity.

Peak-hour link volumes were estimated from simulated daily volumes through the use of a peak-hour percentage (by functional class and area type) taken from traffic counts collected within the region. Speed limits, signal densities, and free-flow speeds were input as a table lookup by functional class and area type. DVRPC travel time surveys have found that daily speeds are on average about 10 percent higher than peak-hour speeds. Recent travel time surveys seem to indicate that the modified Creighton, Hamburg curves given may underestimate actual speeds by about five percent, because drivers have become more acclimated to operating their vehicles under congested conditions (5). For this reason the speeds output by the curves were increased by up to five percent (subject to the minimum speed and speed limit) in the emissions computations that will be described.

TABLE 2 Effect of 50 Percent Across-the-Board Increase in Highway Traffic on Regional Average Vehicle Speed

	1987 Traffic Estimates	50% Traffic Increase	% Difference
Vehicle Miles of Travel (000's)	72,590.8	108,885.9	50.0
Vehicle Hours of Travel (000's)	3,393.8	6767.2	99.4
Average Speed (mph)	21.4	16.1	- 24.8

Table 2 presents regional travel and average speed statistics for the 1987 travel simulation and for a test run that assumed an across-the-board 50 percent increase in vehicular travel. On an average day, about 72,590,800 VMT occur on the highways included in the 1987 simulation network. This travel volume consumes about 58 percent of the theoretical daily capacity of the roadways, which results in 3,393,800 vehicle-hr for an overall average speed of 21.4 mph. A 50 percent increase in the traffic volume/capacity ratio reduces the average speed by about 25 percent to 16.1 mph. Although this VMT growth is arbitrary—some suburban and rural areas are likely to grow faster and urban areas to grow slower in a long-range horizon—it is interesting to see the effect of this increase and corresponding speed reduction on regional emissions. But first we must document the emission factor scenarios that will be used to test the postprocessor calculation method.

MOBILE4 EMISSION FACTOR SCENARIOS

Emission factors produced by the MOBILE4 computer program vary significantly depending on the settings of various policy and climatic options that are included to tailor the output to seasonal conditions and to state and local emissions control programs (6). Therefore, a thorough testing of the postprocessor methodology requires that a series of emissions factor scenarios be tested. This will ensure that findings apply to all MOBILE4 outputs that may be generated during the development of emissions reduction strategies. Generally, a base scenario reflective of summer conditions in the Delaware Valley was specified, and from this base selected MOBILE4 options were tested one at a time. Although the MOBILE4.0 model used herein is now superseded by MOBILE4.1, the range of variation in emission factors generated by the extreme parameter settings in these sensitivity tests far exceeds the differences between MOBILE4.0 and MOBILE4.1 in practical applications. Therefore, the conclusions reached in this analysis will still be valid for MOBILE4.1 and future versions of this emissions model.

Base Case Conditions

The base case is a generalization reflective of 1989–1990 summer emission factors in the Delaware Valley region. It is not possible to have one set of base factors for the entire air quality region, because inspection procedures, Stage II recovery systems, and so forth vary between the four constituent states. However, for testing purposes, we assume that the base and in-use Reid vapor pressure (RVP) of gasoline sold in this low-altitude region was 11.5 psi (although it was reduced to 9.0 during 1990) and that daily temperatures varied from 70 to 94°F. In addition, a computerized, decentralized inspection and maintenance program based on an ideal emissions test was assumed to be in effect for all vehicle types except heavy truck. This program was assumed to begin in 1984 with a 6 percent waver rate and a 93 percent compliance rate. Refueling emissions were assumed to be uncontrolled and the default (EPA-supplied) VMT accumulations by vehicle type and age were used.

Alternative Emissions Scenarios

Nine emissions scenarios were tested that cover the significant policy and climatic parameters of MOBILE4. The first two scenarios substituted RVPs of 15.2 and 7.0 for the 11.5 assumed under the base case. These are the maximum and minimum RVP values allowed in MOBILE4.0. The winter temperature scenario substituted a daily range of 20 to 45°F (VMT was held constant for this exercise, although on average, winter VMTs are thought to be about 5 percent lower than summer). The high-altitude scenario increased the altitude from 500 to 5,500 ft. The fifth scenario eliminated the inspection and maintenance program, and Scenario 6 substituted an antitampering program for inspection and maintenance. The heavy-truck scenario increased the combined heavy gas and diesel truck percentage from the EPA default of 4.9 percent to 25 percent. Twenty-five percent trucks is thought to be representative of the maximum value found on specific general-purpose public highways. The final scenario uses factors that reflect the emissions control devices that are anticipated to be installed in vehicles in calendar year 2020.

Table 3 displays the mean values and standard deviations of the MOBILE4 emission factors that resulted from this series of scenarios. Despite the significant reductions in emissions brought about by the catalytic converter and other vehicle-mounted pollution control devices, carbon monoxide (CO) is still the principal pollutant produced by vehicular travel. MOBILE4.0 factors at the average speed (21 mph) for CO range from 3 to 5 times the values for volatile hydrocarbons (HC), which in most cases are significantly higher than the factors for nitric oxides (NO_x). The large standard deviations with respect to vehicle speed associated with CO emissions factors indicate that the quantity of this pollutant produced varies significantly with speed, with larger factors generally being produced at low speeds, and with much smaller factors at higher speeds. HC emissions factors also vary with speed, but to a lesser degree, and NO_x factors have the least variation with vehicle operating speed and follow a U-shaped curve.

Changes in gasoline vapor pressure mainly affect HC emissions (by up to 70 percent), although CO emissions are also

TABLE 3 Sensitivity of MOBILE4.0 Emissions Factors to Selected Scenario Options

	<-- CO -->		<-- HC -->		<-- NOX -->	
Emissions Scenario	GM/Mile @ 21 mph[1]	Mean/Std Deviation[2]	GM/Mile @ 21 mph[1]	Mean/Std Deviation[2]	GM/Mile @ 21 mph[1]	Mean/Std Deviation[2]
Base Case[3]	24.8	28.6/27.4	5.5	5.8/2.2	2.1	2.2/0.3
RVP 15.2	27.3	31.4/29.9	9.5	9.8/2.3	2.1	2.2/0.3
RVP 7.0	20.7	24.1/23.3	2.9	3.2/2.0	2.1	2.2/0.3
Winter Temperature[4]	28.5	32.5/30.8	2.5	3.0/3.0	2.6	2.7/0.3
High Altitude	33.9	38.3/33.6	6.7	7.1/2.3	2.0	2.1/0.3
No Inspection & Maintenance	31.7	36.9/35.7	5.9	6.3/2.6	2.2	2.3/0.3
Substitute Anti-tampering Only	28.8	33.5/32.4	5.5	5.9/2.3	2.2	2.3/0.3
25% Heavy Truck[5]	26.4	30.1/26.9	5.3	5.6/2.1	4.7	5.0/0.8
Calendar Year 2020	14.4	13.1/ 7.7	3.4	3.6/1.2	1.3	1.3/0.3

[1]The mean simulated traffic speed from the 1987 network assignment was 21.4 mph.

[2]Mean and standard deviation over 53 emission factors calculated by whole mile per hour increment from 3 to 55 mph.

[3]Base case includes an RVP of 11.5 PSI, summer temperature (70-94° F), default vehicle mix (4.9% heavy truck), 1990 Calendar Year, and low altitude.

[4]20°-45° F.

[5]7.5% heavy duty gas truck and 17.5% heavy duty Diesel truck.

affected. Winter temperatures increase CO emissions by some 15 percent and NO_x by 24 percent but reduce HC emissions by more than 50 percent. The high-altitude option causes large increases in CO (37 percent) and HC emissions (22 percent) but reduces NO_x emissions slightly.

Inspection and maintenance programs tend to reduce all pollutants, with the biggest effect on CO (28 percent) and the smallest on NO_x (5 percent). The emissions improvement brought about by antitampering programs appears to be limited to CO and NO_x emissions (16 and 5 percent, respectively). A high concentration of heavy trucks more than doubles average NO_x emissions (123 percent) but tends to slightly reduce HC emissions (by 4 percent) and to increase CO emissions by just over 6 percent.

By 2020, great reductions in vehicle emissions rates are anticipated. CO emissions factors are to be cut by another 42 percent, and HC and NO_x emissions each by 38 percent. Significantly, the standard deviation with respect to speed is also to be reduced dramatically; therefore, much of this pollution relief will occur even if increased highway congestion significantly reduces prevailing highway vehicle operating speed over time.

As of this writing, EPA is still developing refinements to MOBILE4. However, taken together, the given emissions scenarios are adequate for postprocessor testing because they effectively cover the range of factors likely to be produced by MOBILE4 (and its successors) during emissions inventory development and in planning for air pollution reductions by state and local air quality planners.

POSTPROCESSOR VERSUS LINK-LEVEL EMISSIONS ESTIMATES

The postprocessor recalculated 5-km grid-level emissions based on gridded VMT and average speed stratified by highway functional class (freeway, arterial, and local/collector). These grid-level VMT and speed data were tabulated by the base processor from network link volumes and speeds. Off-network VMT is not included in either emissions calculation in order to focus the analysis on link volume/speed aggregation phenomenon. The inclusion of off-network VMT would reduce postprocessor errors because this VMT surcharge is allocated from county control totals and is therefore a constant by grid for both the link-level and postprocessor emissions calculators.

Base Case Scenarios

Table 4 shows that the errors in emissions attributable to postprocessor aggregation are minimal. Regional totals produced by the postprocessor are within 0.5 percent of the link-level–based estimates for all three pollutants. At the grid level, the percentage root mean squared (*RMS*) errors are less than 3 percent; the coefficient of determination (R^2) between emission estimates is essentially 1.0 (i.e., perfect colinearity). Theil tests indicate that almost all mean squared differences are attributable to scatter (UC) and that only minimal differences result from systematic variation in mean (UM) or standard deviation (US) (7).

TABLE 4 Stratified Postprocessor Error Statistics Under Base Case Emissions Scenario

Regional Total Emissions (Tons/Day)	— CO —	— HC —	— NOX —
Post-Processor	1722.6	392.7	159.1
Link Level	1716.4	392.7	159.6
% Difference	0.4	-	- 0.3
Grid Level Emissions Error Statistics	**— CO —**	**— HC —**	**— NOX —**
RMS Error (KG)	110.71	8.22	6.89
% RMS Error	2.78	0.90	1.86
Coef. of Determination (R^2)	0.9999	1.0000	0.9998
Theil Tests	**— CO —**	**— HC —**	**— NOX —**
UM	0.02	0.00	0.03
US	0.00	0.02	0.05
UC	0.98	0.97	0.92

These errors are insignificant for emissions reduction planning and are far less than the expected errors in the underlying emission factors or link-level VMT and speed estimates. Fundamentally, because of continuity of development patterns and connectivity in the highway system, the situation rarely occurs when one freeway or arterial in a 5-km grid is uncongested with high operating speeds while another of the same functional class in the same grid has low speeds resulting from severe congestion. Prevailing traffic conditions tend to apply to all highways of a given functional class within a grid.

Grid-level speed differences do occur by functional class, however, because of differences in access policies from abutting land uses and other design criteria. Freeways generally have higher speeds than arterials, which in turn generally operate at higher speeds than local streets. Table 5 compares link-level emissions with those produced by an alternative postprocessor that omitted the stratification by functional class. The omission of speed differences by functional class significantly increased the postprocessor errors in emissions. This is particularly true for NO_x, which is now underestimated by 2.3 percent at the regional level and has grid-level percentage *RMS* errors in excess of 5 percent. There are now significant Theil components for errors in mean and standard deviation. CO is also significantly affected by the loss of the functional class stratification, but to a somewhat lesser degree. HC is relatively unaffected by the simplification to the postprocessor.

TABLE 5 Postprocessor Error Statistics Without Disaggregation by Highway Functional Class

Regional Total Emissions (Tons/Day)

	— CO —	— HC —	— NOX —
Post-Processor	1728.0	392.8	155.8
Link Level	1716.4	392.7	159.6
% Difference	0.7	-	- 2.3

Grid Level Emissions Error Statistics

	— CO —	— HC —	— NOX —
RMS Error (KG)	172.00	11.28	- 19.98
% RMS Error	4.32	1.24	- 5.40
Coef. of Determination (R^2)	0.9994	0.9996	0.9995

Theil Tests

	— CO —	— HC —	— NOX —
UM	0.03	0.01	0.18
US	0.11	0.01	0.20
UC	0.86	0.98	0.62

This result is somewhat surprising given the small standard deviation with respect to speed in the NO_x emission factors (Table 3). But one should remember that it is emission factor differences in the prevailing range of highway operating speeds (15 to 35 mph), not overall variation, that count. CO emission factors do have a large standard deviation with respect to speed, but this variation largely results from geometric increases as speeds decrease below 8 or 10 mph. Very few roadways in the region operate at speeds this low.

To test the applicability of the stratified postprocessor in congested roadway conditions that may occur in a long-range forecast, all simulated link volumes were increased by 50 percent and the speed and emissions calculations reexecuted. Table 6 compares the postprocessor output (functional class stratified) with the corresponding link-level emissions estimates. Generally, the errors both at the regional and grid levels have increased only slightly over those reported in Table 4 for 1987 highway data. The stratified postprocessor methodology appears to remain applicable when analyzing long-range forecasts.

However, the effect of the reduced speeds under the 50 percent VMT increase on regional emissions is also of considerable interest. For the base case scenario, a 50 percent increase in travel increases regional CO emissions by 95.3 percent, HC emissions by 65.6 percent, and NO_x emissions by 55.2 percent. Although the 50 percent increase in travel is arbitrary for a long-range growth estimate, it is not out of line with the rapid traffic growth rates that occurred in the Delaware Valley region during the 1980s.

TABLE 6 Effect of 50 Percent Increase in VMT on Postprocessor Errors

Base Case Regional Total Emissions (Tons/Day)

	— CO —	— HC —	— NOX —
Post-Processor	3365.8	650.7	247.5
Link Level	3351.7	650.2	247.7
% Difference	0.4	-	- 0.1

Base Case Grid Level Emissions Error Statistics

	— CO —	— HC —	— NOX —
RMS Error (KG)	259.19	20.44	7.33
% RMS Error	3.33	1.36	1.28
Coef. of Determination (R^2)	0.9996	0.9999	0.9999

Percent Growth From 1987 VMT Estimate

Emissions Scenario	— CO —	— HC —	— NOX —
Base Case	95.3%	65.6%	55.2%
Year 2020	-5.9%	3.4%	-7.1%

Given increasing funding restraints on highway construction dollars, it is most unlikely that highway capacity can be increased by anything approaching 50 percent even in the long run. Improvements in vehicular emissions control technology must be relied on to prevent these geometric increases in vehicular pollution. Regional emissions totals calculated from the year 2020 scenario emissions factors (also in Table 6) indicate that anticipated additional vehicle emissions controls are just about adequate to handle this 50 percent increase in VMT. Despite traffic growth, CO and NO_x emissions decline by 5.9 percent and 7.1 percent, respectively, whereas HC emissions increase by 3.4 percent. The national pollution control program improvements in emissions control technology included in MOBILE4.0 appear to be able to absorb about a 50 percent increase in travel in calendar year 2020 without significant increases in mobile source emissions even if fuel volatility is not reduced.

Alternative Emissions Factor Scenarios

None of the emissions factor scenarios produced significant errors in emissions totals. Only the year 2020 scenario pro-

TABLE 7 Differences in Regional Emissions Totals (ton/day) by MOBILE4.0 Scenario Option

	<— CO —>			<— HC —>			<— NOX —>		
Scenario	Post Proc.	Link	% Diff.	Post Proc.	Link	% Diff.	Post Proc.	Link	% Diff.
Base Case	1,722.6	1,716.4	0.4	392.7	392.7	-	159.1	159.6	- 0.3
RVP 15.2	1,895.7	1,887.7	0.4	683.7	683.8	-	159.1	159.6	- 0.3
RVP 7.0	1,443.3	1,439.7	0.3	207.1	207.0	-	159.1	159.6	- 0.3
Winter Temp.	1,970.1	1,960.4	0.5	175.2	175.3	0.1	194.8	195.4	- 0.3
High Altitude	2,358.3	2,374.6	- 0.3	482.0	481.9	-	150.3	150.7	- 0.3
No Insp. & Maint.	2,209.6	2,203.3	0.3	426.0	426.0	-	162.3	162.8	- 0.3
Substitute Anti-tamp.	2,006.4	2,000.8	0.3	398.8	398.8	-	160.4	160.9	- 0.3
25% Heavy Truck	1,859.8	1,851.0	0.5	380.8	379.9	0.2	355.7	357.5	- 0.5
Calendar Year 2020	920.7	891.8	3.2	248.7	248.7	-	92.1	91.3	- 0.9

TABLE 8 Grid-Level Emissions Errors by MOBILE4.0 Scenario Option

	<— CO —>			<— HC —>			<— NOX —>		
Scenario	RMS Error (kg.)	% RMS Error	R^2	RMS Error (kg.)	% RMS Error	% R^2	RMS Error (kg.)	% RMS Error	R^2
Base Case	110.71	2.78	0.9999	8.22	0.90	1.0000	6.89	1.86	0.9998
RVP 15.2	122.45	2.80	0.9997	8.66	0.55	1.0000	6.86	1.85	0.9998
RVP 7.0	92.51	2.77	0.9997	7.49	1.56	0.9999	6.95	1.88	0.9998
Winter Temperature	128.66	2.83	0.9997	11.27	2.77	0.9997	8.13	1.79	0.9998
High Altitude	141.95	2.61	0.9998	8.97	0.80	1.0000	6.52	1.87	0.9998
No Insp. & Maint.	142.30	2.78	0.9997	9.87	1.00	1.0000	6.96	1.84	0.9998
Substitute Anti-tamp.	129.35	2.79	0.9997	8.73	0.94	1.0000	6.96	1.87	0.9998
25% Heavy Truck	121.27	2.82	0.9997	9.44	1.07	1.0000	23.98	2.89	0.9996
Calendar Year 2020	149.19	7.21	0.9996	4.41	0.77	1.0000	4.72	2.23	0.9999

duced errors noticeably larger than the base case, and this was limited to CO—3.2 percent regional overestimate with 7.21 percent *RMS* error by grid (see Tables 7 and 8). Otherwise, both regional and grid-level error statistics presented are without exception comparable to those from the base case scenario. Regional totals differ by 1 percent or less, and grid-level *RMS* errors, by 3 percent or less.

CONCLUSIONS

Clearly, postprocessor methodology is adequate to test all emissions factor reduction strategies that can be generated by MOBILE4. When the postprocessor VMT and speed inputs are stratified by functional class, the magnitude of error in emissions estimates as a result of link and speed aggregation, even at the grid level, is very small. However, aggregation errors increased substantially when the functional class strata were removed.

This does not mean that traffic simulation networks are not needed for preparing emissions inventories and planning for emissions reduction. Travel simulations are the most accurate and cost-effective way to estimate gridded VMT and speed. HPMS data bases contain far too few observations to be statistically reliable at the grid level. Even when supplemental traffic counts are available, the process of calculating VMT and allocating it to grids can be extremely tedious and labor-intensive. Generalized lookup tables of link volumes must still be used for most roadways because it is not feasible to take a current traffic count on each of the many thousands of highway links that exist in an urban area. Travel simulation models produce volumes for all links in the network in a systematic way. Current simulated volumes are subject to screenline validation with traffic counts. Simulated forecasted volumes quantify the impact of projected land use changes and new highway facilities in a consistent fashion.

However, for a given network assignment, using the postprocessor can expedite and streamline the calculation of emissions with little loss of accuracy or consistency. This capability is very useful when conducting sensitivity tests that are needed for refining and customizing the MOBILE4 parameters and options to be used for inventory development and for testing possible emissions reduction strategies.

REFERENCES

1. *Procedures for Emission Inventory Preparation, Volume IV: Mobile Sources*. Office of Air Quality Planning and Standards, Environmental Protection Agency, Research Triangle Park, N.C., Dec. 1988.
2. *1987 Travel Simulation for the Delaware Valley Region*. Delaware Valley Regional Planning Commission, Philadelphia, Pa., May 1988.
3. *Conformity of the Regional Transportation Plan and Program*. Delaware Valley Regional Planning Commission, Philadelphia, Pa., Sept. 1991.
4. Creighton, Hamburg, Inc. *Freeway-Surface Arterial VMT Splitter*. FHWA, U.S. Department of Transportation, 1971, pp. 22–24.
5. *1971–73 to 1986 Highway Travel Time and Speed Comparisons in the New Jersey Portion of the DVRPC Region*. Transportation Planning Data Bulletin 9, Delaware Valley Regional Planning Commission, Philadelphia, Pa., Sept. 1987.
6. *Users Guide to MOBILE4*. Office of Air and Radiation, Environmental Protection Agency, Ann Harbor, Mich., Feb. 1989.
7. H. Theil. *Applied Economic Forecasting*. Rand McNally, Chicago, Ill., 1966, pp. 29–36.

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Toll Plaza Design To Minimize Carbon Monoxide Levels at Roadway Rights-of-Way

ALICE LOVEGROVE AND STEVEN WOLF

Compliance with national and state ambient air quality standards is an important consideration in designing a toll plaza. With the increased focus on air quality created by the enactment of the Clean Air Act Amendments of 1990, air quality concerns must now be regarded along with other design criteria. Meeting ambient air quality standards can often require the acquisition of additional right-of-way (ROW) around the toll plaza. This approach, which increases the distance between source and receptor, is often impractical in terms of land use and cost. Some toll plaza design concepts have been developed at existing and proposed toll plazas to achieve lower pollutant concentrations while minimizing ROW requirements. By designing the plaza to eliminate the overlap of high-emission zones and by introducing automatic vehicle identification toll gates, the ROW needed to avoid an air quality violation can be greatly reduced.

With the enactment of the new Clean Air Act Amendments of 1990, air quality concerns must be regarded along with other highway design criteria. In designing a toll plaza, compliance with national and state ambient air quality standards is an important consideration. Meeting these standards may require additional right-of-way (ROW) to maintain an adequate distance between the toll plaza and either existing or planned sensitive land uses. This study discusses toll plaza design concepts that have been developed to achieve lower air pollutant concentrations while minimizing required ROW acquisition.

These concepts are applicable to existing roadways with limited ROW (i.e., near sensitive land uses) and to new roadways for which the land acquisition costs for ROW are very high.

BACKGROUND OF CLEAN AIR ACT

The Clean Air Act Amendments of 1990 direct the Environmental Protection Agency (EPA) to implement strong environmental policies and regulations that will ensure cleaner air quality. These amendments will affect all proposed transportation projects. According to Title I, Section 101, Paragraph F, "No federal agency may approve, accept or fund any transportation plan, program or project unless such plan, program or project has been found to conform to any applicable (state) implementation plan in effect under this act." Title I of the amendments defines conformity as follows:

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- Conforming to an implementation plan's purpose of eliminating or reducing the severity and number of violations of the National Ambient Air Quality Standards (NAAQSs) and achieving expeditious attainment of such standards; and
- Ensuring that such activities will not
  - Cause or contribute to any new violation of any NAAQS in any area,
  - Increase the frequency or severity of any existing violation of any NAAQS in any area, or
  - Delay timely attainment of any NAAQS or any required interim emissions reductions or other milestones in any area.

CARBON MONOXIDE: HEALTH IMPLICATIONS AND STANDARDS

Toll plazas have traditionally been known as areas with high carbon monoxide (CO) levels. CO is the pollutant of concern for projects involving congested roadways. High CO levels around toll plazas are caused by queueing at and accelerating from the plaza.

CO is a colorless and odorless gas. It is a localized problem; high concentrations are normally limited to locations within a relatively short distance (300 to 600 ft) of a heavily traveled roadway. Because of the increased source strength at a toll plaza, this distance can increase to 1,000 ft.

Exposure to CO can lead to serious health risks. CO is a relatively insoluble compound that easily enters the tiny air sacs, or alveoli, in the lungs along with oxygen. Once in the lungs, CO diffuses through the alveolar walls, entering the bloodstream. In the bloodstream, CO joins with hemoglobin to form carboxyhemoglobin. Hemoglobin normally bonds with oxygen, but it has an affinity for CO that is about 210 times greater. This bonding causes a decrease in the oxygen supply in the blood. As shown in Figure 1, this can have serious health implications. On the basis of this information, EPA established NAAQSs for CO and several other pollutants. The standards for CO are 9 ppm for 8 hr and 35 ppm for 1 hr. These standards are levels to which pollutant concentrations should be reduced to avoid undesirable effects. The primary goal of these standards is to protect public health. The secondary goal is to protect the nation's welfare and account for the effects of air pollution on soil, water, vegetation, and other aspects of general welfare.

This study will use the 8-hr standard to determine the minimum ROW necessary for each configuration to avoid surpassing the regulated levels.

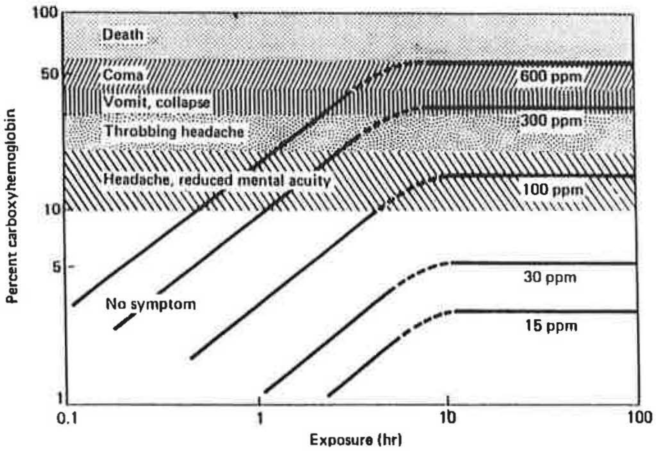

FIGURE 1 Effects of exposure to CO on man.

CO LEVELS AT TOLL PLAZAS

Emission of CO from a vehicle in grams per mile is very high at lower speeds and idling. CO emission also greatly increases during acceleration. All these vehicular operating conditions are present at toll plazas. Because of this, CO levels near toll plazas are often extremely high. To avoid air quality violations, ROW must often be increased. This places sensitive receptor sites beyond the area in which CO levels would exceed the standard. For all practical purposes, this approach is a condemnation of the land; it is also inefficient in both cost and land use. A better way to achieve compliance with the air quality standards is to have the air quality and civil engineers work together to design a toll facility that meets standard toll requirements and minimizes CO levels.

METHODOLOGY

Four toll configurations were analyzed. Each configuration was divided into transition and queueing zones, as shown in Figure 2. The high-emission zones shown in this figure are the queue zone and the acceleration zone. Each toll design is based on the location of these areas.

For each toll configuration, a minimum ROW distance needed to avoid an air quality violation was determined. The minimum ROW required is defined as the distance between a receptor and roadway needed to achieve a predicted CO concentration below the 8-hr standard of 9 ppm. The ROW distances required for each toll plaza design were compared to the ROW requirements without the toll plaza. Each toll plaza design alternative was then ranked in terms of the ROW required to meet air quality standards.

The analysis of the air quality impact of each toll configuration used emission and dispersion modeling programs. Pollutant dispersion analysis uses a line source air quality model that simulates the physical conditions in the study area. It is based on the assumption that the dispersion of pollutants follows a Gaussian, or normal, distribution. The model used for this analysis was CALINE4. A complete explanation of the mathematical formulation, basic assumptions, and limitations of the model are given elsewhere (*1*).

Parameters for the air quality analysis are presented in Table 1. The analysis considered wind directions ranging from 0 to 360 degrees to determine the worst-case level, which was calculated for each receptor. These values, used for all scenarios, were based on design conditions used to evaluate major highway projects in California. Emission factors used were determined through the California emissions program EMFAC7C. This program, using vehicle mix, temperature, California registration and fleet information, and other parameters, determines an average emission factor for a vehicle traveling at a specified speed. Although this information is specific for California, the resulting trends can be applied nationwide.

TABLE 1 Parameters for Toll Analysis

Parameter	Value
Volume -	
Peak Direction	3850
Off Peak Direction	2300
Free Flow Speed	55 mph
AVI Speed through gate	30 mph
Number of Manual Lanes	
No AVI usage	10
With AVI usage	5
Percent of AVI usage	50%
Number of AVI Lanes	5
Average Queue (# of vehicles/gate)	10
Wind Speed	.5 m/s
Stability Class	7
Surface Roughness	100 cm
Ambient Air Temperature	50 F
Persistence Factor	0.7
Vehicle Mix	
Autos	75.00%
Light Duty Trucks	19.00%
Medium Duty Trucks	4.20%
Heavy Duty Trucks	1.80%
Vehicle Operating Mode	
Hot Start	2.00%
Cold Start	10.00%

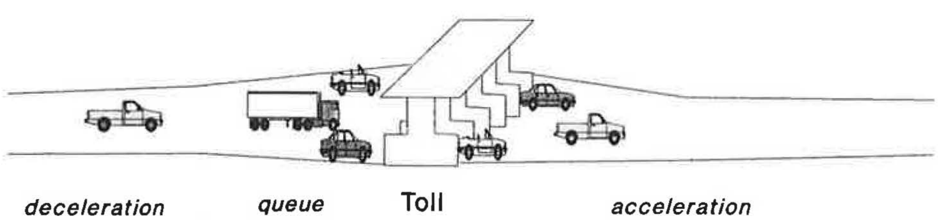

FIGURE 2 Transition and queueing zones.

Emission strength and the distances at which they were applied were determined on the basis of the CALINE4 dispersion model for predicting air quality pollutant concentrations near roadways. The actual emission strength values will vary depending on the defined roadway configuration and use. For this analysis, using the parameters in Table 1, the emission zones ranked from highest to lowest strength were the queue zone, 30- to 55-mph acceleration zone, 0- to 30-mph acceleration zone, and the deceleration zone.

The emissions for the acceleration zones were calculated using the following equations from the CALINE4 model (*1*):

$$\text{EFA} = \text{BAG2} * 0.76 * e^{0.045 * \text{AS}} \quad \text{(at rest)} \tag{1}$$

$$\text{EFA} = \text{BAG2} * 0.027 * e^{0.098 * \text{AS}} \quad \text{(moving)} \tag{2}$$

Average acceleration rates were taken from the CALINE4 mode surveillance driving sequence data. The modal acceleration speed product (AS) was determined to be 30.3 m^2/hr^2-sec for the 0- to 30-mph acceleration zone (Equation 1) and 61.2 m^2/hr^2-sec for the 30- to 55-mph acceleration zone (Equation 2). Both of these values were within the acceptable ranges of the equations.

The total strength of these zones is determined by the combination of source strength and area in which they apply. The distance over which acceleration takes place is greater than the distances needed for queueing. The combination of distance and source strength makes the acceleration zones the areas of most concern for this analysis.

The toll configurations were also analyzed with the introduction of automatic vehicle identification (AVI). AVI is a system used for identifying vehicles passing a specific point. Other expressions for this system include electronic toll and traffic management and electronic toll collection. This system allows vehicles to pass a point—in this case, a toll collection facility—and be identified and charged for passage, all without stopping. The system may be optical or electronic. The driver would receive a monthly bill or would have the toll amount deducted from an account each time the registered vehicle passed the collection point. This system can reduce pollutant emissions resulting from vehicles using this system since it will eliminate the queue and reduce the amount of acceleration emissions.

ALTERNATIVE TOLL PLAZA DESIGNS

The first configuration is the traditional straight line design (Concept 1) shown in Figure 3. The second (Concept 2), shown in Figure 4, is a staggered design in which acceleration zones overlap. The third (Concept 3), shown in Figure 5, is a staggered design with no overlapping zones. Concept 4, shown in Figure 6, is based on Concept 3; it has separated manual and automatic toll gates.

Analysis

The first part of the analysis determines the ROW requirements for each toll configuration, and the second part considers the effects caused by the use of AVI to each of the four designs. The pay-in-one direction toll concept was not analyzed in this study. This concept will generally reduce ROW requirements, but care must be taken when it is analyzed because of the potential change in traffic patterns that may occur if a toll-free route is available nearby. Because of this, the concept was not studied.

Although the distances shown in the analysis are specific for these examples, the trends illustrated can be applied to similar scenarios.

Concept 1: Traditional Design

Traditionally, toll plazas have been designed as straight strings of gates crossing both directions of traffic (Figure 3). As shown in this figure, the queue zone and the opposing acceleration zone are parallel. These two zones generally emit the highest

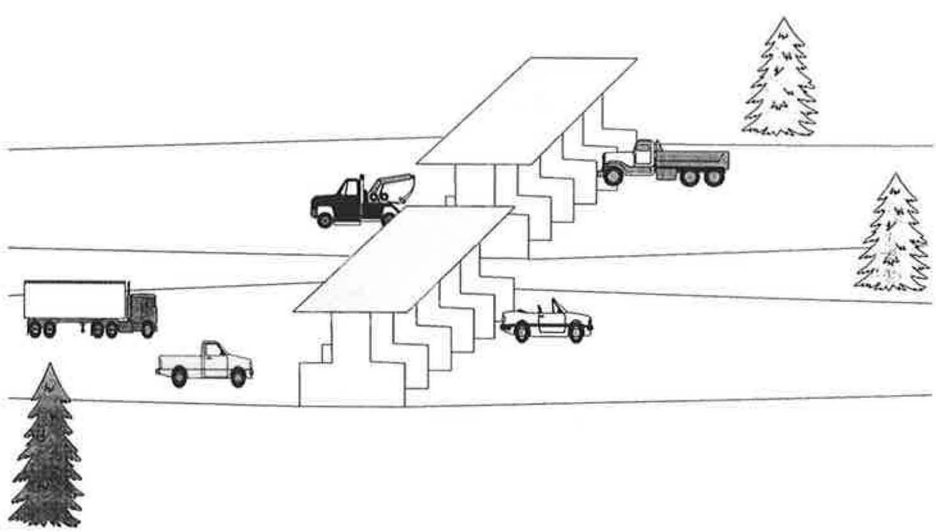

FIGURE 3 Concept 1.

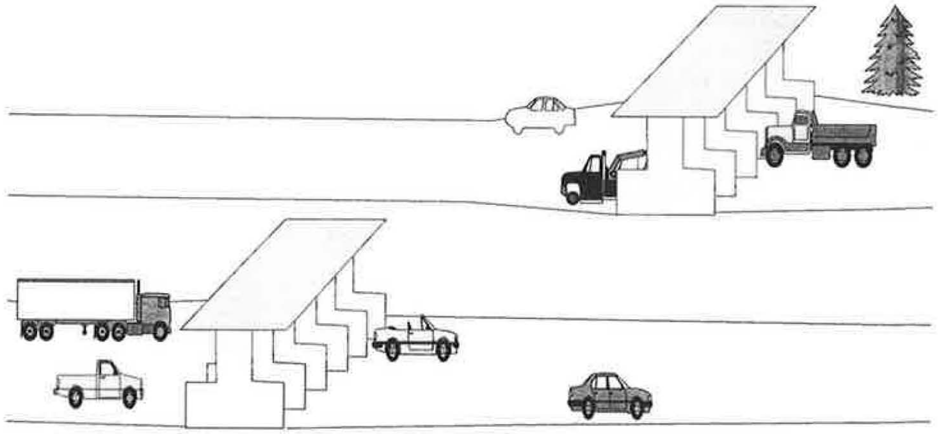

FIGURE 4 Concept 2.

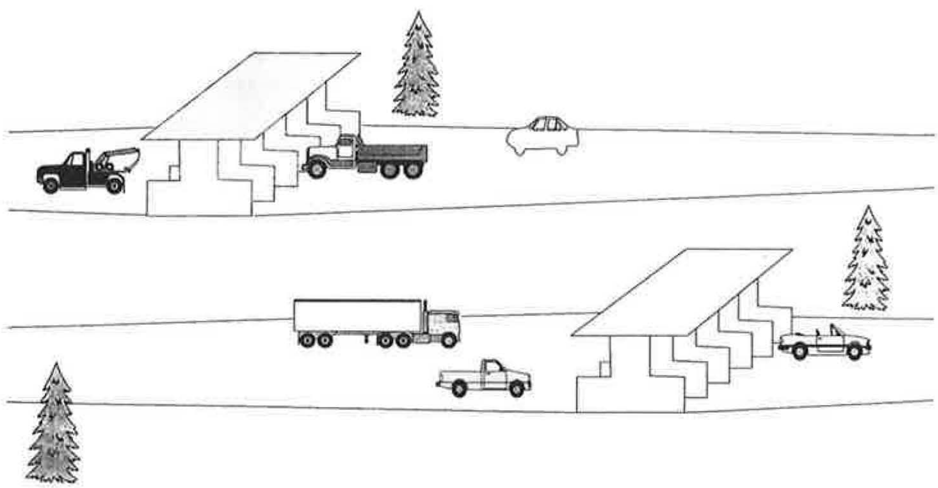

FIGURE 5 Concept 3.

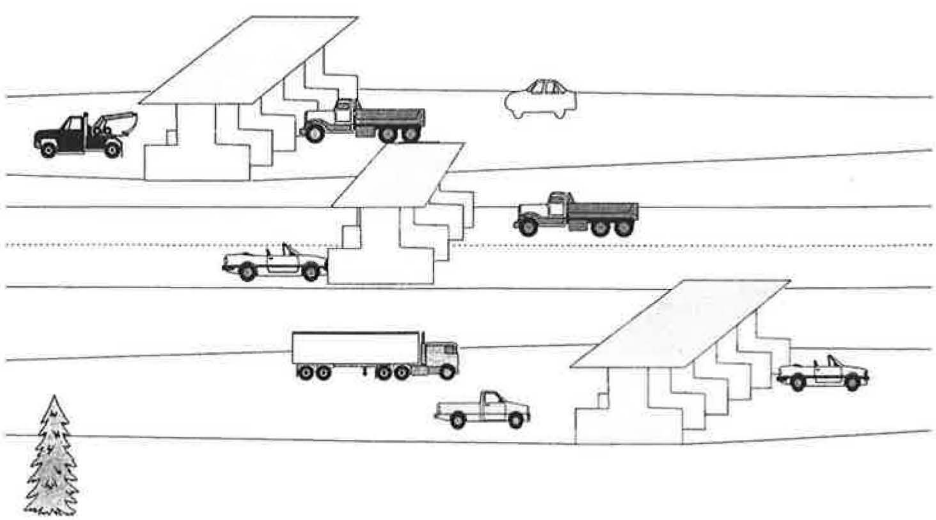

FIGURE 6 Concept 4.

amounts of CO. These zones also combine to create a band of high CO concentrations. This increased source strength will require more space to dissipate, thus increasing the ROW beyond the area right next to the source.

As shown in Figure 7, a ROW to avoid a violation of the 8-hr CO standard under this concept would require roughly 750,000 ft^2, with a maximum distance of 510 ft from the edge of the roadway. The ROW maximum distance without the toll would be only 20 to 30 ft. For an existing roadway this type of land requirement may be impossible to attain. In such a situation, one way to reduce the ROW would be to introduce AVI. Figure 8 shows the ROW with the introduction of AVI technology. It was assumed that 50 percent of the vehicles entering the plaza area used AVI. The required ROW to avoid a violation of the 8-hr CO standard is about 680,000 ft^2 of land, with a maximum distance of 490 ft from the edge of the roadway. This is more than a 9 percent reduction in land requirement when compared with no AVI usage.

Concept 2: Staggered, Acceleration Overlap

The two directions can also be staggered, as shown in Figure 4. This design creates an overlap of each direction's acceleration zone. The acceleration zone is an area of high emissions and thus creates an area of very high pollutant concen-

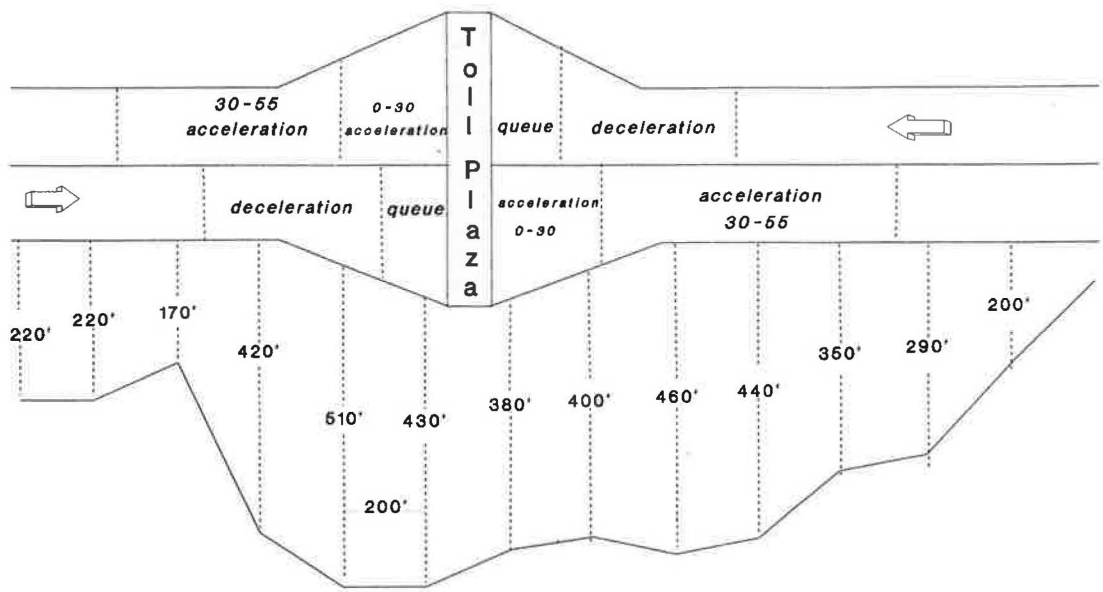

FIGURE 7 ROW for Concept 1.

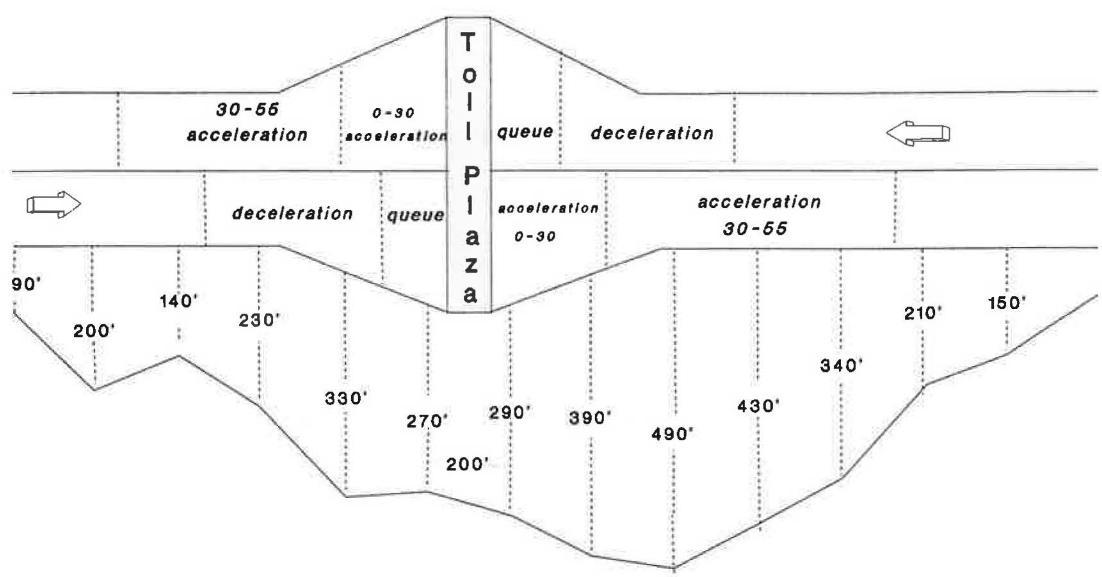

FIGURE 8 ROW for Concept 1 with AVI.

tration. This increased concentration takes more space to dissipate, increasing the ROW around the overlap area (Figure 9). Although this area is greater than the area needed for Concept 1 in this zone, the ROW for Concept 2 returns to its free-flow width in a shorter distance. Thus Concept 2 could be used in areas where a large ROW is attainable within a short span of roadway.

Figure 10 illustrates the ROW using Concept 2 with AVI technology. The required ROW distances were not reduced using this method. This is due to the continuing contribution of the acceleration zone from the AVI vehicles. The AVI vehicles were assumed to decelerate from 55 to 30 mph and then accelerate from 30 to 55 mph. The acceleration of the AVI vehicles combines with the opposing traffic flow's queue

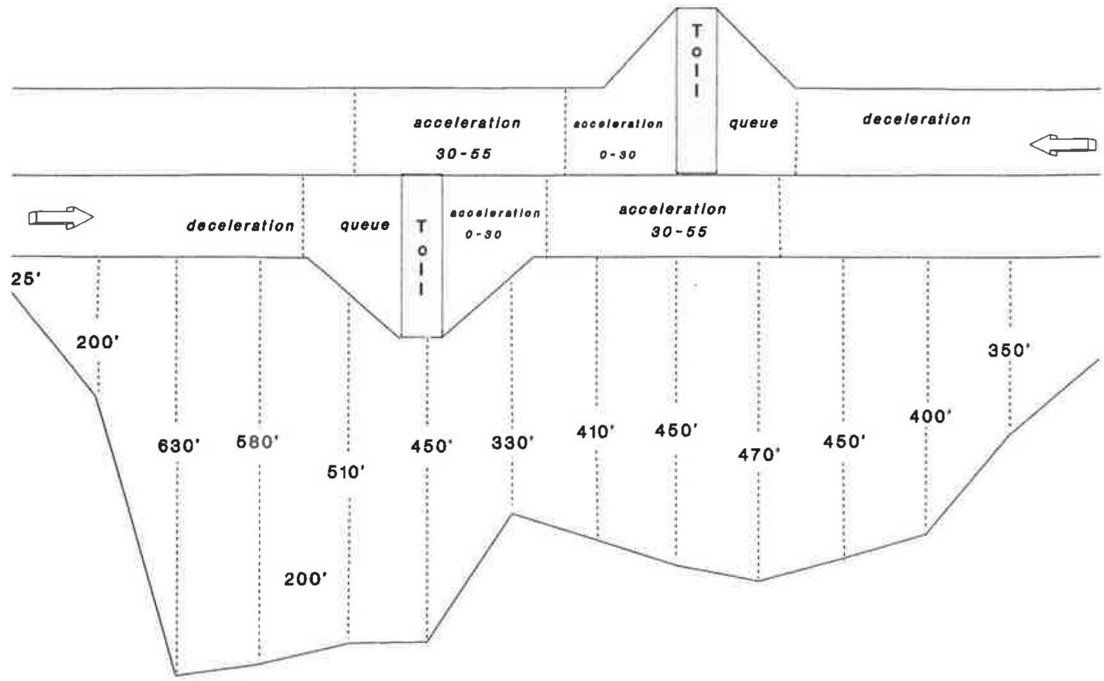

FIGURE 9 ROW for Concept 2.

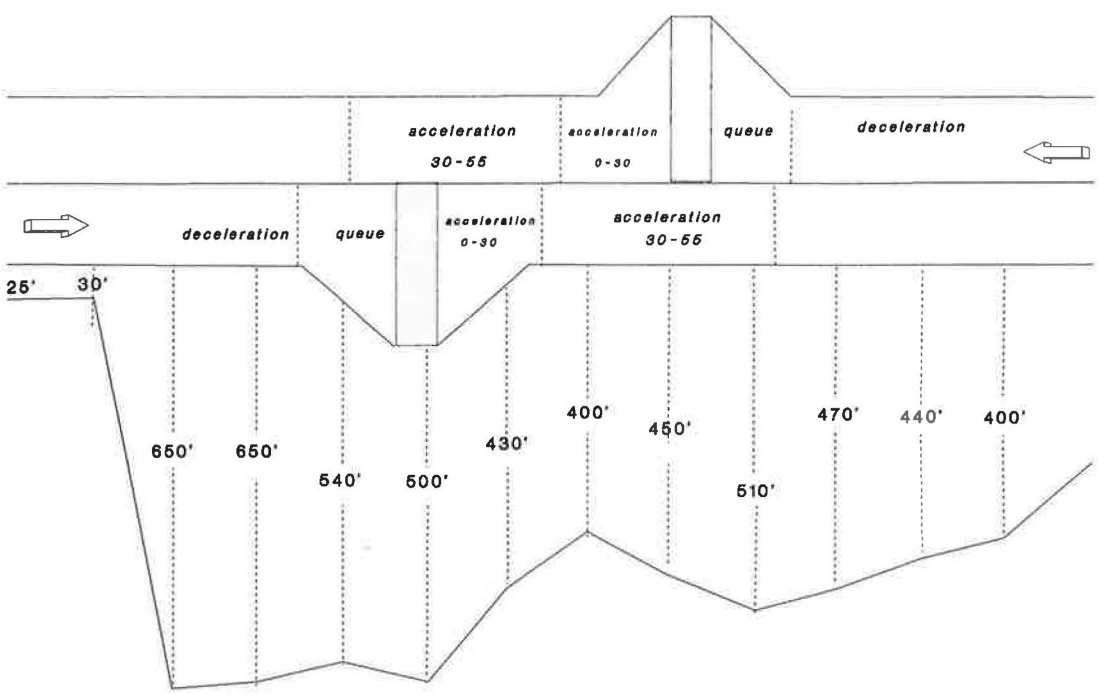

FIGURE 10 ROW for Concept 2 with AVI.

to create a very high emission source—higher than the zone created with no AVI usage. This leads to slightly higher ROW requirements.

This design concept exaggerates the required ROW zone by overlapping each direction's acceleration zones, so no significant improvement is gained by using this concept and AVI.

Concept 3: Staggered, No Overlap

Figure 5 illustrates the staggered design introduced in Concept 2, but here the acceleration zones are separated so that they do not overlap. This design greatly reduces the necessary ROW needed to avoid an air quality violation. By using this design, the high-emission acceleration zones are separated. This design leads to a peak ROW distance of 390 ft and a total area requirement of approximately 635,000 ft^2 (Figure 11). This is a 15 percent savings of land acquisition when compared with Concept 1 without AVI. The introduction of AVI technology to this concept reduces the ROW even more, resulting in an area requirement of approximately 500,000 ft^2. This is a 26 percent decrease in land requirement using no AVI (Figure 12). Concept 3 with AVI also results in a 33 percent decrease in land requirements compared with Concept 1 and a 26 percent decrease compared with Concept 1 with AVI. This design results in the smallest ROW requirement when compared with Concepts 1 and 2.

Concept 4: Concept 3 Design with Lane Separation

Concept 4 uses the principles developed in Concept 3 and adds lane separation. As shown in Figure 6, Concept 4 staggers the traditional toll gates so that there is no overlap of the acceleration zones. It also separates the AVI tolls from the manual tolls. As shown in Figure 13, this design reduces

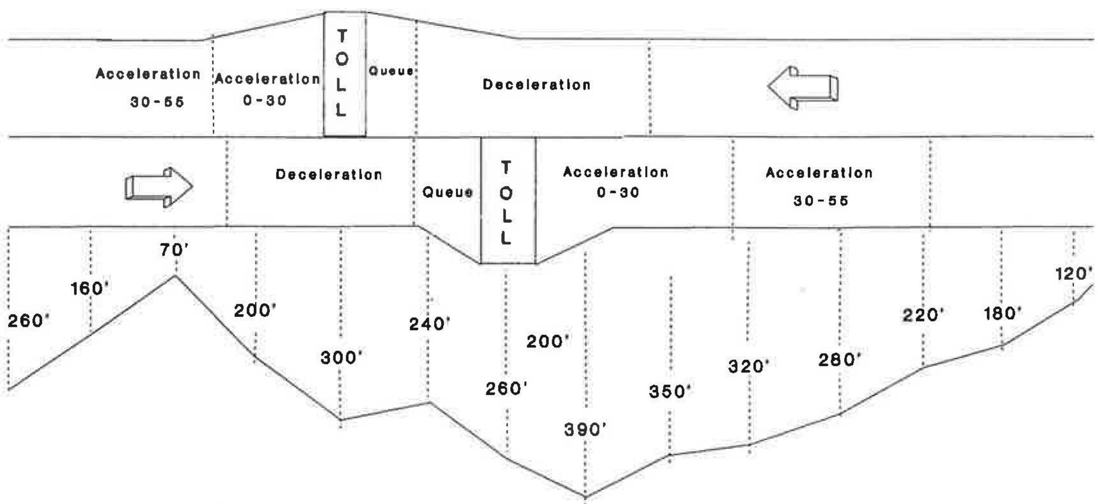

FIGURE 11 ROW for Concept 3.

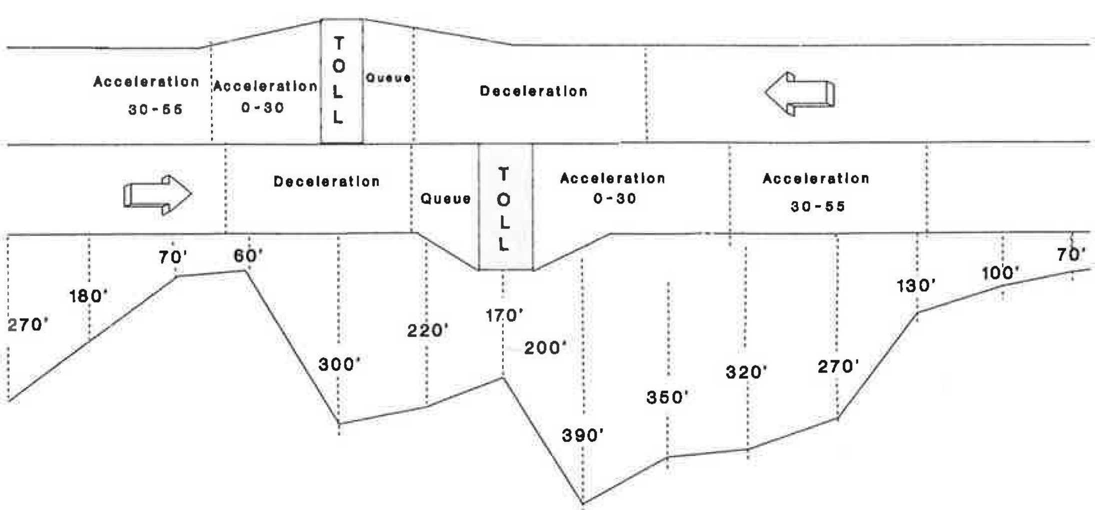

FIGURE 12 ROW for Concept 3 with AVI.

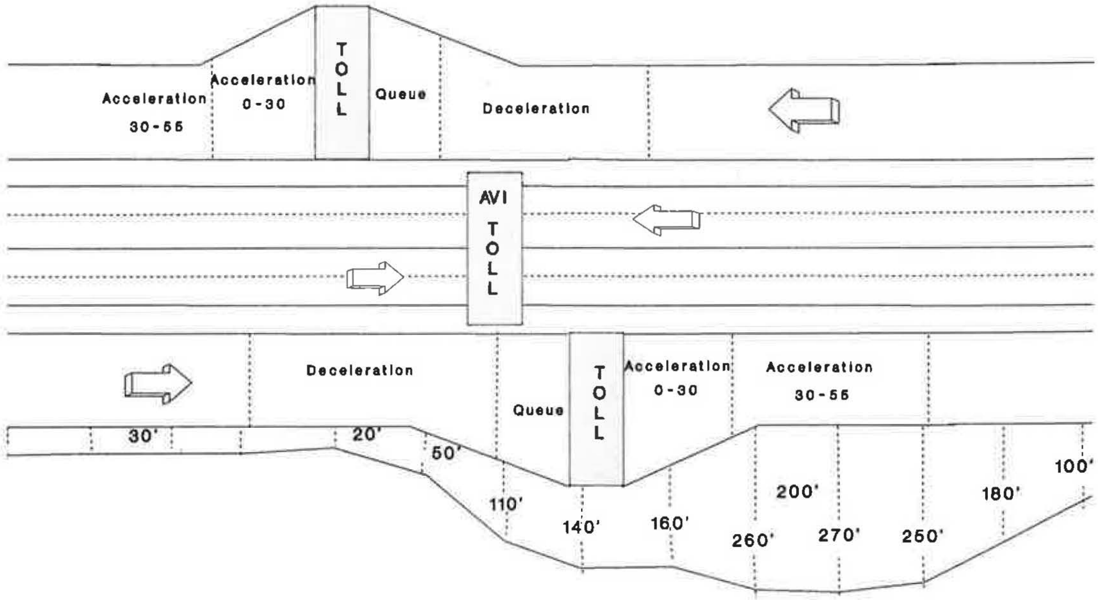

FIGURE 13 ROW for Concept 4.

the ROW requirements to about 331,000 ft^2—which reduces land requirements by more than 33 percent when compared with Concept 3 using AVI. This design, however, requires more land for the roadway design. It would best be applied to a wide roadway on which the lanes could be easily separated.

CONCLUSION

The trends illustrated by the toll designs in this analysis all have specific applications. The choice of the design will be of paramount importance in the reduction of ROW. Existing roadways that are introducing tolls could minimize the ROW requirement due to air quality considerations by used AVI as illustrated with Concept 1. Concept 2 could be used along a roadway that has available ROW in a limited location. Thus this design may be useful in very specific situations. Concept 3 requires a smaller ROW than the previous concepts and appears to be the most applicable to a large number of roadway designs. The introduction of AVI to this concept further reduces the needed ROW, increasing the concept's feasibility. Concept 4 requires an even smaller ROW than Concept 3. It, however, requires more land for the actual construction of the roadway. This design could be applied to roadways on which there are more than three lanes or on which the size of the median would provide additional space.

The addition of a toll facility, regardless of design, will most likely increase CO levels on both a micro (local) and meso (regional) scale level. Steps should be taken to minimize this impact. Air quality considerations are usually taken into account after the toll plazas have been designed. This often leads to costly mitigation and redesigns. By having the air quality and civil engineers work together at the initial design stage, the best design that satisfies both groups can be developed.

This analysis highlights some of the design elements that can be used to allow for a functional toll plaza design that meets air quality standards using the least amount of ROW.

REFERENCES

1. P. Benson. *CALINE4—A Dispersion Model for Predicting Air Pollutant Concentrations Near Roadways.* FHWA/CSA/TL-84/15. Office of Transportation Laboratory, California Department of Transportation, Sacramento, Nov. 1984.
2. P. C. Randall and H. N. C. Ng. *Air Quality Analysis Tools (AQAT-2).* Technical Support Division, State of California Air Resources Board, Sacramento, 1987.
3. J. Seinfeld. *Atmospheric Chemistry and Physics of Air Pollution.* John Wiley & Sons, New York, N.Y., 1986.

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Improving Average Travel Speeds Estimated by Planning Models

RICHARD DOWLING AND ALEXANDER SKABARDONIS

Modeling the impact of air quality requires estimating vehicle volumes and average travel speeds in order to estimate air pollutant emissions. Relatively accurate estimates of average vehicle travel speeds can be obtained from existing traffic operations models. However, when the travel demand impacts of the highway project extend over many miles of the freeway and arterial street network, it is not feasible to apply these detailed traffic operations models to the entire area of impact because of their extensive data requirements. Planners then must resort to areawide planning models to forecast vehicle volumes and average travel speeds. These traditional planning models, however, are not calibrated to produce accurate speed estimates. Free-flow speeds and a speed-flow curve are input into those models and adjusted as necessary to obtain calibrated volume estimates. Typically, the reasonableness of the final travel speeds is not checked once reliable volume forecasts have been achieved. A postprocessor methodology that can be applied at the end of a typical planning model forecast process to improve the estimates of travel speeds output by a planning model is proposed. The methodology uses an improved speed-flow curve and queueing analysis to obtain travel speed estimates that more closely approximate the average speeds estimated by typical operations models. The proposed methodology was applied to a real-life highway network and the results compared to FREQ and TRANSYT-7F simulations for a 5-mi section of freeway and a 3-mi section of a four-lane divided arterial street. The postprocessor significantly improved the original planning model estimates of average speed and delay.

The research herein came about as the direct result of new more stringent guidelines set by the San Francisco Metropolitan Transportation Commission (MTC) for evaluating the air quality impacts of new highway projects. These guidelines, developed in response to the Federal Clean Air Act, require project sponsors to demonstrate that each highway project to be added to the regional Transportation Improvement Program does not in and of itself increase air pollutant emissions for two critical pollutants: carbon monoxide and reactive organic gases. Before this, air quality impact analyses had focused on determining localized exceedances of ambient air quality standards rather than increases in pollutant emission burden. Now, project sponsors had to demonstrate not only that there would be no increases in localized exceedances but also that there would be no net increase in regional pollutant emissions either.

Detailed traffic operational analyses that had been sufficient if they focused on the proposed facility and a few parallel streets were no longer sufficient because any capacity improvement would naturally draw more traffic to the corridor and show a net increase in pollutant emissions within the corridor. It was now necessary to expand the analysis to include areawide coverage so that the reduction in travel in other corridors could be documented and included in the pollutant burden analysis. This requirement for broad coverage exceeds the capabilities of typical traffic operations models such as FREQ (*1*) and TRANSYT-7F (*2*). Thus, areawide planning models must be used, yet these large-scale planning models do not typically provide reliable estimates of operational speeds on a facility. The increased focus on the accuracy of the estimates of air pollutant emissions requires that these planning models be more accurate.

The proposed postprocessor methodology described in this paper was developed in response to these requirements for increased depth and breadth in the analysis of air quality impacts. The methodology seeks to obtain speed and congestion forecasts from areawide planning models that are closer to those that would be obtained if a more accurate operational model such as FREQ or TRANSYT-7F could be applied to each individual facility in a large study area.

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CRITIQUE OF CURRENT PLANNING MODELS' SPEED ESTIMATES

Planning models typically use a speed-flow curve such as the BPR curve to estimate the congested travel speed given the initial free-flow speed and the volume/capacity ratio (v/c) (*3*,p. 35). The standard equation for the BPR curve is

$$\text{congested speed} = \frac{\text{free-flow speed}}{(1 + 0.15 * v/c^4)} \qquad (1)$$

As can be seen in Figure 1, the BPR curve generally overestimates actual average travel speeds for a freeway under all conditions [compared with the standard *1985 Highway Capacity Manual* (HCM) (*4*) speed-flow curve for an eight-lane freeway with a design speed of 70-mph].

The error in speed estimation is greatest at v/c ratios approaching 1.00, which is also where air pollutant emission rates are most sensitive to estimates of the average speed. A difference of 20 mph in the speed estimate (30 mph versus 50 mph at v/c ratios of 1.00) can double the emission rate.

It is also fairly common in planning models to have estimated demands in excess of the capacity of individual facilities. However, there are no observed speed-flow curves for v/c ratios in excess of 1.00. Consequently, the BPR curve (or some similar curve) is applied in situations in which volumes exceed the facility's capacity.

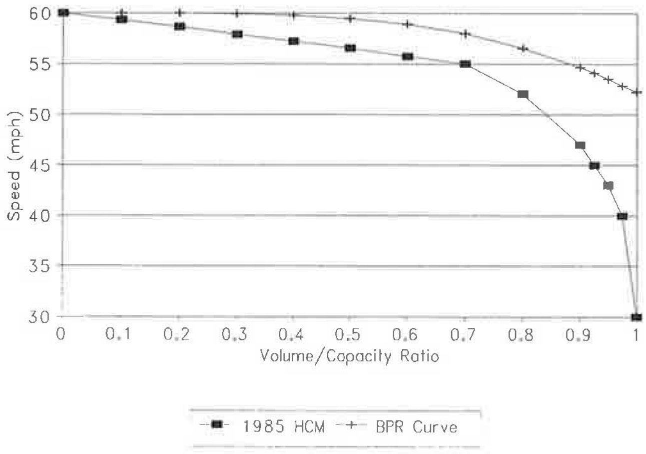

FIGURE 1 HCM and BCR speed-flow curves.

The BPR curve does not account for the effects of queueing on travel speeds and demand. Planning models consequently will significantly overestimate speeds of facilities near, at, or over capacity.

PROPOSED APPROACH

The discussion has identified two basic problems with the speed estimation procedures in typical planning models:

1. The standard BPR speed-flow curve overpredicts congested travel speeds, and
2. The BPR curve does not accurately represent speed-flow conditions when queueing is present.

The proposed methodology for improving planning model speed estimates therefore would replace the standard BPR speed-flow curve with an improved curve based on actual field observations, and it would add queueing analysis techniques to the calculation of travel speeds for situations in which the model-predicted volumes exceed capacity.

It would be most desirable to input the corrected speed-flow relationship directly into the planning model, but many software packages allow the user to specify only minor variations on the BPR curve when equilibrium assignment is being used (software packages typically allow a great deal of latitude in specifying the speed-flow curve, except when equilibrium assignment is used). For example, users of the MINUTP software package can change the coefficient but not the power of the BPR curve (*5*). It is thus not always possible to use a completely new speed-flow curve in the actual assignment process of many planning models.

Planning models are typically calibrated against traffic counts. This calibration process must compensate (at least partly) for the inaccuracies typically contained in the speed-flow curve. If the model's traffic volume forecasts are presumed to be reliable, then as a "second choice" we can choose to accept the model's traffic forecasts and merely correct the resulting speed estimates output by the model for congested situations. These speed estimates can be corrected not only by using a better speed-flow curve but also by considering queueing that arises in over-capacity situations.

It is consequently proposed that the improved speed-flow curves discussed be applied after the traffic assignment stage to improve the estimate of actual average travel speeds on the presumption that the planning model's volume forecasts are accurate (even if they used a different speed-flow curve).

Because the HCM speed-flow curve does not deal with v/c ratios in excess of 1.00 (conditions that can't occur in real life, but that can occur in planning models), another method involving queueing analysis must be used to estimate speeds for v/c's in excess of 1.00. Queues would be estimated for the peak period by dividing the peak period into hour-long intervals and performing a queueing analysis for each 1-hr time slice. The average speed over all 1-hr time slices is then calculated for the peak period.

The resulting planning model postprocessor methodology was tested against simulation results using traffic operations models for freeway and arterial street sections. The resulting speed and delay estimates were then compared to determine the effect of the postprocessor on planning model speed estimates.

PROPOSED SPEED-FLOW CURVE

Several investigators have tried different speed-flow curves to improve speed estimates and the estimated traffic volumes output by planning models. Most of these efforts have focused on changing the parameters of the the BPR curve while retaining the basic form of the equation. Different equations have been developed for freeway and arterial street facilities.

Speed-Flow Curve for Freeways

Several researchers have shown that the idealized freeway speed-flow curve in the HCM may not accurately reflect actual freeway operational conditions (*6*). The proposed postprocessor methodology is flexible enough to allow the researcher to input any desired speed-flow curve, as long as it can be specified in an equation using function available in most planning model software packages. However, for purposes of illustrating the approach, we will confine ourselves to the basic BPR equation and modify the parameters to best fit the freeway speed-flow curve in Figure 3-4 of the HCM. It does not particularly matter which curve (the HCM curve or another curve based on local data) is assumed to best represent actual operational conditions. The intent here is to demonstrate the improvements in estimated speeds that can result when a superior speed-flow curve is used to estimate speeds.

Figure 2 shows the results for two curves visually fitted to the HCM curve. The first curve (labeled "Mod.BPR4" in the figure) is

$$\text{congested speed} = (\text{free-flow speed})/[1 + (v/c)^4] \qquad (2)$$

This curve changes only the coefficient for the curve from 0.15 to 1.00. This change forces the BPR curve to drop more rapidly to 30 mph at v/c of 1.00, as does the HCM curve. This simple change in the coefficient can be implemented in many available software packages such as MINUTP.

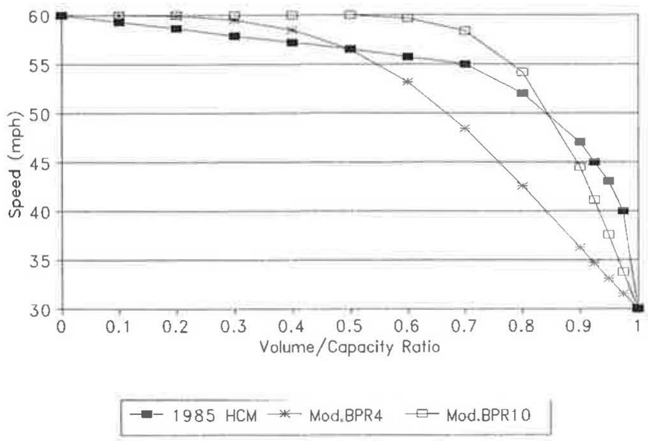

FIGURE 2 Fit of modified BPR curves to HCM curve.

The second curve seeks to correct the underprediction of speed by the first curve for values of v/c above 0.5. The curve (labeled "Mod.BPR10" in the figure) is as follows:

$$\text{congested speed} = (\text{free-flow speed})/[1 + (v/c)^{10}] \quad (3)$$

This latter curve provides a much better fit to the HCM curve, especially at the higher values of v/c ratios.

Speed-Flow Curves for Arterial Streets

Whereas operational models such as TRANSYT-7F can estimate average running speeds for an arterial street using free-flow speeds and queueing analysis at intersections, planning models must make their estimates solely on the basis of flow and capacity data for each link of the network. The development of satisfactory and relatively simple speed-flow curves for arterials has been particularly complex.

Several investigators have developed speed-flow curves for arterials (*6*). Chapter 11 of the 1985 HCM contains a method for determining average speeds on arterials on the basis of free-flow speed, intersection spacing, signal timing, and arterial class. From this information, the running speed between intersections is determined along with the average delay per intersection. The running speed and total intersection delay are then combined to determine average speed for the arterial.

$$\text{average speed} = \frac{\text{segment length}}{\text{run time} + \text{intersection delay}} \quad (4)$$

$$\text{run time} = \frac{\text{segment length}}{\text{running speed}} \quad (5)$$

A Class I arterial is a principal arterial primarily serving through traffic with speed limits of between 40 and 45 mph. A segment is generally the distance between signals on an arterial street. The running speeds are computed from the running times given in Table 11-4 of the HCM.

Intersection delay is calculated according to Equations 11-2 and 11-3 of the HCM. These equations require assumptions of the green time per cycle (g/c) for the arterial and the cycle length. The through-lane capacity and v/c ratios are also used in these delay equations.

The estimated signal delay varies for different g/c ratios, cycle lengths, and v/c ratios for an arterial with a single through-lane approach. The g/c ratio has as great an effect on intersection delay as the v/c ratio. Even for a known g/c ratio and v/c ratio, the assumed cycle length (60 or 180 sec) can double the estimated delay per intersection.

The estimated intersection delay will be quite unstable when the actual g/c ratios and cycle lengths for signals along an arterial are unknown.

Figure 3 shows the range in estimated average speeds for a Class I arterial, with a 40-mph free-flow speed, and signals at 1-mi spacing, assuming a certain range in g/c ratios and cycle lengths. As can be seen in this figure, the range in speeds is quite large, even for a given v/c ratio.

Figure 3 also compares the modified BPR curve developed for freeway links with the average speed estimates resulting from the method in Chapter 11 of the HCM. The modified BPR curve results in speed estimates higher than the 60- and 180-sec cycle estimates derived from Chapter 11. At v/c ratios in excess of 0.85, the modified BPR curve tends to overestimate the impact of congestion on arterial speeds.

It is apparent that a separate, flatter speed-flow curve could be developed to better match the Chapter 11 HCM estimates of average arterial travel speeds. However, the high variation in average speeds for a given v/c ratio appears to imply that little accuracy would be gained by such an effort. The variation in speeds for a given v/c ratio is greater than the variation in speeds across v/c ratios.

Therefore, for demonstrating the postprocessor methodology we will use the same speed-flow curve for both freeway and arterial street facilities. This curve is a modified BPR curve with a coefficient of 1.00 and with v/c raised to a power of 10. The more data-intensive method contained in Chapter 11 of the HCM is too volatile and is too dependent on data not generally available in planning studies for effective use in a planning model.

PROPOSED QUEUEING ANALYSIS TECHNIQUE

For v/c ratios in excess of 1.00 it is necessary to develop a simple queueing process that can work with the relatively limited data that would be available to a postprocessor after the planning model has completed its assignment process.

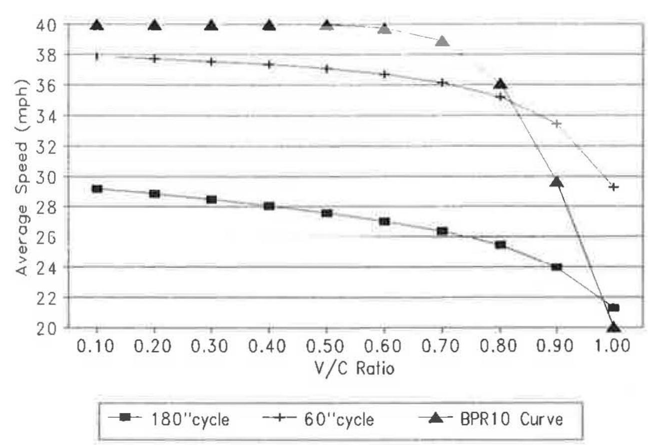

FIGURE 3 Modified BPR curve versus Chapter 11 HCM.

Queueing is a complex process that requires the determination of the variation of demand over time and consideration of the spatial characteristics of the queue itself.

Spatial Treatment of Queueing

The proposed treatment of the spatial nature of queueing is discussed for two situations: a freeway and an arterial street.

For a freeway, bottlenecks occur when there is a sudden increase in demand or a sudden drop in capacity. Figure 4 shows a typical queueing situation that could arise when an on-ramp merges with the freeway. The sudden increase in demand contributed by the on-ramp exceeds the capacity of the downstream segment of the freeway and causes traffic to back up on the ramp and on the upstream portion of the freeway itself. The congestion also reduces the traffic volumes able to continue downstream on the freeway, thus reducing the net demand on the downstream segments.

The postprocessor (since it must function after the assignment stage has been completed) must work with the individual link data from the loaded highway network. Consequently, it is not feasible for the postprocessor to track the upstream or downstream impact of queueing. The queueing analysis must be confined to the specific link at which it is detected.

The postprocessor therefore must assume that the queue on a freeway segment will occur in the segment in which the demand exceeds capacity. The effect of this assumption (shown in Figure 5) is similar to that of physically shifting the mainline and ramp queues from upstream of the bottleneck to the bottleneck itself. All vehicles entering the bottleneck section from the freeway mainline and the on-ramp would be presumed to share equally in the total congestion; in the real world, however, drivers would experience different delays depending on whether they are on the on-ramp or the freeway mainline section. This difference would affect the estimated delay for individual drivers but is not expected to significantly affect the estimate of average speed for all drivers on that freeway segment.

Notice that the length of freeway section operating at capacity (downstream of the artificial queue) would be reduced from its true real-world length. The mainline queue, however, would be increased in the idealized case (since it also includes the ramp vehicles queueing) over its true length in the real-world case. The net effect of this assumption is probably to underestimate congestion—but note that the processor is un-

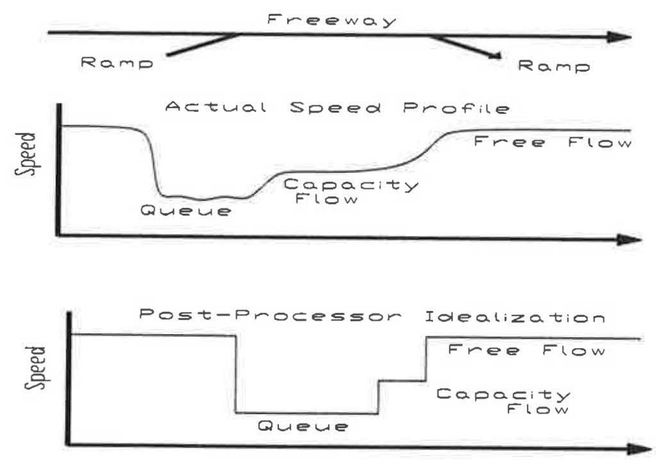

FIGURE 5 Actual versus idealized speed profiles for freeways.

able to reduce the demand on downstream links due to queueing, so when it analyzes flow conditions on downstream links, it will tend to overestimate congestion and thus underestimate average travel speeds over the entire freeway facility.

For a typical arterial street the queueing situation is different. The arterial has a relatively high through capacity until it encounters a major cross street. At this location, the through capacity is cut roughly in half by a traffic signal and queues form upstream of the traffic signal (see Figure 6). The signal also reduces the volume of traffic proceeding downstream from the intersection; however, this effect is partially counteracted by the addition of traffic turning from the cross street onto the arterial at the intersection.

The postprocessor's assumption of queueing occurring on the same link on which demand exceeds capacity works quite well for nonfreeway links (see Figure 7). Note in Figure 7 that queues due to midblock signals are not captured in the queue calculation if these midblock bottlenecks have a higher capacity than the controlling bottleneck for the link.

For freeways and arterial streets, the postprocessor is unable to deal with queues extending upstream beyond the link causing the queue. The queue in excess of the length of the individual link must be assumed to stack vertically on the same link. The total delay due to the queue is preserved with the exception that impacts on upstream links (blocking of intersections for example) are neglected. The effect of this limitation is to underestimate the congestion resulting from queues.

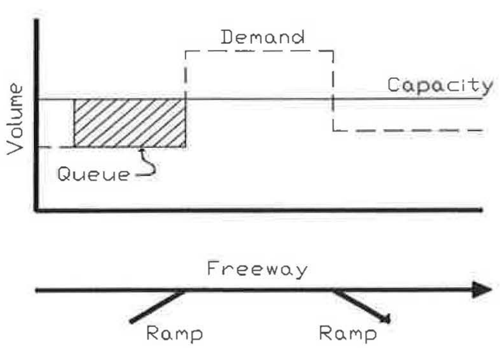

FIGURE 4 Freeway queueing pattern.

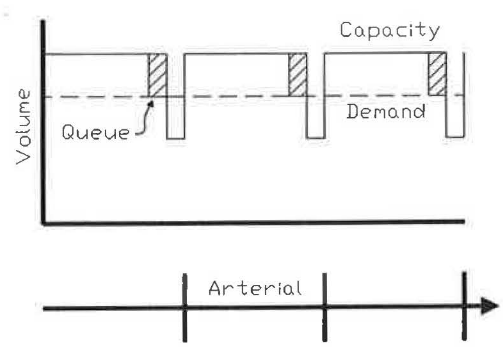

FIGURE 6 Arterial street queueing patterns.

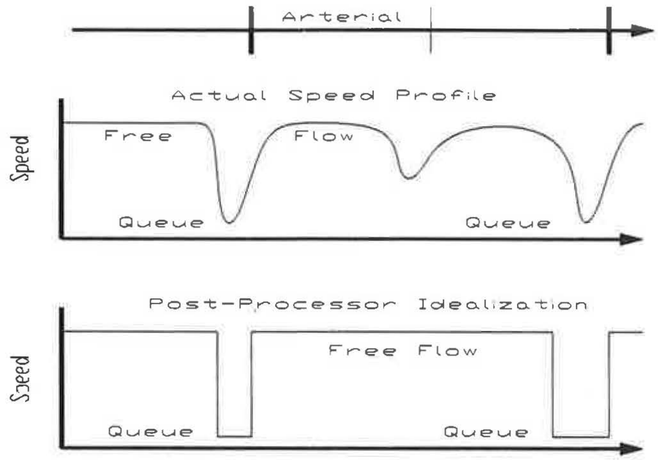

FIGURE 7 Actual versus idealized speed profiles for arterials.

The postprocessor is also unable to incorporate the reduction in downstream flows that occurs with queueing. Thus higher demands are carried through to all of the queueing calculations for the downstream links. The effect of this limitation is to overestimate the congestion effects of queueing, thus counteracting the underestimate cited earlier.

Temporal Treatment of Queueing

The temporal variation in demand can be dealt with by dividing the peak period into a sequence of time slices. For example a 5-hr peak period might be divided into five hour-long time slices with a percentage of the total demand estimated to occur during each hour. The hourly volumes can be estimated using peak-hour factors derived from traffic counts or home interview surveys.

The temporal variation of queues is handled in the postprocessor by splitting each peak period into hour-long time slices. Within each time slice, the demand and the capacity are assumed to be constant. This assumption underestimates the effects on queueing of peaking within the hour; however, the underestimate is considered to be minor. The computation requirements for the hourly time slices were approaching the capacity of the MINUTP software package, so it was not deemed feasible to go to 15-min time slices.

For each 1-hr time slice, the following two computations are made:

$$\text{average link speed} = \text{average queue speed} * \left(\frac{\text{average queue length}}{\text{link length}}\right) + \text{uncongested speed} * \left[1 - \left(\frac{\text{average queue length}}{\text{link length}}\right)\right] \quad (6)$$

$$\text{VHT} = \text{demand} * \text{link length/average link speed} \quad (7)$$

where

VHT = vehicle hours traveled;
average speed in queue = capacity/lane * 25 ft/vehicle;
average queue length = average queue * 25 ft/vehicle;
average queue = $(Q1 + Q2)/2$;
$Q1$ = queue at start of time slice;
$Q2$ = queue at end of time slice = $(Q1)$ + (demand rate) * (1 hr) − (capacity rate) * (1 hr); and
uncongested speed = free-flow speed/$[1 + (v/c)^{10}]$.

If the average queue length exceeds the link length, then the average link speed is set equal to the average queue speed, and the link length is set equal to the queue length.

At the end of the computations for all time slices, the following computations are made:

$$\text{VMT} = \text{total demand} * \text{link length} \quad (8)$$

$$\text{delay} = \text{VMT} * [(1/\text{average link speed}) - (1/\text{free-flow speed})] \quad (9)$$

where VMT equals vehicle miles traveled.

The density of 25 ft/vehicle (used in calculating queue length and queue speed) is typical for vehicles queueing on an arterial street at a traffic signal but not for vehicles on a freeway. To determine the density for vehicles queueing on a freeway, it is necessary to know the v/c ratios for the upstream links at which the queue occurs; however, this information is not available to the postprocessor. Consequently, the 25 ft/vehicle assumption has been applied to freeway queues as well. Other researchers may wish to improve on this assumption.

The resulting travel speeds in queues are shown as follows:

Facility	*Capacity (vph)*	*Speed (mph)*
Freeway	2,000	9.5
Expressway	1,200	5.7
Arterial	900	4.3
Collector	600	2.8
Ramp	1,700	8.0

Queues remaining at the end of the last time slice for a peak period are not added to the computations. This will tend to underestimate peak-period delay when it extends beyond the peak period, thus partially compensating for the overestimates in delay resulting from the postprocessor's inability to reduce demands downstream of the queue.

EXPERIMENTAL APPLICATION OF POSTPROCESSOR METHODOLOGY

The improved speed-flow curve and rudimentary queueing analysis was coded into a network processing utility (Netmerge) in the MINUTP software package. This routine was run on a 185-zone, 1,100-link loaded highway network that had been developed to forecast year 2010 traffic for the city of Hayward. The resulting VMT, VHT, average speed, and delay estimates were then isolated for two test facilities in this network: the proposed Route 238 freeway, and the existing Mission Boulevard (see Figure 8).

1. The proposed Route 238 freeway is a four-lane freeway that would extend 5.03 mi from Interstate 580 to Mission Boulevard at Industrial Parkway.
2. The Mission Boulevard test section is 3.23 mi long and has five traffic signals spaced an average of 0.8 mi apart. The

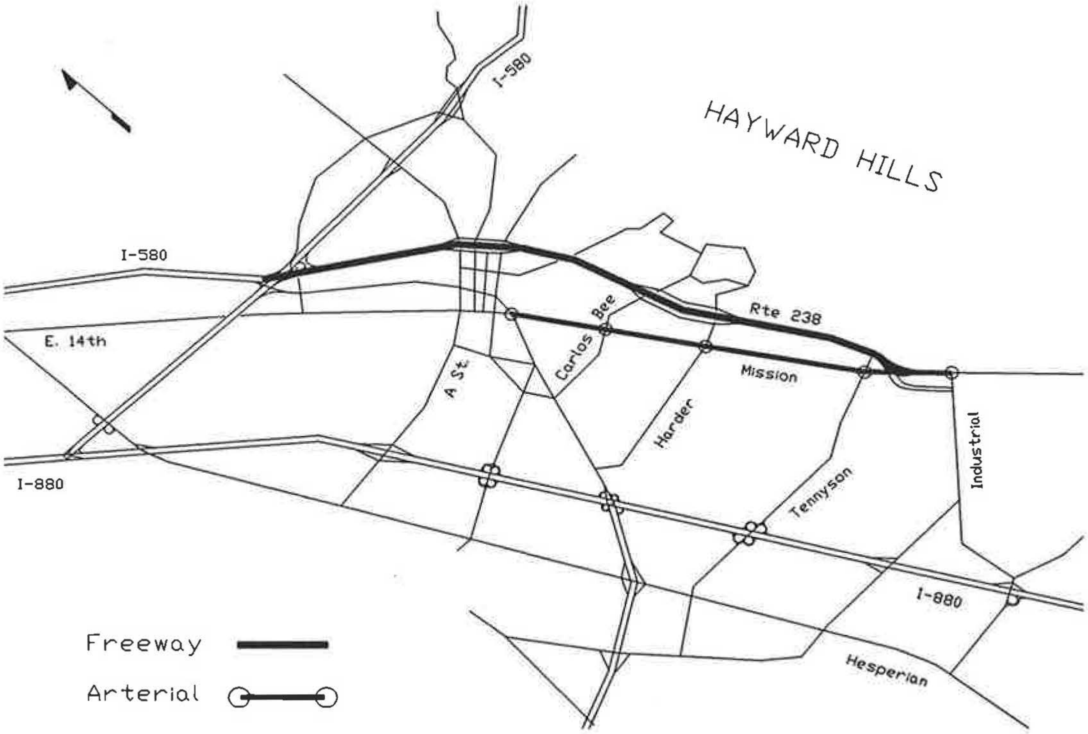

FIGURE 8 Hayward model study area: Route 238 and Mission Boulevard test sections.

signals are uncoordinated. Mission Boulevard varies between four and six lanes with left-turn pockets. The speed limit is 35 mph.

The average speeds and delay estimated by the described method (using the MINUTP software package) were compared for the Route 238 freeway facility against comparable speed estimates generated by FREQ freeway model simulation runs for the same freeway. The same link lengths, free-flow speeds, and capacities were coded in the FREQ model as were used in the MINUTP model run. The weaving analysis capability in FREQ was turned off to ensure that both models were using the same capacities. Auxiliary lanes (under 1,000 ft long) were coded in the FREQ model at major on-ramps to ensure that the idealized capacities assumed in the MINUTP model run were matched by the operational analysis at ramp merge points contained in the FREQ model.

Table 1 shows the results of this comparison. Note that FREQ calculates ramp delay but not ramp VMT or speeds. Note also that in this example there is relatively little congestion predicted for the freeway mainline. Some queueing is predicted for the on-ramps, thus limiting the amount of congestion predicted to occur on the freeway mainline.

The results show four tests of the postprocessor speed-flow curves and queueing analysis against the results of the FREQ run. The first two entries show the results of using standard planning model runs without a postprocessor (MINUTP). The next two columns show the results of applying the postprocessor to calculate queues and the improvements that result from using a speed-flow curve of $1 + v/c^{10}$.

The first column shows the results using the standard planning model run using the standard BPR curve. The planning model significantly underestimates mainline and ramp delay and overestimates average travel speed on the freeway mainline.

The second column shows the improvement in delay estimates that result if the $1 + v/c^4$ curve is used in a standard planning model instead of the standard BPR curve. The ramp delay estimate is significantly improved, but unfortunately the model now appears to overestimate delay on the freeway mainline and underestimates the average travel speed on the freeway mainline.

The postprocessor is applied in the third column, using the same curve that was used in the second column. Because there is little congestion on the freeway, the addition of the postprocessor's queueing analysis has little effect on the mainline freeway speed estimates (there is a slight improvement due to splitting the peak-period demand into five hourly periods and averaging the speed over the entire peak period rather than for the peak hour as used in the standard model runs). The postprocessor, however, does a significantly better job of identifying the delay that FREQ predicts would occur at the on-ramps.

TABLE 1 Straight Planning Model Versus Postprocessor Versus FREQ for Route 238 Freeway

Freeway	Standard MINUTP Runs		MINUTP Runs with Post-Processor		FREQ No Weave
Speed Eqn.	$1+0.15(v/c)^4$	$1+(v/c)^4$	$1+(v/c)^4$	$1+(v/c)^{10}$	HCM Curve
Queue Calc.	No Queue	No Queue	Queueing	Queueing	Queueing
VMT (Mainline)	115,979	115,979	115,979	115,979	116,255
VHT (Mainline)	2,066	2,820	2,790	2,484	2,328
Ave.Speed (Mainline)	56.1	41.1	41.6	46.6	49.9
Delay (Mainline)	134	884	856	552	390
Delay (Ramp)	15	100	510	480	483
Total Delay	149	984	1,366	1,032	873

The estimates of ramp delay and average freeway mainline speeds are even further improved when the curve of $1 + v/c^{10}$ is included in the postprocessor (Column 4).

TRANSYT-7F was run for various combinations of cycle lengths and signal coordination conditions to determine the likely range of average travel speeds to be expected on Mission Boulevard. Unfortunately, field data were not available to verify these results independently. Given the many years that TRANSYT-7F has been successfully used in California and the United States to simulate arterial signal operations and optimize signal timing, this was not considered to be a serious deficiency.

The peak hour was simulated with TRANSYT-7F, and the results expanded to the total peak period by multiplying VHT, VMT, and delay by 4 (since the peak-hour volumes are 25 percent of the total peak-period volumes). Average travel speed was obtained directly from the peak-hour results output by TRANSYT-7F. A test was also made running TRANSYT-7F five times (once for each hour of the peak period) and summing the results; however, this was found to significantly underestimate total delay because queues from one run of TRANSYT-7F could not be carried over to the following period.

Table 2 shows the results of the TRANSYT-7F runs. It illustrates the volatility of arterial street operations for different cycle lengths and signal coordination conditions.

The TRANSYT runs include various intersection design refinements to optimize the operation of Mission Boulevard (such as left- and right-turn pockets) that could not be specified in the MINUTP planning model run. TRANSYT was allowed to find an optimal equi-sat (equal degrees of saturation) signal timing solution for the splits for the noncoordinated condition. The MINUTP planning model run used in the postprocessor, however, coded only the number of through lanes at each intersection and assumed a 50:50 g/c split for each street at each major intersection for the purposes of determining link capacity.

Two TRANSYT runs were selected from Table 2 for comparison with the postprocessor. The 190-sec cycle runs were selected to show the results for an optimal cycle length under uncoordinated conditions and the impacts of signal coordination at the same cycle length.

Table 3 shows the results of various tests of the postprocessor against the two selected TRANSYT-7F simulations of Mission Boulevard. The reader can also compare these results with the additional TRANSYT-7F results shown in Table 2. The postprocessor comes quite close to predicting the correct average speed when the cycle length on the arterial is 100 sec.

TABLE 2 Summary of TRANSYT-7F Runs: Estimates of Delay and Average Speeds on Mission Boulevard

Arterial Street	190 Second Cycle		120 Second Cycle		100 Second Cycle	
	No Coordination	Coordination	No Coord	Coordination	No Coord	Coordination
VMT	18164	18164	18164	18164	18164	18164
VHT	1308	981	1333	903	1679	1315
Delay	699	469	820	390	1167	803
Ave.Speed	13.9	18.5	13.6	20.1	10.8	13.8

TABLE 3 Straight Planning Model Versus Postprocessor Versus TRANSYT-7F: Estimates of Delay and Average Speeds on Mission Boulevard

Arterial Street	Standard MINUTP Runs		Standard MINUTP Runs with Post-Processor		TRANSYT7-F 190 Second Cycle	
Speed Eqn.	$1+0.15(v/c)^4$	$1+(v/c)^4$	$1+(v/c)^4$	$1+(v/c)^{10}$	simulation	
	No Queueing Analysis		Queueing Analysis		No Coord	Coordination
VMT	17,258	17,258	17,258	17,258	18,164	18,164
VHT	533	755	1,665	1,648	1,308	981
Delay	38	260	1,162	1,145	699	469
Ave.Speed	32.4	22.9	10.4	10.5	13.9	18.5

The standard MINUTP model runs both significantly underestimate delay and overestimate average speed. The postprocessor runs overestimate delay and underestimate speed, but they are generally much closer at predicting the speed for noncoordinated conditions. The postprocessor appears to better estimate speeds that would occur under nonoptimal signal timing and turn pocket situations. A less conservative link capacity assumption (using an assumed g/c higher than 50 percent) would probably result in higher speed estimates from the postprocessor that more closely approximate the average travel times that could be achieved under optimized signal timing and coordination.

CONCLUSIONS

The proposed postprocessor methodology results in significantly improved estimates of delay and travel speeds compared with the raw planning model output. For the specific speed-flow curves and queueing density assumptions used in this example, the methodology appears to conservatively underestimate travel speeds. But this underestimate is still much closer to the speeds that would be predicted by traffic operations models than are the raw estimates coming from the planning model. Further improvements in the speed-flow curves for arterial streets and in the queue density assumptions used for freeways would probably improve these results.

REFERENCES

1. T. Imada and A. D. May. *FREQ8PE: A Freeway Corridor Simulation and Ramp Metering Optimization Model.* UCB-ITS-RR-85-10. Institute of Transportation Studies, University of California, Berkeley, June 1985.
2. *TRANSYT-7F User's Manual.* Transportation Research Center, University of Florida, Gainesville, Oct. 1988.
3. Comsis Corp. *Traffic Assignment.* FHWA, U.S. Department of Transportation, Aug. 1973.
4. *Special Report 209: Highway Capacity Manual.* TRB, National Research Council, Washington, D.C., 1985, Figure 3-4.
5. *MINUTP Technical User Manual.* Comsis Corp., Mountain View, Calif., Feb. 1988.
6. Cambridge Systematics. *Analysis of Temporal Demand Shifts to Improve Highway Speed Modeling.* Arizona Department of Transportation, Phoenix, April 1988.

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Comparison of Vehicular Emissions in Free-Flow and Congestion Using MOBILE4 and Highway Performance Monitoring System

WAYNE D. COTTRELL

Vehicular emissions in free-flow and congested traffic conditions are quantified and compared. The 1-hr volume/capacity ratio ranges that define free-flow and congested are 0 to 0.1 and 0.975 to 1.0, respectively. The MOBILE4 computer model was used to estimate the carbon monoxide (CO), hydrocarbon (HC), and nitrogen oxides (NO_x) speed correction factors and emission factors for a 1988 light-duty gasoline-powered vehicle (LDGV) executing a 10-mi trip from a cold-engine start. The Highway Performance Monitoring System analytical process (HPMS AP) was used to estimate average travel speeds and idling times in free-flow and congestion on urban surface streets and expressways. For the analytical conditions, emissions of CO are generally from 11 to 22 percent, depending on the facility type; they are greater in congestion than in free-flow. HC emissions are 3 to 8 percent greater in congestion. Congested NO_x emissions are 81 to 92 percent of free-flow emissions. Only CO and HC emissions increase as congestion worsens. One problem is that the HPMS AP is unable to simulate speeds lower than 13 mph. Another is that MOBILE4 was used to estimate single LDGV emissions rather than as a standard application to a vehicle fleet. Measurement of congestion with average speeds may oversimplify the true jammed driving aspects of accelerating, decelerating, and idling, each of which has a different emissions characteristic. The variable driving conditions of congested corridors, such as operating modes and the rate and amount of speed fluctuation, need more recognition in future model improvements.

The relationship between traffic congestion and vehicular emissions is unclear. A recent General Accounting Office (GAO) study stated that traffic congestion tends to increase the level of mobile source emissions of carbon monoxide (CO) and hydrocarbons (HC) (*1*). This is primarily because CO and HC emissions are greater when a vehicle is traveling at a slow speed, which increases the travel time and, thus, the running emissions (*2*). The emissions of CO and HC are also greater when a vehicle is accelerating, decelerating, or idling than during cruising at a steady, constant speed. The purpose of this paper is to quantify and compare mobile source emissions from light-duty gasoline-powered vehicles (LDGVs) in uncongested, free-flow travel conditions with those in congested conditions. From GAO's findings, it is expected that CO and HC emissions will be greater for an LDGV in congestion than for the same LDGV in free-flow travel. Emissions of nitrogen oxides (NO_x) will also be studied.

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MOBILE4 COMPUTER PROGRAM

MOBILE4 Method for Computing Emissions Factors

The MOBILE4 computer model calculates the grams of HC, CO, and NO_x emitted per vehicle mile for eight types of gasoline and diesel-fueled highway motor vehicles for low- and high-altitude regions of the United States (*3*). Among the eight individual vehicle types considered are LDGVs, light- and heavy-duty gasoline-powered trucks, light- and heavy-duty diesel-powered vehicles, and motorcycles. The emission factors can be used, when combined with estimates of vehicular travel, to estimate the total daily or annual mobile source emissions of a pollutant within an urbanized area or region.

MOBILE4 emission factors are dependent on a series of assumptions about ambient temperature, fuel volatility level, air conditioning use, loading and trailer towing, and vehicular operating mode, speed, age and mileage accumulation. Emission factors for LDGVs can be estimated for any year between 1960 and 2020, with the previous 20 model years presumed to be operating in any calendar year. The program estimates the mileage accumulation of each model year and the distribution of model years composing the fleet in each calendar year. The emission factors for any model year gradually increase with advancing calendar years because of mechanical deterioration resulting from mileage accrual.

MOBILE4 assumes that a vehicle of a given model year, in any chosen calendar year, accrues mileage and is affected by air conditioning, loading, towing, and environmental conditions to the same extent as all other vehicles of that model year in that calendar year. These are simplifying assumptions that eliminate consideration for different makes of LDGVs, the maintenance practices of individual owners, individual driving habits, different driving environments, or unusual mileage accrual rates.

Determination of Emission Rates and Use of Program

The mathematical form of a MOBILE4 emissions factor is a basic emission rate multiplied by a series of correction factors that account for the variables listed above. The values of the basic emission rates and the correction factors were determined from the results of the Federal Test Procedure (FTP).

The FTP for gasoline-fueled vehicles is a travel pattern, conducted under known driving and environmental conditions on city streets and highways, during which exhaust emissions are measured with a dynamometer (*4*). The major parts of the test are two 7.5-mi drives—one from a cold-engine start and one from a hot-engine start—into hot-stabilized mode. This provides cold-start, hot-start, and hot-stabilized emission estimates. During the FTP driving sequences, the vehicle accelerates and decelerates as in normal urban travel. The average overall speed of each drive is 19.6 mph. The maximum speed is 56.7 mph, while a total of 17.6 percent of the test time is spent idling.

The Environmental Protection Agency (EPA) provides suggestions and recommendations of the use of MOBILE4, particularly with respect to the determination of user-supplied values for variables. The EPA expects that many users of MOBILE4 will be developing regional mobile source emission inventories and projections for use in their State Implementation Plan processes. The program is periodically updated: MOBILE1, MOBILE2, and MOBILE3 successively preceded MOBILE4. MOBILE4.1, the most recent version, was introduced in November 1991 (the program's arrival postdated much of the work for this paper); MOBILE5 is under development.

One of MOBILE4's useful features is its sensitivity to model years, environmental conditions, and operating modes when assessing LDGV emissions. For example, a 1988 LDGV traveling 10 mi at 25 mph from a cold-engine start in calendar year 1989 with a fuel volatility of 9 psi at low altitude would emit 3.3 g of CO/vehicle-mi at 75°F and 55.2 g CO at 0°F. The same vehicle under similar conditions but from a hot-engine start would emit 1.9 g CO at 75°F and 54.7 g at 0°F. At high altitude, these values would increase from 1 to 13 percent. With a fuel volatility of 11.7 psi, the emissions would increase by about 9 percent. In this example, the CO emissions from a 1988 LDGV traveling at 25 mph would vary by more than 1,000 percent under various environmental and operating conditions (*3*). Emissions of CO per mile would further increase with older model year LDGVs, shorter trip lengths from a cold-engine start (because the LDGV would be "cold" for a greater portion of the trip), and slower average travel speeds.

Application of MOBILE4

To simplify the calculation of speed correction and emission factors, several assumptions were made. First, only LDGVs were considered, because they make up the bulk of the traffic stream. Second, LDGV model year 1988 and calendar year 1989 were used for the computations. A 1988 LDGV in calendar year 1989 would have accrued 13,118 mi on the basis of MOBILE4's assumptions. Third, the LDGV begins its trip from a cold-engine start, then moves into hot-stabilized mode. From the FTP, the initial 505 sec of any trip would be the cold-start emissions phase. Fourth, a trip length of 10 mi was used. Figure 1 shows the amount of time spent in the cold-start and hot-stabilized modes as functions of average speed for a 10-mi trip. The cold-start period, at 505 sec, is constant; the hot-stabilized time increases as the speed decreases. Finally, the following assumptions were used: 75°F ambient temperature, humidity of 75 grains of water per pound of dry air, fuel volatility of 9 psi, and no effects of tampering, an inspection and maintenance program, air conditioning use, trailer towing, or loading.

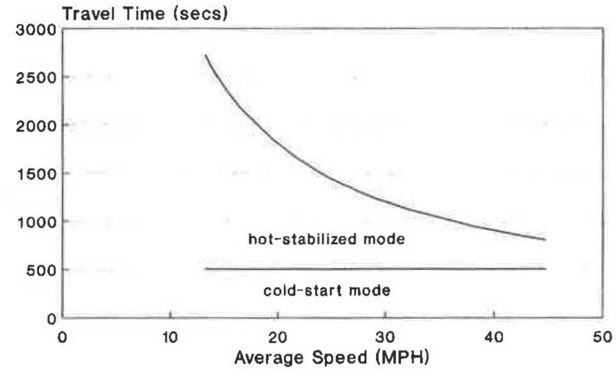

FIGURE 1 Cold-start and hot-stabilized fractions, 10-mi trip.

Speed Correction Factors

Equation 1 shows how MOBILE4 computes an LDGV emission factor. The basic emission rate for each model year in each calendar year is weighted by the fraction of total travel contributed by the model year. The composite correction factor, SALHCF, determined from Equation 2, accounts for speed, loading, and humidity variations as well as air conditioning usage and trailer towing. Variations in traffic conditions, as measured by a vehicle's average speed, are included in the speed component of the composite correction factor, SCF.

$$\mathrm{COMP}_{pn} = \sum_{i=n-19}^{n} \{\mathrm{TF}_{in} * [(\mathrm{BEF}_{pin} * \mathrm{SALHCF}_{ips} * \mathrm{RVPCF}) + \mathrm{REFUEL}_i + \mathrm{RNGLOS}_i + \mathrm{CCEVRT}_i]\} \quad (1)$$

$$\mathrm{SALHCF}_{ips} = \mathrm{SCF}_{ips} * \mathrm{ACCF}_i * \mathrm{XLCF}_i * \mathrm{TWCF}_i * \mathrm{HHH} \quad (2)$$

where

i = model year,
n = calendar year of analysis,
COMP_{pn} = composite emission factor for pollutant p,
TF_{in} = fraction of total miles driven by model year i by January 1 of calendar year n,
BEF_{pin} = basic exhaust pollutant p emission rate,
SALHCF_{ips} = composite correction factor, and
RVPCF, REFUEL_i, RNGLOS_i, CCEVRT_i = fuel volatility correction and the refueling, running loss, and crankcase and evaporative emissions factors, respectively.

The latter three are adjustments to HC emissions whereas RVPCF applies to HC and CO emissions from vehicles from model years 1971 through 1979. In Equation 2, SCF_{ips}, $ACCF_i$, $XLCF_i$, $TWCF_i$, and HHH are the speed, air conditioning, extra load, trailer towing, and humidity correction factors, respectively. HHH is used for NO_x emissions.

SCF is determined from Equations 3 through 6. For this paper it is assumed that, of all the emission correction factors, only the speed correction factor is affected by traffic conditions. Other emission correction factors may be affected by the additional miles driven to avoid congested situations; however, because traffic diversion is difficult to measure, this will be ignored.

$$SCF_{ips} = SF(s)/SF(sadj), \tag{3}$$

where

$$SF_{ip}(s) = (A_{ips}/s) + B_{ips} \tag{4}$$

or

$$= \exp(A_{ips} + B_{ips} * s + C_{ips} * s^2) \tag{5}$$

where s is the average travel speed in miles per hour and A_{ips}, B_{ips}, and C_{ips} are constants that each vary with i, p, and s. Equation 4 is used for HC and CO, and Equation 5 is used for NO_x. These equations apply to all post-1976 model year LDGVs. Other equations, not shown here, apply to older LDGVs. The variable $sadj$ is the FTP speed, 19.6 mph, adjusted for the fraction of cold-start and hot-start operation.

$$sadj = 1/\{[(w + x)/26] + [(1 - w - x)/16]\} \tag{6}$$

where w and x are the fractions of vehicles in hot-start and cold-start modes, respectively. During the FTP, the test vehicle spends 27.3 percent of its operation in hot-start, 20.6 percent in cold-start, and the remaining 52.1 percent in hot-stabilized mode. In cold- or hot-start mode, for the first 505 sec of each of the 7.5-mi test trips, the average speed is 26 mph. In hot-stabilized mode, for the remaining times of both test trips, the average speed is 16 mph. The overall average speed for both test trips is 19.6 mph. When local data indicate a greater cold- or hot-start modal fraction than occurs during the FTP, the average test speed is adjusted from 19.6 toward 26 mph. When data indicate a hot-stabilized modal fraction that is greater than in the FTP, the average test speed is adjusted from 19.6 toward 16 mph.

Most of the speed correction factors were determined using tests that supplement the FTP. Seven additional tests had average speeds ranging from 2.5 to 47.9 mph. In each test, the amount of accelerating, decelerating, and idling varied. In MOBILE4, which uses the results of all seven test cycles, SCF applies to all speeds between 2.5 and 55 mph; values of SCF for speeds between 48 and 55 mph are estimated. Four new tests, the results of which are incorporated in MOBILE4.1, had average speeds of 48, 51, 58, and 64 mph, thereby allowing the accurate determination of SCF for speeds between 48 and 65 mph (*5*). Because of these revisions, MOBILE4's SCFs, for this paper, apply only to speeds between 2.5 and 47.9 mph.

HIGHWAY DESIGN TYPES AND DRIVING CHARACTERISTICS

HPMS Travel Speed Estimation

To compare vehicular emissions in free-flow and congested conditions, estimates of average speeds, accelerations, decelerations, and idling for different highway design types were made using the Highway Performance Monitoring System analytical process (HPMS AP). The HPMS AP computes average travel speeds in miles per hour for various vehicle types, classes of road, geographic areas, and other strata (*6*). The average speed is a function of initial running speed (IRS), pavement condition, roadway curves and gradients, speed change cycles (SCCs), stop cycles (STPs), and the fraction of time spent idling (IDL).

From HPMS, IRS is the initial speed in miles per hour at which vehicles tend to travel under a set of influential factors. These include the highway's location (i.e., urban or rural), functional class, design speed, speed limit, and peak-hour volume/capacity ratio (v/c). Initial running speed includes the cruising modes of a vehicle and excludes accelerations, decelerations, and stops. A speed change cycle consists of a deceleration to a final speed (FS), followed by an acceleration to IRS. A stop cycle is a speed change cycle with FS equal to zero. The percentage of travel time spent idling (at FS = 0) is IDL. Including only the effects of traffic conditions and ignoring pavement condition, curves, and gradients, an average travel speed (ATS) can be calculated from IRS, SCC, STP, and IDL, as follows:

$$\begin{aligned} ATS = 1/\Big[& \{[D_{SCC} * (SCC - STP)]/[(IRS + FS)/2]\} \\ & + [(D_{STP} * STP)/(IRS/2)] \\ & + \Big(\{1 - [D_{SCC} * (SCC - STP)] \\ & - (D_{STP} * STP)\}/IRS\Big) + \{IDL/[IRS * (1 - IDL)]\}\Big] \end{aligned} \tag{7}$$

The numerator of Equation 7 is the unit distance of travel (1 mi), and the denominator is the time it takes to travel 1 mi, accounting for accelerations, decelerations, and idling. The variables in Equation 7 are calculated as follows:

$$SCC = 0.5 * (A - \log_{10}IRS + B) \tag{8}$$

$$STP = 0.5 * 10^D * (2 * SCC)^C \tag{9}$$

$$FS = [(1.0 + F) * IRS] - G \tag{10}$$

$$D_{SCC} = (IRS^2 - FS^2)/RATE \tag{11}$$

$$D_{STP} = IRS^2/RATE \tag{12}$$

where

SCC, STP = number of speed change and stop cycles per vehicle mile, respectively;

FS = average final speed, or low point, of all speed change cycles;
A, B, C, D, F, and G = constants that depend on the facility type; and
D_{SCC}, D_{STP} = distances traveled during a speed change and a stop cycle, respectively.

Both equations assume constant acceleration and deceleration rates (RATE) of 2.5 ft/sec/sec for speeds above 30 mph and 5 ft/sec/sec for speeds below 30 mph (*7*). These rates are similar to those recorded by vehicles during the FTP and in supplemented test cycles.

The HPMS AP is primarily used to identify highway conditions, estimate capital investment needs, and measure changes in highway conditions over time (*8*). To identify highway conditions, the HPMS AP can be used to compute performance measures such as average travel speed. The HPMS AP average speed estimation procedure calculates ATS as a function of initial running speed and the number of speed change and stop cycles. IRS, which is associated with cruising rather than speed changes, is a function of v/c. The values of SCC and STP are the estimated numbers of deceleration-acceleration cycles; these two variables are functions of IRS. With this approach, the effects of the demand-supply relationship on ATS may be double-counted. That is, v/c, SCC, and STP, which are measures of roadway capacity usage, each reduce the initial running speed. The result is that average travel speeds may be slightly lower than they should be. This issue is to be examined in proposed research. For this paper, the HPMS AP speed estimation procedure has been used as is.

The speed estimation procedure considers idling in addition to running speed, accelerations, and decelerations. IDL is a constant based on the facility type and v/c. Assuming that the desired travel time is the time that would have been spent traveling at the initial running speed, the total travel time is the desired time plus the time spent idling. The time spent idling per mile traveled (T_{idle}) is therefore

$$T_{idle} = \mathrm{IDL}/[(1 - \mathrm{IDL}) * \mathrm{IRS}] \tag{13}$$

The HPMS AP uses values of IDL that, for v/c less than 0.9, range from 1.9 percent on freeways and expressways to 8.1 percent on multilane roads. For v/c greater than 0.9, IDL ranges from 6.5 percent on freeways and expressways to 31.4 percent on multilane roads. These are fairly similar to the idling times experienced by vehicles in the FTP and supplemented test cycles.

Highway Design Types

Equations 7 through 13 show that, in the HPMS AP, average speed estimation is related to the initial running speed and to some facility type constants. The facility types considered by the AP fall within four classes: freeways and expressways, multilane (nonfreeway) roads, two- or three-lane roads, and one-way roads. A different initial running speed is associated with each unique combination of functional class, speed limit, highway design speed, lanes, and v/c. Initial running and average travel speeds can be estimated for urban facilities for both free-flow (v/c = 0 to 0.1) and congested traffic conditions (v/c = 0.975 to 1.0). Considering only urban facilities with average speeds within the range that MOBILE4 is suitable for (2.5 to 47.9 mph) and excluding one-way roads, there are 27 facility types having unique sets of free-flow and congested IRS values. These 27 facility types, which include all urban nonfreeway roads (expressways, multilane, and two- or three-lane roads), have 15 free-flow and 11 congested IRS values. Tables 1 and 2 list these 26 values of IRS along with descriptions of the associated facility types. Then, employing Equations 7 through 13, the tables summarize the average travel speed associated with each initial running speed and facility

TABLE 1 Initial and Average Travel Speeds and Urban Highway Design Types: Midpoint of v/c Range = 0.05 (*6*)

IRS (mph)	Facility Type	Speed Limit (mph)	AHS (mph)	Lanes	ATS (mph)
48.8	multilane	≥ 45	≥ 40	≥ 4	32.3
48.5	2-3 lanes	≥ 45	any	2-3	29.4
47.9	expressway	≥ 45	≤ 55	any	44.7
46.0	expressway	40	any	any	42.8
44.0	multilane	40	≥ 40	≥ 4	28.8
43.6	2-3 lanes	40	any	2-3	26.4
41.0	expressway	35	any	any	38.0
39.0	multilane	≥ 35	≥ 30	≥ 4	29.7
38.7	2-3 lanes	30-35	any	2-3	23.7
35.0	expressway	30	any	any	32.2
33.0	multilane	30	any	≥ 4	25.3
29.0	expressway	≤ 25	any	any	27.4
28.0	multilane	≤ 25	any	≥ 4	21.7
25.0	multilane	any	< 30	≥ 4	19.6
24.8	2-3 lanes	≤ 25	any	2-3	19.4

Note: AHS = highway design speed.

TABLE 2 Initial and Average Travel Speeds and Urban Highway Design Types: Midpoint of v/c Range = 0.9875 (*6*)

IRS (mph)	Facility Type	Speed Limit (mph)	AHS (mph)	Lanes	ATS (mph)
29.0	expressway	≤ 25	> 65	≥ 4	26.2
28.7	expressway	≥ 30	60-65	any	25.9
28.5	expressway	≤ 25	60-65	any	25.7
27.3	expressway	any	≤ 55	any	24.6
22.8	multilane	any	≥ 40	≥ 4	16.4
22.1	multilane	any	30-40	≥ 4	15.9
21.1	multilane	any	< 30	≥ 4	15.3
19.7	2-3 lanes	≥ 45	any	2-3	14.2
19.4	2-3 lanes	≤ 40	any	2-3	14.0
19.2	2-3 lanes	30-35	any	2-3	13.9
18.2	2-3 lanes	≤ 25	any	2-3	13.2

Note: AHS = highway design speed.

type. The values of ATS range from 13.2 to 44.7 mph. From Figure 1, at 44.7 mph, the cold-start operating mode fraction is 62.7 percent; at 13.2 mph, the fraction is only 18.5 percent of the 10-mi trip travel time. Thus, any increased emissions that occur with slower speeds and longer travel times may be offset somewhat by the greater cold-start fraction that is present with high average speeds.

INTEGRATION OF MOBILE4 AND HPMS AP

Relationships Between Speed Correction and Emission Factors

An important assumption of this paper is that the speed correction factor is proportional to the composite emission factor for each pollutant. This relationship is shown in Equations 1 and 2. Figure 2 shows SCF versus the emission factors for CO, HC, and NO_x for a 10-mi trip by a 1988 LDGV at speeds ranging from 2.5 to 47.9 mph. The driving and environmental conditions assumed are as follows: calendar year 1989, low altitude, an ambient temperature of 75°F, humidity of 75 grains of water per pound of dry air, fuel volatility of 9 psi, a cold-engine start (with the initial 505 sec composing the cold-start phase), and no effects of tampering, an inspection and maintenance program, air conditioning use, trailer towing, or loading. Additionally, free-flow conditions are represented by a *v/c* of 0 to 0.1, whereas congestion has a *v/c* of 0.975 to 1.0. Both CO and HC emissions increase, nearly linearly, as SCF increases. The emissions of NO_x decrease, almost linearly, as SCF increases. The CO emissions factor ranges from 2.9 g/mi at 47.9 mph to 7.4 g/mi at 2.5 mph. The corresponding range of SCFs is from 3.17 to 0.45. The HC emissions factor increases from 1.0 to 2.1 g/mi, with corresponding SCFs between 6.72 and 0.45 for the same range of speeds. Hence, CO emissions are more sensitive to changes in speed than HC emissions are. Notably, based on MOBILE4.1, both the CO and HC emission factors would begin to increase at speeds greater than 48 mph (*9*). Both the SCFs and emission factors for NO_x increase between 2.5 and 47.9 mph.

Values of Speed Correction and Emission Factors

The next steps in the analysis integrated MOBILE4's method for computing SCF and emission factors with the HPMS AP speed estimation procedures for free-flow and congested traffic conditions. Values of speed correction and emission factors were computed using the average speeds summarized in Tables 1 and 2. Table 3 summarizes free-flow and congested SCFs for CO, HC, and NO_x for a 10-mi trip on seven common urban facility types. The facilities are expressways with 45-mph speed limits, 55-mph design speeds, and six lanes; multilane (i.e., four-lane) roads with 45-, 40-, and 35-mph speed limits; and two- or three-lane roads with 30- and 25-mph speed limits. The associated values of SCF indicate how LDGV emissions would be expected to differ between free-flow and congested traffic conditions.

From Table 3, for CO emissions in free-flow conditions on urban facilities, SCF is lowest on an expressway (0.47) and greatest on a two- or three-lane road with a 25-mph speed limit (0.97). From Table 4 and Figure 3, the related CO emission factors are 2.93 and 3.77 g/mi, respectively. In congestion, SCF for CO emissions is lowest on an expressway (0.75) and greatest on a two- or three-lane street (1.12). The CO emission factors are 3.27 and 3.70 g/mi, respectively. The free-flow CO emission factor on a two- or three-lane road is slightly greater than in congestion. Table 4 and Figure 3 show that the CO emission factor increases as speed decreases from 44.7 to 19.4 mph, decreases as the speed falls to 13.9 mph, then increases as the speed drops to 13.2 mph. Hence, between 13.2 and 44.7 mph, for a 10-mi trip done under the assumed conditions, the CO emissions per mile are greatest in free-flow on a two- to three-lane road with a 25-mph speed limit. In congestion, CO emissions would be nearly equivalent on all urban surface streets. On a congested expressway, the CO emission factor would be about 12 percent less than on congested surface streets.

Table 3 shows that values of SCF for HC emissions are similar in magnitude to those for CO. In free-flow SCF is lowest on an expressway (0.47) and greatest on a two- or three-lane road with a 25-mph speed limit (0.92). From Table 4 and Figure 3, the corresponding HC emission factors are

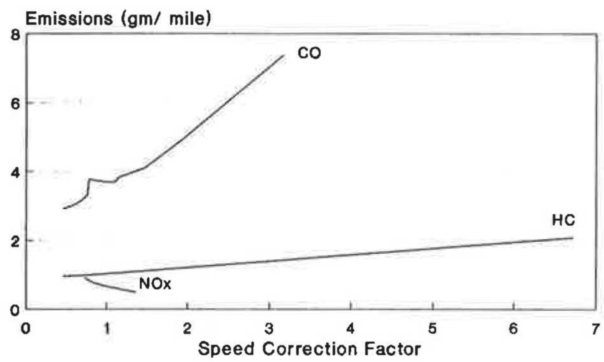

FIGURE 2 Speed correction and emission factors: 1988 LDGV, 10-mi trip, 2.5 to 47.9 mph, MOBILE4 estimates.

TABLE 3 Speed Correction Factors for Some Common Urban Facility Types

Facility Type	Traffic Conditions	ATS (mph)	SCF CO	SCF HC	SCF NO_x
expressway (45/6)	free-flow	44.7	0.47	0.47	0.74
	congested	24.6	0.75	0.75	0.90
multilane (45/4)	free-flow	32.3	0.60	0.60	0.82
	congested	16.4	1.03	1.08	1.02
multilane (40/4)	free-flow	28.8	0.66	0.66	0.85
	congested	16.4	1.03	1.08	1.02
multilane (35/4)	free-flow	29.7	0.64	0.64	0.84
	congested	15.9	1.04	1.10	1.03
2 or 3 lane (30/2)	free-flow	23.7	0.77	0.77	0.91
	congested	13.9	1.09	1.26	1.07
2 or 3 lane (25/2)	free-flow	19.4	0.97	0.92	0.97
	congested	13.2	1.12	1.32	1.08

The numbers in ()'s following the facility type designation give the speed limit and the number of lanes.

SOURCE: HPMS AP, MOBILE4; Figure 2 and Tables 1 and 2.

TABLE 4 LDGV Emissions Estimates for Some Common Urban Facility Types

Facility Type	Traffic Conditions	ATS (mph)	Emissions (gm/ mile) CO	HC	NO_x
expressway (45/6)	free-flow	44.7	2.93	0.96	0.89
	congested	24.6	3.27	0.99	0.72
multilane (45/4)	free-flow	32.3	3.04	0.97	0.78
	congested	16.4	3.70	1.04	0.65
multilane (40/4)	free-flow	28.8	3.13	0.98	0.75
	congested	16.4	3.70	1.04	0.65
multilane (35/4)	free-flow	29.7	3.10	0.97	0.76
	congested	15.9	3.70	1.05	0.65
2 or 3 lane (30/2)	free-flow	23.7	3.31	0.99	0.71
	congested	13.9	3.69	1.07	0.63
2 or 3 lane (25/2)	free-flow	19.4	3.77	1.01	0.68
	congested	13.2	3.70	1.08	0.62

The numbers in ()'s following the facility type designation give the speed limit and the number of lanes.

See the text for the conditions assumed for the computation of emissions.

SOURCE: HPMS AP, MOBILE4 and Table 3.

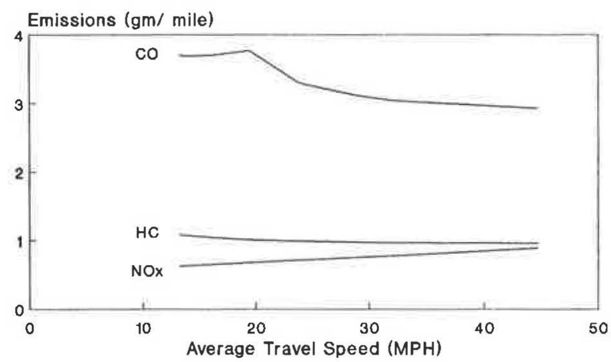

FIGURE 3 LDGV emissions at HPMS speeds: 1988 LDGV, 10-mi trip, cold-engine start, 1989 MOBILE4 estimates.

0.96 and 1.01 g/mi, respectively. In congestion, SCF is lowest on an expressway (0.75) and greatest on a two- or three-lane road (1.32). The corresponding emission factors are 0.99 and 1.08 g/mi, respectively. On all urban facilities studied, the HC emission factor is greater in congestion than in free-flow for a 10-mi trip executed under the assumed conditions.

From Table 3, SCF values for NO_x emissions in free-flow are lowest on an expressway (0.74) and greatest on a two- or three-lane road with a 25-mph speed limit (0.97). From Table 4 and Figure 3, the corresponding NO_x emission factors are 0.89 and 0.68 g/mi, respectively. In congestion, the SCF values for NO_x are lowest on an expressway (0.90) and greatest on a two- or three-lane road (1.08). The corresponding emission factors are 0.72 and 0.62 g/mi, respectively. On all urban facilities studied, the NO_x emission factors in congestion are lower than those for free-flow conditions for a 10-mi trip executed under the assumed conditions. The relationship between NO_x emissions and congestion is inverse to those of CO and HC emissions.

TABLE 5 Free-Flow and Congested LDGV Emissions

Facility Type	Free-Flow and Congested ATS (mph)	Ratio of Congested to Free-Flow Emissions CO	HC	NO_x
expressway (45/6)	44.7/ 24.6	1.12	1.03	0.81
multilane (45/4)	32.3/ 16.4	1.22	1.07	0.84
multilane (40/4)	28.8/ 16.4	1.19	1.07	0.87
multilane (35/4)	29.7/ 15.9	1.19	1.07	0.85
2 or 3 lane (30/2)	23.7/ 13.9	1.11	1.08	0.88
2 or 3 lane (25/2)	19.4/ 13.2	0.98	1.07	0.92

The numbers in ()'s following the facility type designation give the speed limit and the number of lanes.

See the text for the conditions assumed for the computation of emissions.

SOURCE: HPMS AP, MOBILE4 and Table 4.

Table 5 lists the ratios between the congested and free-flow CO, HC, and NO_x emission factors for six urban facility types. On five of the facilities, CO emissions would be 11 to 22 percent greater in congestion than in free-flow. For a two- or three-lane road with a 25-mph speed limit, CO emissions would be about 2 percent greater in free-flow than congestion. On all six facilities, HC emissions would be from 3 to 8 percent greater in congestion than free-flow. Both CO and HC emissions are generally greater in congested than in free-flow conditions, but CO is more sensitive to changes in average speeds than HC. On all six facilities, NO_x emissions in congestion would be from 81 to 92 percent of free-flow emissions. Hence, emissions of NO_x are lower in congestion than free-flow because of the lower average travel speeds. These results, of course, are applicable to the conditions assumed for the computations.

Other combinations of assumptions would likely yield results different from those reported here. For example, for the 10-mi trip used for the computations, the cold-start fraction ranged from 18.5 to 62.7 percent of the total trip for the speeds studied (13.2 to 44.7 mph), as shown in Figure 1. For a 20-mi trip, over the same range of speeds, the cold-start fraction would range from 9.3 to 31.4 percent of the trip time. The CO, HC, and NO_x emission factors would be lower for the longer trip because a reduced amount of driving would be conducted in cold-start mode. For a 1-mi trip, the cold-start fraction for the entire range of speeds would be 100 percent. For this trip, the emission factors would be greater than for a 10- or 20-mi trip. The emission factors of all three pollutants would also be greater for an LDGV older than 1988, at high altitude, at a higher fuel volatility, or with air conditioning use, trailer towing, or loading. A higher humidity would contribute to greater NO_x emissions, and a lower ambient temperature would lead to greater emissions of all three pollutants. A very high ambient temperature would propagate higher CO and HC emissions. The emission factors of all three pollutants would be lower with a hot-engine start or with an inspection and maintenance program in effect. The effect of variations in driving and environmental conditions presents many opportunities for further analysis of emissions in free-flow and congested travel.

Further examination of SCF reveals that its value increases sharply at speeds below 15 mph, particularly for CO and HC

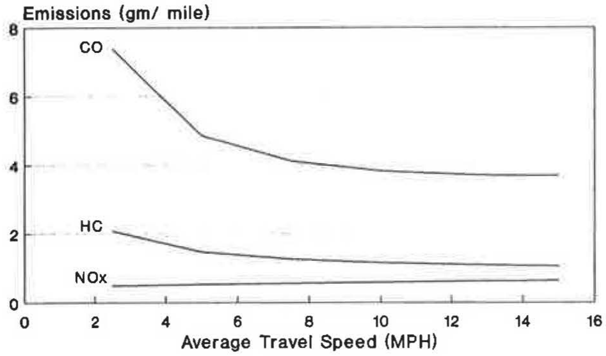

FIGURE 4 LDGV emissions at low speeds: 1988 LDGV, 10-mi trip, cold-engine start, 1989 MOBILE4 estimates.

(*3*). For example, for a 1988 LDGV traveling at 13.2 mph for a 10-mi trip from a cold-engine start, the SCFs for CO, HC, and NO_x would be 1.12, 1.32, and 1.08, respectively. From Figures 2 and 4, at 10 mph, the SCF values would be 1.27, 1.72, and 1.16 for CO, HC, and NO_x, respectively. At 5 mph, the values would be 1.90, 3.39, and 1.29; at 2.5 mph, they would be 3.17, 6.72, and 1.36. Between 13.2 and 2.5 mph, from Figure 4, the CO emissions factor would increase from 3.70 to 7.38 g/mi. Over this range of speeds, as shown in Figure 4, the HC emissions factor would increase from 1.08 to 2.08 g/mi, and the NO_x emissions factor would decrease from 0.62 to 0.49 g/mi. The HPMS AP does not calculate an average speed of less than 13.2 mph on any urban facility, even for congested surface streets experiencing a *v/c* of 0.975 to 1.0. Analysis of the severe congestion implied by such low speeds would require local speed, delay, and queueing data rather than the procedure used by the HPMS AP. It is also not clear that the MOBILE4 model would be appropriate for emissions analysis on the scale at which localized traffic data would be collected.

CONCLUSION AND RECOMMENDATIONS

It is evident that MOBILE4 calculates emissions assuming that the driving cycle values of the FTP and supplemental test cycles are typical for their speeds. The amounts of cruising, acceleration, deceleration, and idling during the tests are presumed to be applicable to all other driving situations. However, many combinations of the amount of time spent in each of the four driving cycle elements can produce the same average speed. The speed correction factor is an attempt to recognize this. For example, the emissions test procedure for very low driving speeds uses more accelerating, decelerating, and idling than the FTP does. However, here, too, the emission rates are specific to the driving cycle values of the test. This is a concern because the emission rates of the driving cycle elements may be very different. Sensitivity to the amount of cruising, accelerating, decelerating, and idling, and to the intensity of accelerations and decelerations, is not included in the test procedures and therefore does not exist in MOBILE4.

The ideal model for this paper would integrate estimation procedures for exhaust emissions with a driving simulation model that is sensitive to ambient traffic conditions. The simulator would quantify the amounts of driving in each of the cruising, accelerating, decelerating, and idling modes. Evans (*10*), for example, examined the relationship between the driving modes and emissions using the FTP and 12 automobiles from the 1975–1976 model year. Using the limited, and now old, data, Evans showed that HC emissions are strongly correlated with average travel speed. He also demonstrated a weak correlation between CO and NO_x emissions and average speed but a high correlation between the two pollutants and acceleration-deceleration measures. Similar research is needed on modern vehicles to determine the current relationships between traffic variables and emissions. One requirement of the 1990 Clean Air Act Amendments is the testing of "off-cycle" emissions at steeper acceleration and deceleration rates than have been used in past test procedures. The results may update earlier research findings with emission rates that are accurate for traffic conditions on surface streets and in congestion.

Further points are indicated by this study, even though there is a need for an improved approach at quantifying and comparing free-flow with congested emissions. First, for the assumed driving and environmental conditions, emissions of CO and HC are generally greater in congestion than in free-flow on urban surface streets and expressways. Second, for the assumed conditions, emissions of NO_x are lower in congestion than in free-flow on urban surface streets and expressways. Traffic conditions measured with local data could produce different results. That is, local data could exhibit higher average free-flow or lower congested speeds than those computed by HPMS for similar highway design types. The retrieval and analysis of such data are recommended to provide an alternative to the HPMS speed estimates.

To reduce urban area CO and HC emissions, the easing of congestion would be beneficial because travel speeds would increase and travel times would decrease. The emissions of NO_x, however, may increase with reduced congestion and greater speeds. This proposes a problem, since NO_x builds up in the atmosphere, potentially contributing to the greenhouse effect. Reducing trip ends, for the purposes of reducing motor vehicle travel as well as cold-engine starts and HC evaporation, would certainly ease emissions. Advances in vehicle technology should also continue to benefit the emissions-versus-speed relationship.

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REFERENCES

1. General Accounting Office. *Traffic Congestion: Trends, Measures, and Effects*. GAO/PEMD-90-1. Report to the Chairman, Subcommittee on Transportation and Related Agencies, Committee on Appropriations, U.S. Senate, Nov. 1989.

2. *The Air Pollution-Transportation Linkage*. Office of Strategic Planning, California Air Resources Board, Sacramento, 1989.
3. *Supplement A to Compilation of Air Pollutant Emission Factors, Volume II: Mobile Sources*. AP-42 Project, Test and Evaluation Branch, U.S. Environmental Protection Agency, Jan. 1991.
4. *Protection of the Environment*. 40 C.F.R., Part 86, 1989.
5. *Speed Correction Factors in MOBILE4.1*. Office of Mobile Sources, U.S. Environmental Protection Agency, April 1991.
6. *Highway Performance Monitoring System Analytical Process, Volume II—Version 2.1, Technical Manual*. Office of Planning, FHWA, U.S. Department of Transportation, Dec. 1987.
7. R. Winfrey. *Economic Analysis for Highways*. International Textbook Company, Scranton, Pa., 1969.
8. General Accounting Office. *Highway Needs: An Evaluation of DOT's Process for Assessing the Nation's Highway Needs*. GAO/RCED-87-136. Report to Congressional Requesters. Aug. 1987.
9. *Overview of Differences in MOBILE4.1*. Office of Mobile Sources, U.S. Environmental Protection Agency, Boston, Mass., Nov. 1991.
10. L. Evans. *Exhaust Emissions, Fuel Consumption and Traffic: Relations Derived from Urban Driving Schedule Data*. GMR-2599. General Motors Research Laboratories, Warren, Mich., Dec. 1977.

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Carbon Monoxide Emission Effects of Drive-Up Facilities

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The effects of drive-up facilities such as banks, restaurants, liquor stores, and laundries on air quality are investigated; particular attention is given to carbon monoxide emissions. The primary purpose of the study is to provide guidance on whether regulation of drive-up business is warranted and desirable. Carbon monoxide monitoring, on-site traffic counts, queueing and service times, and arterial traffic counts were simultaneously obtained at three typical drive-up facilities in Tucson, Arizona. The field data were analyzed to help determine the magnitude of air quality impact and the amount of traffic generated by the various facility types. A mail-out survey of drive-up facility regulations was also conducted. Significant insight about the current operation, use, and air quality effects of drive-up facilities is provided. First, field measurement is not an effective means of determining the level of carbon monoxide air pollution generated by a drive-up business. Second, computer modeling can be used effectively to estimate pollution levels at drive-up facilities. Third, many communities regulate the design and location of drive-up businesses, but not for reasons of air quality. Fourth, the use of drive-up facilities can produce less carbon monoxide pollution than the park-and-enter alternative if the service time is less than 2 min. Fifth, the traffic impacts of drive-up facilities are difficult to assess because only limited data are available. The amount of pass-by and diverted linked trips has not been adequately quantified for the broad range of drive-up services. The findings of this study reflect conditions in the Tucson metropolitan area. Recommendations and conclusions of the study may be also valid in other regions after local needs and conditions are carefully considered.

Drive-up facilities are commonplace in today's American cities. Their prevalence reflects American reliance on the automobile for personal transportation, enjoyment of a high degree of mobility, and insistence on speed, value, and convenience in many aspects of our daily lives. There has been continuing concern over the environmental implications—both energy use and air pollution—of drive-up restaurants, banks, stores, and related enterprises since their inception in the 1970s. Poorly planned and designed drive-up lanes can increase congestion, cause traffic circulation problems, and provide less-than-welcome intrusion into neighborhoods. Additionally, drive-up businesses often are at or near congested locations that may already be hot spots for air pollution.

This study addresses the interrelationship between air quality and the use of drive-up facilities in the Tucson, Arizona, metropolitan area. The study was authorized by the Tucson City Council and managed by the Tucson Department of Transportation, Planning Division, through a grant from the Arizona Department of Transportation.

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STUDY PURPOSES

This study investigates the air quality impacts of drive-up facilities such as banks, restaurants, liquor stores, and laundries. Some public services are also provided on a drive-up basis. Therefore, the private sector should not be singled out as the sole user of drive-up establishments, although public use is small in comparison to the private sector. The thrust of the study is to determine what air quality impacts are associated with drive-up facilities and the magnitude of these impacts on an individual location or in the aggregate, and to determine whether or not such facilities should be regulated by local government. If such regulations are desirable, an ancillary purpose is to suggest what types of regulations should be imposed and how sites may be evaluated for their air quality impact. Another purpose of this study is to develop a local information data base on the service times at typical facilities, as well as the trip generation rate and the temporal distribution of demand for these kinds of enterprises. This information can then be correlated with other local and national data.

The study also provides an overview of existing air quality computer models developed by the Environmental Protection Agency (EPA) and state and local government, as well as other models available from the private sector. From these models, one or more may appear satisfactory for use in analyzing drive-up facilities.

Drive-up facilities are not all the same; therefore, this study attempts to categorize the different types of facilities and describes their relative effectiveness by design and use. A site-specific evaluation procedure is described that is based on current practice and may be used in the future whether or not the city of Tucson adopts specific air quality regulations. In other words, these procedures can be used to help refine the design of the facility itself relative to circulation, demand management, traffic flow, and parking. Mitigation strategies are recommended to help improve the design of drive-up facilities that will help minimize the amount of waiting time (vehicle idling time) encountered at typical drive-up businesses. Mitigation strategies may be mandated by city government as part of the development approval process. Alternatively, mitigation guidelines may be used by the design sector on an advisory basis to help improve the design of the project site.

This project has been conducted using several concurrent levels of investigation and analysis. First, a literature search of similar or related studies was undertaken using manual methods, as well as an on-line, interactive data base. We also used the Planning Advisory Service of the American Planning

Association to find ordinances and regulations of similar agencies that would otherwise not be identified through a literature search. This information proved invaluable in describing the historic development of drive-up facility regulation as well as the evolution of the industry itself.

Second, several levels of field analysis and data collection were undertaken. Air quality monitoring, on-site traffic counts, queueing and service times, and arterial traffic counts were simultaneously obtained at three typical drive-up facilities in the city of Tucson. The sites were monitored for a typical midweek business day during November 1990. Air quality data were collected by Pima County's Department of Environmental Quality, using a mobile monitoring van. Traffic counts were obtained into and out of the facilities, as well as on adjoining streets. An on-site observer monitored customer arrival, service, and departure times at the various facilities. The field data were analyzed to help determine the magnitude of air quality impact and the amount of traffic generated by the various facility types. The field studies also included gathering service time data at several other types of businesses. This information was used to help understand how various drive-up facility designs and services impact vehicle idling time.

Third, we obtained information from the drive-up industry about service goals, trends in technological innovation, and related background information. This effort included telephone discussions with local and corporate spokesmen for Burger King and McDonald's, as well as review of industry publications.

A mail-out survey of drive-up facility regulations was also conducted. More than 70 surveys were mailed out to a variety of municipalities throughout the United States. More than 40 surveys were returned and analyzed.

BACKGROUND: AUTOMOBILES AND AIR POLLUTION

The use of the automobile is directly responsible for a major portion of the air pollution problem in major metropolitan areas, including the Tucson region. The pollutants of concern include carbon monoxide (CO), nonmethane hydrocarbon (NMHC), sulfur oxides (SO_x), nitrogen oxides (NO_x), lead (Pb), and particulate matter 10 μm or less in aerodynamic diameter (PM_{10}). Ozone (O_3) is also a major concern, but it is a secondary pollutant formed by NMHC and NO_x in the presence of sunlight and is not directly emitted in auto exhaust. All of these pollutants are generated by transportation vehicles including cars, trucks, trains, and aircraft. Private vehicles (cars and light-duty trucks) are the greatest contributing factor to several of these pollutants (*1,2*). Most significant to this study are CO and PM_{10}. These two primary pollutants have caused the eastern Pima County air shed to fall out of compliance with national air quality standards. Pima County and the city of Tucson are generally in compliance with all other national standards.

The automobile produces differing levels of CO and PM_{10} under various operating modes and driving conditions. For example, CO concentrations can become excessive whenever a large number of vehicles are operating in a given location. This can happen in very slowly moving traffic during periods of congestion, and at signalized intersections. Vehicle idling also occurs at other locations, such as parking lots, service facilities, and drive-up lanes. PM_{10} includes microscopic dust particles from vehicle braking systems, tire wear, and engine exhaust. In the Tucson area, dust is created about equally by natural sources and transportation systems. The vortex of a moving vehicle causes surface dust to become airborne, or reentrained, resulting in impairment of visibility and potential hazards to health at high concentrations. The amount of dust produced by vehicles in motion is proportional to vehicle speed, particularly on unpaved surfaces. Since this study addresses drive-up facilities where vehicle speeds are less than 5 mph on paved surfaces, PM_{10} is not a major concern. Therefore, from an air quality perspective, CO is the principle pollutant of concern and will be emphasized throughout this study. The study looks only at the atmosphere outside the vehicle, so CO concentrations inside vehicles or drive-up establishments are excluded.

HISTORY OF DRIVE-UP FACILITIES

Drive-up restaurants and banks became increasingly popular in the 1970s, although their use has been traced back to the 1930s (*3*). Drive-up windows are popular because they offer perceived speed and convenience compared with parking a vehicle and entering a business for specific services. A wide range of businesses and public sector services are provided at drive-up windows. Banks, restaurants, liquor stores, laundries, and other private businesses rely on drive-up operations for a major portion of their revenue. Public sector drive-up services, such as vehicle registration, emission testing, and postal services have been used with great public acceptance. Drive-up facilities have become a recognized part of the American lifestyle.

TRENDS IN DRIVE-UP FACILITY USE

More and more businesses are using drive-up facilities to remain competitive. Several national fast-food restaurant chains have improved the design of their facilities by providing separate menu boards and pay windows in addition to the necessary service window. This trend has allowed faster service times and a greater proportion of business revenue to be generated through the drive-up window. Emerging technologies, such as radio communication and interactive video, are also being incorporated into drive-up design. The convenience and speed demanded by the customer has resulted in some businesses establishing service time goals. For example, Burger King and McDonald's have target service times of 90 sec or less. The trend towards more and faster drive-up services is apparent, both in the literature and the marketplace.

Emerging technologies may have an important influence on how, when, and where we travel in the future. A recent report announces a new petroleum refining process that reduces automobile air pollution by a third (*4*); oil companies encourage the public to drive less and carpool (*5*); and vehicle manufacturers are marketing the next generation of fuel-efficient vehicles. Telecommunications already allow us to conduct business without traveling, and mobile communications make it possible to be more productive while traveling. Vehicle guidance and navigation systems will help increase highway capacity and improve traffic safety (*6*). These technological trends will continue, resulting in an even more mobile society. The role of drive-up businesses is expected to increase, not diminish, as service becomes faster and mobility-enhancing technologies are implemented.

The recently adopted Clean Air Act Amendments of 1990 are already having a marked effect on urban planning, vehicle manufacturing, and fuel refining (*7*). Emission standards for light-duty vehicles have become more stringent, and the use of alternative or oxygenated fuels is mandated for some areas. As automobile usage increases, vehicles are becoming simultaneously "cleaner." The net result is that levels of carbon monoxide, overall, are improving in Tucson, and probably will continue to improve over the next 10 years until offset by rising regional vehicle miles of travel. This same phenomenon could also occur in larger, more severely polluted cities.

AIR QUALITY MODELS AND LAND USE CONTROLS

This section of the study addresses current computer models and the relevant land use controls exercised by other communities. Computer models are useful for estimating aggregate emissions or emission concentrations near stationary or mobile sources. This section emphasizes mobile source emission models applicable to slow moving traffic in urban areas, similar to traffic conditions at drive-up facilities.

Air Quality Models

Air quality models primarily include computer-based algorithms developed by governmental agencies or private software vendors. Air quality models, however, may also include simple manual calculations, as described in the literature. Computer models must rely on a set of assumptions about emission rates, dispersion methods, and physical processes affecting ambient air quality levels. Generally, the models are defined as numerical, Gaussian, statistical, or physical (*8*). Numerical models are extremely complex and use a finite element approach to estimating downwind pollution concentrations. These models are computationally rigorous, therefore, not utilized often. On the other hand, Gaussian models use a simplifying assumption that the pollutant plume is transported according to a normal distribution. This assumption greatly simplifies the computational process at the expense of theoretical precision. Virtually all models approved by the EPA for regulatory and screening purposes are Gaussian dispersion models. Statistical and physical models are used for special studies, not for routine analysis.

Project analysis for mobile source emissions of carbon monoxide should include preliminary screening techniques. If predicted concentrations using these techniques exceed the ambient air quality standards, then more refined techniques should be used to determine compliance with the standards. CALINE3 is the preferred model when refined analysis is required. A newer release of this model, called CALINE4, is also available, but it has not yet been approved by EPA. For free-flow sources, the latest version of mobile source emission factors available from the model MOBILE4 is required as input to CALINE3, and for interrupted flow sources, such as signalized intersections, procedures to calculate mobile emission factors are contained in other EPA guidance documents. Point and area sources are modeled using algorithms other than CALINE3.

Proprietary models are available from several private vendors. These models, however, vary little from the EPA models. Private vendors frequently obtain the public domain software and modify the software to make it more user friendly. Modifications typically include simplification of model input parameters, simplified file maintenance algorithms, and addition of screen graphic and printer output options. The costs of proprietary models are significantly higher than the public domain models, often by a factor of 10 to 20. Proprietary sources, however, can provide training on the use of these models which may not be available from public agencies. Proprietary models are as acceptable for regularly and screening purposes as the public domain version of the same models.

Research has recently been undertaken to evaluate the effectiveness of carbon monoxide models in the urban environment (*9–11*). There is some agreement among researchers that (a) the models tend to underestimate actual pollutant concentrations, (b) the models are difficult to calibrate with field data, and (c) the models tend to oversimplify the impact of meteorological conditions on the location of highest concentration downwind of an intersection.

Current Regulation of Drive-Up Facilities

For this study, we conducted a survey of other community's regulatory practices. The survey was distributed to more than 70 cities throughout the United States, but emphasized the Southwest. The surveys were mailed to cities ranging in size from 50,000 to 1,000,000 residents. Approximately 42 surveys were returned. The univariate results of the survey are tabulated in Figure 1. A multivariate analysis was also conducted, though not tabulated herein. The high response to the survey indicates that there is a significant interest in the regulation of drive-up facilities. This is also substantiated by the 92 percent of the respondents requesting a copy of this final study.

Almost one-third of the cities enforce some kind of specific drive-up business regulation, yet almost none requires an air quality impact study. Current regulations of drive-up businesses are generally not a response to local air quality problems. Most cities that currently have drive-up regulations feel that the regulations are ineffective. Those that feel their regulations are effective generally have special zoning ordinances for drive-up facilities, whereas regulations considered ineffective are general, governing all types of development. It can be concluded that effective regulation of drive-up facilities should (a) be specifically directed to drive-up uses, (b) include site design guidelines, and (c) include a development review committee process.

FIELD DATA COLLECTION

This section of the study describes the field data collection effort and presents the results of this effort.

I. BACKGROUND

C. Air Quality Status: Please indicate your community's compliance with National Ambient Air Quality Standards.

Pollutant	Compliance	Noncompliance	Don't Know
CO (carbon monoxide)	(18) 42%	(13) 30%	(12) 28%
PM_{10} (particulate matter)	(18) 42%	(10) 23%	(15) 35%
NO_x (nitrous oxides)	(25) 58%	(3) 7%	(15) 35%
SO_x (sulfur oxides)	(25) 58%	(2) 5%	(16) 37%
O_3 (ozone)	(16) 37%	(12) 28%	(15) 35%
Pb (lead)	(24) 56%	(1) 2%	(18) 42%

II. DRIVE-UP FACILITY REGULATIONS

A. Does your community enforce special planning, analysis, or design requirements on drive-up facilities?

Yes	NO	
(27) 63%	(16) 37%	(If No, Skip to F)

B. Which of the following do you require?

	Yes	No	Don't Know or (no response)
Air Quality Impact Study	(1) 2.0%	(21) 49.0%	(21) 49%
Design Review Committee	(20) 46.5%	(8) 18.5%	(15) 35%
Noise Impact Study	(2) 5.0%	(18) 42.0%	(22) 51%
Queueing Analysis	(17) 40.0%	(10) 23.0%	(16) 37%

C. Do you have a special section of your zoning or development code addressing drive-up facilities?

Yes	No	Don't Know or (no response)
(17) 39.5%	(15) 35%	(11) 25.5%

D. Approximately when were these regulations adopted?

1950 = Earliest
1960s = 4
1970s = 2
1980s = 11
1990s = 2
Most were adopted in the mid- to late 1980s.

E. Are your drive-up facilities regulations generally effective, and not in need of revision?

Yes	No	Don't Know or (no response)
(13) 30%	(14) 33%	(16) 37%

F. Are you currently considering adopting new regulations?

Yes	No	Don't Know or (no response)
(14) 33%	(27) 63%	(2) 5%

III. FURTHER INFORMATION

B. Indicate whether you would like a copy of the final study.

Yes	No	Don't Know or (no response)
(40) 93%	(3) 7%	(0) 0%

FIGURE 1 Survey results.

Multifaceted Collection Plan

A multifaceted field data collection effort was utilized to help quantify air quality impacts of three typical drive-up facilities in the Tucson area. The data collection effort was also used to evaluate whether monitoring a drive-up facility is a practical method of enforcing site-specific air pollution regulations using locally available equipment. Concurrent data were collected for carbon monoxide and meteorological conditions, on-site traffic, queueing/service data, adjoining arterial traffic volumes, and ambient carbon monoxide background concentrations. Data were collected in December 1990. These data were collected in a joint effort by the City of Tucson Department of Transportation, Pima County Air Quality Control District (PCAQCD) and the project consultant. The three sites selected include a fast food restaurant, a bank, and a vehicle emissions testing facility. The selected businesses and their locations are considered typical of the Tucson area. The emissions testing facility was included to help demonstrate that some public and quasi-public services are also provided on a drive-up basis. The sites were also selected because of their physical orientation allowing on-site parking of the PCAQCD van, adequate electrical outlets, ease of observation of the drive-up lane operation, and the cooperation of the facility manager.

The van was placed 100 to 400 ft from the drive-up lane, and CO data were collected at each site for 12 to 24 hr. The van was equipped with a single Monitor Labs 8830 infrared CO monitor, a Sumx 445 data logger, and a Linear 156 strip chart recorder. The CO sample inlet was located approximately 9 ft above ground level. Wind speed and direction were monitored at 13 ft above ground level. Air temperature inside the van was monitored to assure calibration of the equipment. Outside temperatures were obtained from the National Weather Service and the ACQD's permanent monitoring sites. For each site, a field visit and coordinating meeting with the facility manager or owner were conducted prior to data collection.

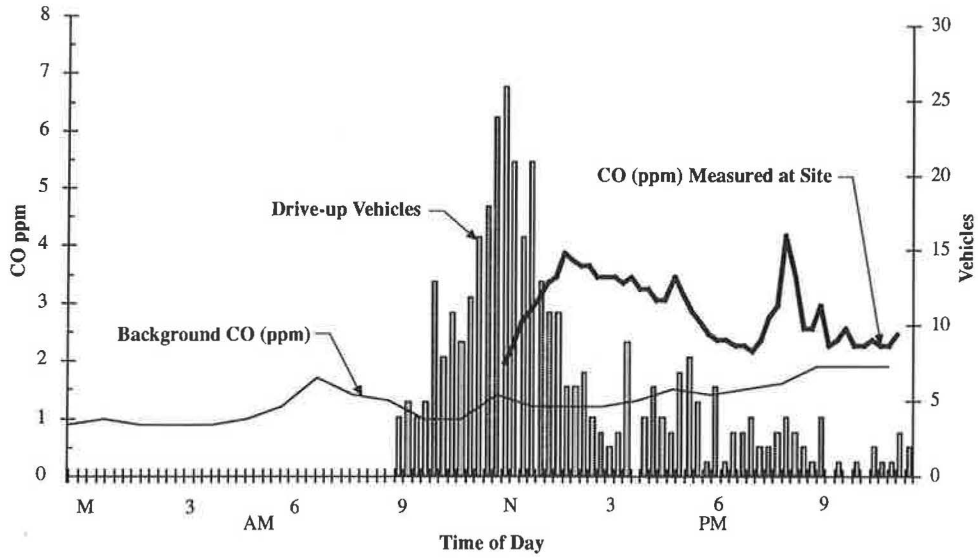

FIGURE 2 Field data summary: fast-food restaurant.

Typical results of the data collection efforts are shown in Figure 2. This exhibit contains time-based data regarding: (a) the number of vehicles queued at the fast-food facility, (b) the CO concentration measured at the site, and (c) the background CO concentration recorded at a permanent monitoring site several miles away.

The CO concentrations recorded at the sites are less than 9 parts per million (ppm), much less than the national standard of 35 ppm for 1-hr exposures. A graphical analysis approach using superposition of the data has been used. This technique is similar to an approach used by EPA (*12–14*). Although the data collection effort went smoothly, interpretation of the data shows little, if any, correlation between the amount of traffic through the drive-up facility and the CO level detected at the monitoring van. The major difficulties with this effort included: (a) variable winds that shifted the concentration point away from the single CO monitor at the sampling van, and (b) the low concentration of CO being emitted by vehicles in queue, as well as very low ambient levels of CO. The finding that measured CO concentrations do not correlate well with field observations of the emission source, although a disappointment in light of the amount of effort to collect the data, has also been concluded by others. For example, EPA (*8*) states that

> calibration of short-term models is not common practice and is subject to much greater error and misunderstanding. There have been attempts by some to compare short-term estimates and measurements on an event-by-event basis and then to calibrate a model with results of that comparison. This approach is severely limited by uncertainties in both source and meteorological data and therefore it is difficult to precisely estimate the concentration at an exact location for a specific increment of time. Such uncertainties make calibration of short-term models of questionable benefit. Therefore, short-term model calibration is unacceptable.

The findings of independent researchers are also consistent with this EPA viewpoint.

It can be concluded that the air quality monitoring approach used here is not a viable tool for enforcing locally adopted emission regulations at drive-up facilities. A much more sophisticated (and more expensive) approach using numerous monitors and extensive on-site traffic data collection would be needed. Drive-up facility regulation would be more effective in the form of site design and performance standards rather than the establishment of incremental emissions levels contributed by drive-up operations. Physical site constraints, variable meteorological conditions, the limitation of monitoring equipment, and the cost of data collection simply preclude site-specific monitoring as a realistic regulatory tool.

Industry Data

Personal communication with spokesmen of McDonald's and Burger King yields important insights into the fast-food business as well as the use of drive-up facilities. The industry is highly competitive and some of the major corporations have established service time goals that generally range from 90 to 120 sec from arrival time to departure time (*15,16*). To help improve service time, new technologies and architectural design strategies have been incorporated into drive-up facilities. The single-window design of the 1970s has evolved to a two-window (pay/pick-up) system with a separate menu board. This design allows speed of service during peak hours, and flexibility to operate with only one window open during lesser demand periods. State-of-the-art drive-up lanes provide service times, according to our field study, as quick as fifteen seconds. Industry spokesmen claim that video technology will be introduced into the next generation of drive-up facilities. This will allow "face-to-face" communication between patron and server that will minimize mistakes in order preparation.

The drive-up industry goal of quicker service time is consistent with any proposed regulatory program to minimize vehicle idling. However, the industry also attempts to site their businesses at high traffic locations. These locations also tend to be CO hot spots. Introduction of additional idling vehicles at already congested locations may be counter productive. This consideration must be weighed against the hours of peak air pollution at a given location. According to the spokesmen, the busiest time for drive-up restaurants is at lunch—that is, from 11:00 a.m. to 1:00 p.m.—and again after 5:30 p.m. for dinner. Drive-up financial services peak in the midafternoon. Therefore, industry data indicate that service time goals of 1½ to 2 min generally support environmental goals, whereas the desire to locate drive-up businesses near congested intersections does not.

Traffic Queueing Data

The analysis of the queueing portion of on-site traffic data collected for this study is identified in Table 1. It is evident that the national fast-food chains attempt to provide quick service in an effort to conform with their service goals. However, fewer than 20 percent of the customers were served within 90 sec and fewer than 30 percent were served within 120 sec. The peak periods identified by major fast-food industry spokesmen were verified by field observations. The maximum wait observed was 26 min in a line of 14 waiting vehicles at the emission testing station. The queueing/service discipline utilized at a specific site has a correlation with anticipated service time. Fastest service can be offered by the menu board/two-window system for already-prepared food. The range of service times at drive-up banking windows is highly variable depending upon the services provided. Businesses which use a single-window system for food items prepared on request can take the longest amount of time.

Results of Data Collection

Based on graphical interpretation and linear regression analysis, there is little observed correlation between the measured CO concentration and the amount of vehicle idling occurring at the various drive-up facilities. As mentioned previously, this is attributable to the low emission rates generated at the facilities, overall low CO concentrations, and the variability of wind direction limiting measured impact at the sampling van. Other elements of the data collection phase provide insight about trip generation rates at drive-up facilities, average service times and peak hours of service as these factors occur in the Tucson metropolitan area. The calculated trip generation rates are within the range of expectation of the ITE's Trip Generation handbook (*17*).

The evaluation of average service times can be related to both air pollution and energy consumption using a break-even analysis. This analytic approach relates the mass of CO emitted by a vehicle under hot start conditions with the time needed for an idling vehicle to emit the same mass of CO. Such an analysis is dependent on the vehicle fleet using drive-up facilities (generally passenger cars and light-duty trucks) and the air quality models used to estimate emission characteristics of that fleet. A range of break-even times can be established using a range of reasonable model input assumption.

For the 1990 vehicle fleet, the minimum break-even point for equal level of CO emission under the drive-up versus the park and enter scenarios has been calculated based on an incremental hot start emission rate of 15 g using EMFAC7D and idle emission rate of 147 and 423 grams per hour using EMFAC7D and MOBILE4, respectively. For CO, the primary concern of this study, the break-even range is approximately 2 to 6 min (K. D. Drachand, California Air Resources Board, unpublished data). Similarly, this compares with the average service time measured by this study at some of the project sites.

It can be concluded that an efficiently run drive-up lane (i.e., one that provides service in less than two minutes) produces less carbon monoxide than if a vehicle was stopped and restarted. This conclusion is generally consistent with those cited in the literature. Most of the observed service times, however, were within the 2- to 6-min break-even point range. For the observed facilities, the net impact on air quality is elusive.

An additional consideration in the break-even analysis is the effect of technological changes in the vehicle fleet. The estimates cited here are for the 1990 California fleet. A similar analysis for the 1995 California fleet results in longer break-even times because idle emission rates for this fleet are lower. The results of the break-even analysis are sensitive to factors such as inspection and maintenance programs, fuel oxygenation, and the composition of the vehicle fleet, among others. Care must be used in developing and applying break-even analysis for a specific location.

TABLE 1 On-Site Traffic Data

Site	Business	Type of Queue*	Service Time (sec) Min	Max	Std Dev	Observations	Average
Valley Bank	Financial	One-to-Many	50	1,065	201	201	399
Emission Testing	Govt. Service	One-to-Many	180	1,575	236	212	366
Burger King	Fast Food	2-Window/MB	55	450	75	100	154
Liquor Store	Liquor	1-Window	65	295	150	10	176
KFC	Fast Food	1-Window/MB	145	430	153	3	320
Hardee's	Fast Food	1-Window/MB	74	180	33	12	111
Dairy Queen	Fast Food	1-Window	115	340	95	7	218

* The definitions of queuing disciplines are as follows:
MB - Menuboard
One-to-Many - One wait lane discharges to many service windows.
1-Window - A single window is used to take the customer's order, receive payment, and serve the order.
1-Window/Menuboard - Service is ordered at menuboard; payment and product are exchanged at the service window
2-Window/Menuboard - Service is ordered at a menuboard; payment is made at the first window; service is provided at the second window.

From an energy conservation perspective, a break-even analysis can also be conducted. The amount of fuel consumed during an average 2-min service time cycle is approximately 0.02 gal (*18,19*). This compares with a consumption of approximately 0.002 gal under the stop/restart scenario. It is apparent that approximately 10 times more fuel is needed to use the drive-up lane than to park and enter a business, primarily because restarting an engine relies on battery power, not gasoline. The amount of additional gasoline consumed in drive-up service lanes is not considered an important issue during times of abundant petroleum supply. More air pollution in the form of carbon monoxide could be created if use of drive-up lanes were restricted in an effort to conserve fuel. The characteristics of today's vehicles and drive-up operations have lead to an apparent conflict between public air pollution and fuel conservation policies. Energy conservation and contingency programs currently emphasize supply problems, not air pollution levels.

RECOMMENDATIONS AND CONCLUSIONS

The community survey and data collection efforts, as well as the many sample regulations obtained from other cities, provide significant insight and policy guidance. The city currently has no specific drive-up facility regulations, design guidelines, or mechanisms for review by city staff. Tucson currently complies with the NAAQS and is expected to remain in compliance in the foreseeable future. However, the region is officially designated as a CO nonattainment area by the EPA primarily because recertification as an attainment area has not been requested by local officials. The study effort leads to a series of recommendations regarding air quality maintenance and planning, site design guidelines, and agency review.

Overview of Study Results

This study provided needed information about the current operation, use, and air quality impacts of drive-up facilities. The results are summarized here as a preamble to the recommendations and concluding remarks.

1. Field measurement is not an effective means of determining the level of CO air pollution generated by a drive-up business. Ambient CO levels vary throughout the day, as do wind speed, wind direction, and traffic on nearby streets. The high variability of pollution sources and meteorological factors makes separation of source variables extremely difficult. The amount of pollution associated with operation of a drive-up facility can not effectively be measured in the field under real world conditions.

2. Computer modeling can be used to estimate pollution levels at drive-up facilities. The accuracy of current models is uncertain and model output represents an approximation of actual emission concentrations. Appropriate EPA-approved models currently include CALINE3 using MOBILE4.1 (adjusted for fuel oxygenation) as emission factor input.

3. Many communities successfully regulate the design and location of drive-up businesses, although not primarily for reasons of air quality. The most successful approach, according to the community survey, is adoption of a specific drive-up business ordinance and design review process. None of the communities responding to the survey prohibit drive-up businesses entirely, although some prohibit drive-up businesses in congested areas or in sensitive neighborhoods.

4. The use of drive-up facilities can produce less carbon monoxide pollution than the park-and-enter alternative, if the service time is less than the approximately two minute break-even point. Conversely, the park-and-enter alternative uses only about one-tenth the fuel required for the average drive-up operation. These relationships will undoubtedly change as technological advances in engine efficiency, zero emission vehicles, and cleaner fuels are introduced.

5. The traffic impacts of drive-up facilities are difficult to assess because only limited data are available. Applications of standard traffic engineering techniques are limited because on-site distribution of customers to park-and-enter and drive-up services has not been well documented in the literature. Further, the amount of pass-by and diverted linked trips has not been adequately quantified for the broad range of drive-up services.

Drive-Up Facility Design Guidelines

Effective drive-up facility planning includes consideration of many design elements. The city currently has no specific regulations for drive-up facilities, and should develop its own guidelines or ordinances based on local need and the success of other communities. The following recommendations will help attain this goal.

- Prepare site planning and design guidelines that regulate access, circulation, traffic impacts, location, noise, odors, lighting, and other elements important to the community. These guidelines should be advisory and made part of the design review process. The sample regulations obtained from other communities for this study, ITE guidelines, and the resources cited herein provide adequate guidance for this purpose.
- Because of the localized effects of carbon monoxide, air quality modeling should only be required for proposed facilities at critical locations, and not indiscriminately required of each proposed drive-up establishment.
- The regulations and review processes should be evaluated for their effectiveness on a periodic basis, and revised to reflect changes in technology, air pollution regulations, or community need.

Drive-up facilities are an expected part of the urban experience. They provide convenience, and efficient drive-up lanes (those with less than 2-min service times) create less CO, yet use more gasoline than the park-and-enter alternative. Site design guidelines can help provide more efficient drive-up operations. Adoption and implementation of appropriate design guidelines and air quality analysis procedures should be viewed as mutually beneficial to businesses and regulatory agencies. Additional research is needed regarding how the location of drive-up businesses along major arterials detracts from highway capacity. This interrelationship may actually be more significant than the design of the site itself because diminished arterial highway capacity increases congestion, which in turn, increases CO emissions and the need for highway improvements.

The findings of this study reflect conditions in the Tucson metropolitan area, and are subject to the uncertainty inherent to the use of air quality models. Recommendations and conclusions of the study may be valid in other regions after careful consideration of local needs and conditions.

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REFERENCES

1. T. Godish. *Air Quality*. Lewis Publishers, Inc., Chelsea, Mich., 1985.
2. B. James. *Highways and Air Pollution, A Citizens' Primer*. Sullivan Center for Science in the Public Interest, Washington, D.C., 1974.
3. A. McAllister. *Zoning For Fast-Food and Drive-In Restaurants*. Report 320. Planning Advisory Service, American Society of Planning Officials (now American Planning Association), 1976.
4. *Air Pollution—Air Quality Implications of Alternative Fuels*. General Accounting Office, July 1991.
5. *Shell Answer Series 5—Better Mileage Tips*. Shell Oil Company, 1991.
6. *Smart Highways—An Assessment of Their Potential to Improve Travel*. General Accounting Office, May 1991.
7. Pub. L. 101-549 (Nov. 15, 1990), *1990 Clean Air Act*.
8. *Guideline on Air Quality Models*, rev. ed. Environmental Protection Agency, Research Triangle Park, N.C., 1986.
9. *Air Pollution Reliability and Adequacy of Air Quality Dispersion Models*. GAO/RCED-88-192. U.S. General Accounting Office, Aug. 1988.
10. G. J. Schewe and T. Braverman. Evaluation of CO Intersection Modeling Techniques. Presented at the 70th Annual Meeting of the Transportation Research Board, Washington, D.C., Jan. 1991.
11. J. Zamurs and R. Conway. A Comparison of Intersection Air Quality Models' Ability To Simulate Carbon Monoxide Concentrations in an Urban Area. Presented at the 70th Annual Meeting of the Transportation Research Board, Washington, D.C., Jan. 1991.
12. *Carbon Monoxide Hot Spot Guidelines Volume I: Techniques*. Environmental Protection Agency, Research Triangle Park, N.C., Aug. 1978.
13. *Carbon Monoxide Hot Spot Guidelines Volume II: Rationale*. Environmental Protection Agency, Research Triangle Park, N.C., Aug. 1978.
14. *Carbon Monoxide Hot Spot Guidelines Volume III: Workbook*. Environmental Protection Agency, Research Triangle Park, N.C., Aug. 1978.
15. *Should Drive-Thru Restaurants Offer Service-Time Guarantees?* Restaurants and Institutions, 1989.
16. *Is Drive-Thru Service Ever Fast Enough?* Restaurants and Institutions, May 1987.
17. *Trip Generation*, 5th ed., ITE, 1991.
18. J. D. Fricker and H.-S. Tsay. Drive-Up Windows, Energy, and Air Quality. In *Transportation Research Record 1092*, TRB, National Research Council, Washington, D.C., 1986.
19. J. R. Baxter and D. B Stafford. User Costs and Fuel Consumption at Drive-Through Facilities. In *Transportation Research Record 1049*, TRB, National Research Council, Washington, D.C., 1985.
20. *Air Quality Impacts of Drive-Up Facilities*. JHK & Associates, Tucson, Ariz., 1991.

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PART 4

Historic Preservation

Finland's Highway and Traffic Museum: Preserving the Nation's Transportation Legacy

PENTTI ELONIEMI AND ANTTI TALVITIE

The Road Museum in the Finnish National Road Administration (FINNRA) is described. The Road Museum is a museum that preserves culturally, socially, or technically important historical bridges, road sections, and ferries in their natural settings. The Road Museum at FINNRA celebrated its 175-year anniversary in 1974. So far, 54 road and bridge sites have been selected as museum displays. The number of sites will be increased slowly. The sites are marked on highway maps, and there is a short history on display in four languages at the location. A museum building will be erected to exhibit road construction and maintenance equipment and to feature culturally important road-related events. It is shown that keeping historically significant road sections and bridges in original form as objects for historical study is a concrete way of preserving a country's historical memory. The paper briefly relates the modest history of road transportation in Finland to her history. Finland's written history is young, and writings surviving from before the 12th century are scarce, even though people have lived there for at least 9,000 years. The history of road transportation in Finland should not be looked at in comparison with the accomplishments of ancient civilizations.

On the 175th anniversary of the Finnish National Road Administration (FINNRA) in 1974, a motion was made to create a Road Museum in Finland. Since then sections of roads and old bridges have been preserved or restored, and machines, implements used in road construction or maintenance, and historically interesting highway materials have been set aside for road museum use. The museum building will be opened in Kangasala near the city of Tampere on June 12, 1992. A plan has been made for future buildings in the area. The main idea is to create a road and traffic museum with enough room to add a car museum later.

At present the Road Museum has a small staff. In 1988 a foundation was formed to support the project financially. The Road Museum has a small permanent show in FINNRA's central administration building in Helsinki. The museum has published studies about its mission to preserve the history of road and bridge construction in Finland.

So far 34 bridges and 20 road sections have been selected as museum display sites. The number of displays will be increased gradually. The displays will be marked on highway maps and with signs along the roads on which there will be a short history of the display in four languages.

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DEVELOPMENT OF ROADS AND ROAD NETWORK IN FINLAND

"The road, be it a stony path, a dirt road or a multilane freeway, is a part of the landscape where nature and culture meet" (P. Fogelberg, poet).

Before 1100

There is a common misconception that waterways were the only prehistoric traffic routes. But, according to new research, traffic routes by land are much older than previously thought. The first roads appear to have developed from simple paths that were cleared to make traveling easier. Heavier loads were pulled on a litter or a sled. Ancient burial grounds show some signs that horses were ridden and used as beasts of burden as early as the fourth century (about 300 A.D.) in Finland.

The oldest and most famous ancient road in Finland is the Giant's Road in Pohjanmaa, a county in western Finland. The Giant's Road dates to about 600 A.D. It is a ritual road 540 m long that is paved with large stones. The road connects two hills, both of which are ancient burial grounds (Figure 1).

The oldest prehistoric road still in use is the so-called Häme Ox Road, which connects the cities Turku and Hämeenlinna (Figure 2). It dates to approximately 800 A.D. and is roughly 170 km long. Most of the road is still in use, though basically as a private road. The Ox Road kept the same basic direction from the Middle Ages up until last century. Its alignment is not the shortest path between the two cities, but because of the terrain it is the best route. During the Middle Ages the road also connected many economic and government centers. The road was built specifically for long-distance travel. Another road branches off the Ox Road at its southwest end; this road is called the Devil Road.

Swedish Period, 1100–1809

At the end of the prehistoric period, roads were beginning to be built to suit the needs of the Sweden-Finland government. Bridges were also built over some of the more important waterways. The rivers in the southwestern part of Finland were rough and full of rapids, making them difficult to travel by. Therefore, by the time of the Middle Ages, roads had

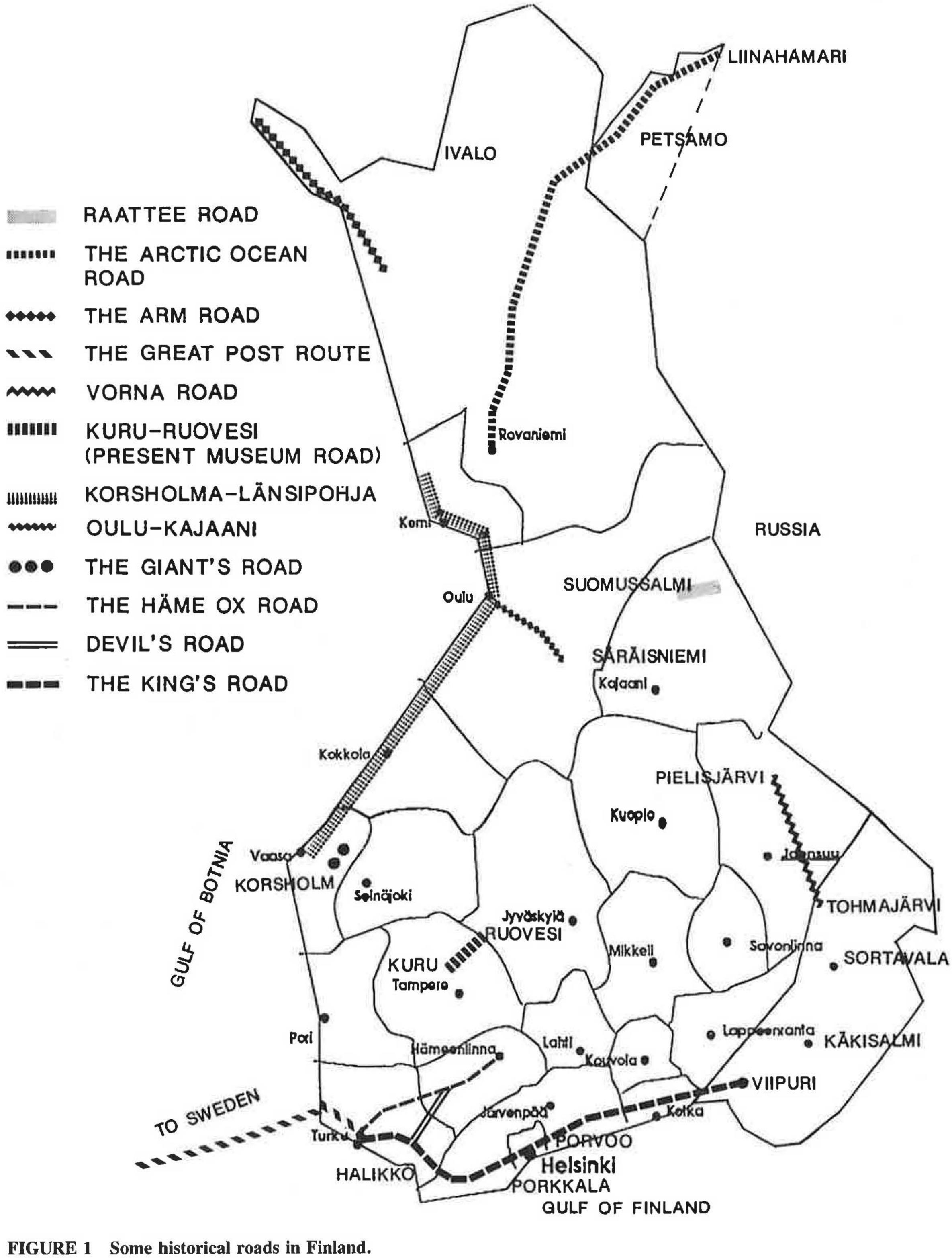

FIGURE 1 Some historical roads in Finland.

FIGURE 2 Häme Ox Road.

become more important than waterways. The roads were divided into highways and local roads (using modern terms). The medieval road network connected forts (or castles), which were the government centers. According to a list made in 1556, eight roads formed the trunk road network. These roads were used basically for government and military purposes.

The most important medieval road was the Uusimaa Coastal Road, which went from Turku along the coast all the way to Viborg, now a Russian city. The road was in use during the 14th century, but it was not firmly established until the 16th century. This road already had a great deal of traffic during the Middle Ages. A 35-km section of the road near the eastern border has been selected as a museum display site. This section of the road is also culturally important, because a noted Finnish composer, U. Klami (1900–1961), lived on it. Another section, Fagervik Road, has also been selected as a museum site.

By the end of the 16th century, at the latest, there were more roads than what was on the historic list. A good example is the route from Turku to Sweden over the Finnish archipelago. This very important route was named the Great Mail Route or the Archipelago Route. In 1638 it became Finland's first mail route.

Duke Juhana, who ruled from 1568 to 1592, made a trip by horse-drawn carriage in 1562 in returning to Turku from his honeymoon in Poland. Duke Juhana was the first of the Swedish kings to give orders, in 1587, to make the roads good enough for carriages. The long wars in which Sweden-Finland was involved in the 16th century slowed the development of the roads.

In the 17th century, when Sweden-Finland was a major power, the road network in northern, inner, and eastern Finland was expanded. The new roads were necessary because the population began spreading, and the kingdom gained more land in Karelia. It was during this period that the coastal road from Korsholma Castle to northern Sweden was built. When the Kajaani Castle was being built, it caused the need for a road from a northern city of Oulu east to Kajaani. In the beginning of the 17th century the government gave numerous orders to make roads fit for travel by carriage.

At the turn of the 17th century, there were many wars and many bad harvests; horses almost became extinct. Traffic growth retarded dramatically. During the Russian occupation in 1741–1743, the building of the Karelia Road was ordered; a section of this so-called Vorna Road has been selected as a museum site.

Toward the end of the 18th century many new roads were built in inner and eastern Finland. An example of a road from this period is the scenic Ruovesi Road in northern Häme. According to maps this road has existed since the 1790s or earlier. The 26-km road is still beautiful with all its hills and curves. The older parts of the road used to be a part of and complete a waterway system. These roads in inner Finland were primarily church roads. A four-layer stone structure has been found through archeological excavations in a swampy section of the Ruovesi Road. The unusual structure has been determined to be some sort of bridge.

Much Finnish cultural history is a part of the Ruovesi Road. A famous 19th-century Finnish painter, W. Holmberg (1830–1860), has painted many pictures about this road; Finland's national poet, J. L. Runeberg (1804–1877), wrote poems about this road when he worked as a private tutor in Ruovesi. The road has been proposed as a museum display site, but it has not been decided how much of the road will be left untouched when its reconstruction begins. Fewer than 9000 km of roads were built during the entire Swedish period. In 1807 the length of the road network was 9060 km.

Russian Period, 1809–1917

Road construction became important again when Finland became an autonomous Grand Duchy of the Russian empire in 1808. During this time, decisions about roads were made in Finland, and most of the taxes that were collected in Finland also stayed in Finland. A budget was given by the state to be used for building roads.

Because there were many famine years in the previous century, roads were built for pay in food. This was called relief work, and it sped up the construction of roads in Finland. Hunger and disease killed many relief road builders, and group graves were built along the roadside. Most of these roads were built in Lapland and northern Finland, though some roads were also built in southern Finland. Without this relief work road construction, it would have been almost impossible to connect traffic from northern Finland to the rest of the country.

The Royal Rapid Clearing Administration (RRCA) was formed toward the end of the Swedish period in 1799. After several name changes it finally became the present FINNRA. During the 1800s its offices were organized in a military fashion. At first FINNRA concentrated on waterways, but it was also responsible for government-funded road design and construction. One of the RRCA responsibilities included training foremen for general construction work.

During the early years of autonomy, reconstructing old roads was RRCA's main job. Roads were straightened, ditches were dug, and road surfaces were leveled. Following the rehabilitation work, but still in the early part of autonomy, roads that helped road traffic and traffic from inland to coastal cities were built. A boom in road construction occurred between 1826 to 1835. Altogether 1500 km of new roads were built, or about one-sixth of the length of the entire road network in 1809. This brought about a new stage to social development, especially in Pohjanmaa, the coastal area of the Gulf of Bothnia. Before this traffic had traveled along the riverside, or on the river, to the sea.

Toward the end of autonomy and with the arrival of the railroad, most of the new roads were built to reach the railroad stations. In northern Finland some long relief work roads were built. During autonomy, information about winter roads—that is, roads opened only during winter on ice or frozen ground—was available in travel guides. The marked winter roads were usually more direct and much shorter than the normal roads.

During the 1890s Finland saw its first automobile, and bus service began in 1905. The quality of the roads became quite important toward the end of the Russian period. In 1915 there were 31 190 km of roads of Finland. Thus, more than 20 000 km of roads has been built during autonomy.

Early Independence, 1918–1939

Soon after Finland gained its independence in 1917, the number of cars increased dramatically. Long-distance traveling was shifting more and more to the roads, away from railroads and waterways. This shift was, of course, hampered by the fact that most of the roads were curvy, hilly, and bumpy. In the beginning road development was slow because the transportation ministry favored railroads. In the 1920s the government was a little more favorably disposed toward automobiles, though it was thought that cars were good only for the local transportation of goods and people. Even still, the roads in southern and central Finland were upgraded and made more suitable for automobile travel in the 1920s and early 1930s.

During the early years of independence, two major plans were made by committees for the development of automobile traffic. The plans were made by the North Finland and Karelia Committee in 1928 and the Ministry of Transportation in 1933. Both plans were completed with surprising accuracy, despite the fact that the unemployment problem was still being handled as a relief work. This meant that roads were built where there were serious unemployment problems.

According to the plans of the latter committee, 29 truck highways were to be built in Finland. These roads became the basis for a strong trunk highway network. Military strategy was also taken into account in planning of these roads. One of the more interesting relief work roads was the 20-km Raattee Road (Figure 1), which goes from an eastern community of Suomusalmi to the Russian border. During the famous Winter War of 1939–1940, a Finnish division totally destroyed two Russian divisions that had reached Suomusalmi by means of this road. The Raattee Road has been chosen as a museum display.

The Arctic Ocean Road was constructed between 1916 and 1933. Because of the importance of this road to Finland at that time, the level of service of the first section of this road had to be upgraded already in 1939. This 531-km road was considered to be the most gallant road of northern Europe. It connected Rovaniemi with Petsamo's Liinahamari port. During a brief period of peace in the Second World War, the armistice from 1940 to 1941, the Arctic Ocean Road was Finland's only access point to the rest of the world. The road was built so well that it resisted extreme arctic weather; after the war it did not even experience freeze and thaw problems like most of the other roads in Lapland did. The exploitation of the resources in Barents Sea has made this road important once again.

World War II, 1940–1945

World War II, which began in 1939 for Finland, seriously interrupted the growth of traffic and the development of traffic conditions. The war put FINNRA through a new test—FINNRA was responsible for keeping the transports moving—that it passed with honors despite its small budget. The bad shape of many roads seriously affected warfare in some areas.

Most new road construction took place in northern Finland. During the war only 667 km of new roads were built. The Germans built the 157-km so-called Arm Road, which connected Finland and Norway. FINNRA was occupied mainly with maintaining and rehabilitating roads. Most of the reconstruction took place in war areas and Karelia, which Finland had captured from Russia in the beginning of the war.

During the "continuation war" (1941–1945) the road network was hit hard, especially in areas in which there was much military activity and evacuation of population. When the Germans pulled out of northern Finland, they destroyed almost 5500 km of roads. When the war was over, Finland's roads were in sad shape. During the spring thaw some large centers of population were left practically isolated from the rest of Finland. Almost the entire highway budget had to be used to rebuild the road network in northern Finland. Most of the road construction was also done there. The Germans had placed mines extensively when they left, which made the reconstruction very dangerous business: more than 30 people were killed and more than 60 were injured during the reconstruction.

ROAD SERVICES

The inn system dates to the Middle Ages, when churches took care of the inns. According to the 1347 land law of Mauno Eerikinpoika (Mauno Erik's son, who ruled 1319–1374), there had to be taverns and inns along public roads. The distance between inns was 10 to 20 km. In 1649 came an additional order to build separate rooms for royalty. Until now the houses had been peasants' houses, and people from higher classes usually stayed in churches and the like when they traveled. The innkeeper was to have good bedding, a decent kitchen, and plenty of horses and equipment. He could usually hire local peasants to be drivers.

The inn system functioned well and was in good order at the end of the Swedish period in 1808. Steady improvements were made. At the beginning of the 18th century Finland switched to a stagecoach system. This meant that there had to be a certain number of stage horses and drivers ready every day, even if there were no travelers. After 1766 inns had to keep a diary into which travelers wrote their names and any other notes. This diary was then sent monthly to the governor of the land for inspection. Inns were built at the same time a road was built. A few old inns serve currently as museum cafes, a typical Arctic Ocean Road Inn in Nautsi is also still in existence.

The transport service hit its peak in 1916–1917, but the automobile signaled the end of an era. According to the transport law of 1918, the new, independent state took over the system and begin to use automobiles. The last information on the transport service is from 1944. The old transport law was abolished as late as 1955.

ADMINISTRATIVE DEVELOPMENT OF ROAD MAINTENANCE

According to Mauno Eerikinpoika's Land Law of 1347, the construction and maintenance of roads was the responsibility of the land owner. Long roads were often the responsibility of communities. During the Middle Ages circuit judges inspected roads. In the 16th century this job was transferred to special boards, and a special institution called bridge ombudsman was created in the 1800s. In 1892 road maintenance was given to the sheriff's office, which was given the "road master" as a technical helper in 1920. Even today the person responsible of a FINNRA's maintenance area is called road master.

In 1921 all highway (state and county roads) maintenance was transferred to FINNRA. The upkeep of local roads was left to their users. Because of insufficient resources FINNRA often maintained only the more important highways. All other roads were left to be maintained by county governments, which, in reality, maintained the roads through the boards of rural counties. It wasn't until 1940 that the maintenance of all highways was taken over by FINNRA in fact.

TECHNOLOGICAL DEVELOPMENT OF ROAD CONSTRUCTION AND MAINTENANCE

Road Alignment

According to instructions given in 1619, roads were to be built along ridges and other flat but strong land, and around larger rocks and hills. Being level and not having any hills were the two most important characteristics of roads during the 1800s. The maximum grade allowed was 10 degrees. New instructions were given in 1910, according to which villages, houses, and waterways were to be considered when planning road alignment. Consequently, roads were built heading straight to the narrowest point of a waterway, where the bridge was built perpendicular to it. Depending on the class of the road and the environment, the maximum grade was supposed to be 5 to 9 degrees. The minimum horizontal curve radius was set at 50 m.

During the 1930s driving speeds began to increase, and the minimum radius was increased to 250 to 500 m; in the 1940s the suggested radius was further increased to 1000 to 2000 m. Aerodynamics and sight distances have been the basis for highway planning since the 1920s.

Road Widths

According to the previously cited land law, highways and bridges were to be 5.9 m wide and local roads were to be between 1.8 and 3 m wide. At the beginning of the New Age most roads in Finland were 3 m wide. FINNRA gave its first general instructions in 1917, according to which the width of a first-class highway had to be at least 5.4 m and of a local road, 3.0 m. Bridges were to be built the same width as the roads.

Road Substructure

On ancient roads, swampy or soft ground was crossed using long wooden planks. During the Middle Ages, twigs were used to make the ground more firm. Later, stones were used—on the Ruovesi Road, for example. During the 1930s the most common way to increase a road's bearing capacity was to lay decks made of logs.

Road Superstructure

Plain dirt was first used as both the superstructure and the wearing surface. According to a law passed in 1734, public roads had to be raised and built in such a manner that water did not gather on the surface. The cross section had to be ridgelike, sloping. When traffic became faster and heavier, the dimensioning of the different structural layers became more accurate: different structural layers were adopted for use in 1910; dimensioning of layers has been done on the basis of the dynamic strain of wheels since the 1920s; and elasticity of earth materials was taken into account in the early 1930s.

Technical advice was also given for materials of structural layers. At first the insulation layer against thaw was made of organic materials, but since the 1940s rough, homogeneous sand has been used. During the 1920s the subbase layer was made of materials that could freeze in the winter; during the 1940s that was changed and nonfreezing materials were ordered to be used. In the 1920s a bearing course was built on trunk roads.

Cars have had, of course, the greatest effect on the development of wearing surfaces and pavements. In 1926 a short test road was made with a concrete pavement. At the end of the 1920s bitumen binding agents were being used; in the 1930s Finland switched to light asphalt (a cold-mix cut back asphalt) pavement.

Road Construction

As late as the 1930s the road construction equipment consisted of manual lift equipment, block and pulley, water pumps, and cement mixers. Most freighting was done with horse-drawn carriages, even though trucks were already used for moving earth in the 1920s.

Meticulous records were kept on input usage showing the man, horse, and car labor hours used in building roads. It wasn't until the end of the 1930s that trucks finally replaced horses. Steam rollers were put into use toward the end of the 1800s. Pneumatic drills became common in the 1930s.

Water was poured on heated rocks to split them. The use of gun powder on rocks dates to at least 1652. Dynamite was not used until the end of the 19th century; it was replaced by safer explosive materials in the 1920s.

Summer Maintenance

Road maintenance was done as needed early on, but the initiation of the "relay" service and the inn system led to regularly performed road maintenance. Roads were to be inspected twice a year, according to land laws dating to 1347 and 1442. Roads and bridges were to be inspected regularly starting in 1635.

A law passed in 1883 gave general instructions for the maintenance of public roads. An instruction book from 1920 said that spring and fall maintenance was still enough but that damage that affected traffic had to be repaired immediately. Special road guards were hired for this job. Maintenance was handled as a contract job, according to laws passed in 1918 and 1927. The contractors were usually persons who formerly were responsible for "roadkeep" so the quality of the work was the best possible. The road law of 1927 made it possible to give up the contract work system, but it was not until 1948 that road maintenance was transferred entirely to the road administration.

The growth of automobile traffic caused the dirt roads to become washboard very quickly. To prevent this, the wearing surface was usually made of clay-gravel mixture, which was bound with calcium chloride. The drawback of this new road surface was that it caused the roads to freeze; still a major problem in gravel road maintenance. Trucks were used in road maintenance already during the 1920s. In 1925 domestically manufactured road graders were taken into use. Before that roads were planned with horse drawn wooden or metal planes.

Winter Maintenance

Finland has had several types of winter roads, which were cleared and marked, since the beginning of time. "Normal" roads were not used for winter traffic until about the 17th century. In 1786 a law was passed stating that the person responsible for "roadkeep" had to also keep roads open during the winter.

A new stage in the development of winter maintenance began in 1920s. In 1925 some highways were ordered kept open for auto traffic year-round. A 1927 law made it mandatory to keep all the highways open during the winter. Snow fences were used to protect the roads from snow drifts. Manufacturing of snowplows was started in 1926. With plows developed in Finland and with powerful enough trucks it was possible to throw the snow off from the road. Scientific evaluation of snow plows evolved also; the first test plowing was done in 1924.

In 1939 a third of the highway network was kept open with motor vehicles and the rest by horses. When necessary some important maintenance connections were kept open with snow scooters as in the case of the Pallastunturi hotel. Keeping all public roads open for autos during the winter became standard procedure during the 1940s.

Keeping highway lanes in good condition has been central to winter maintenance from the very beginning. During the first few years of the automobile, a 10- to 12-cm layer of snow was left on roads for sleds, but this was cut down to about 3 cm in the 1940s. In addition to leveling the road surface, sanding hills was started in the 1930s. At first sanding was done by shovel from a truck, but during the war sanding machines replaced manpower.

BRIDGE DEVELOPMENT

"Two types of constructions, temples and bridges, show the technical and artistic skill of man from the beginning of time until this very day" (B. Kivisalo, professor of bridge design, Helsinki).

Bridges in Finland are short, because of the terrain and shallow bodies of water. The largest bridge span in Finland is only 220 m long, and the longest bridge is only about 800 m long. On the other hand, reasons of appearance have been pushing the bridge length upward.

Swedish Period, 1100–1808

The first regulations for building bridges were given in the Land Law of 1347. The more important bridges from the middle ages were connected with the castles and forts but also elsewhere there were important bridges. All of these bridges were made of wood. Bridges for carriage wagons were made by laying a few logs across the water, and then a few smaller pieces of wood across these. This is how modern composite timber deck bridges got their start.

In Pohjanmaa, where the ground was soft, piers and supports were log-formed stone-filled foundations. In order to get longer spans, king and queen posts and A-frame supports were developed. A man from Pohjanmaa was said to have built a suspension bridge over a river in 1682. A king-post bridge was most likely what was built. The road museum has selected a pile girder, a king-post, and several A-frame bridges as museum sites.

During the 1600s bridges were built over the great rivers of Pohjanmaa. The floods are still a problem there today, but back in those days the bridges had to be taken down during the flood season for them not to be ruined. Because of this the bridges were mostly pile girder bridges. They were taken apart before the ice on the rivers melted and rebuilt after the flood season was over. During the 1700s floating bridges attached to the shore were often built.

Finnish bridge builders did not learn how to make stone arches until a large sea fortress in front of Helsinki was built. Arched bridges were made out of unshaped stones of different size, and without mortar. The bridge experts of that time period were officers who worked out of a special office. The oldest of these bridges is the Espoo mansion bridge, built on the Uusimaa Coastal Road in 1777, now a museum site.

Russian Period, 1809–1917

An intendent's office was created to design large bridges to replace the old ones destroyed by the 1808 war. The persons or agencies responsible for road construction did not always use these services; instead, they often had their bridges made by self-taught bridge master builders. A major problem with that was that usually the foundations were built poorly. In the beginning of the 20th century, foreign construction companies arrived in Finland.

Timber was the main material used in bridge construction, even though stone pier supports became very common. The use of A-frame and king-post bridges increased. Finland's oldest remaining original wood bridge, the Eteläkylä Big Bridge (a museum site), was built in 1837. Its span measurements are 18 × 18 × 19 × 18.5 m.

During the 1800s two large arch bridges were built, whose arches were made out of bent boards. The older of the two, with span measurements 37.5 × 45 × 37.5 m, was finished in 1896. In the early 1900s, the Kuorikoski family, which was well known for building churches, built covered bridges in Pohjanmaa. The bearing structure of these bridges was a cross between A-frame and truss type structures. The church builder's bridges were based on intuitive technical skills. No bridge drawings were made—the master builder *explained* how each part of the job was to be done.

During the late 1800s, small stone arch bridges became common because of cement mortar. There were stone master builders all over Finland. The best-looking stone arch bridge is the Aunes bridge in Tampere, which was built in 1899, it has a span width of 19 m. Stone beam techniques were used in Pohjanmaa. In 1870 the 8-m Möykky bridge, planned by a vicar, was built. It resembles an A-frame bridge.

Finland's first cast iron bridges were made during the construction of the Saimaa Canal in 1853. Two of these rolling bridges are still standing in the canal zone, rented by Finland from the former Soviet Union. With the railroad construction, Finland got its first "cooked iron" bridges. The Koria Bridge, which was built in England and meant for the Helsinki–St. Petersburg track, was taken into use as a road bridge in 1924.

Carpenters from northern Finland had been working in Norway near the end of the century where they learned to build suspension bridges. The oldest suspension bridge is in Pohjanmaa, built in 1909 with a span width of 81.45 m. It took the village blacksmith a whole year to make the steel parts of the bridge. The bridge has been repaired, and is now a museum bridge.

Finland's first cement factory was founded in 1877. Shortly afterwards the first concrete bridge, an unreinforced concrete arch bridge over the harbor railroad tracks, was built. The oldest reinforced concrete bridge was built in 1911 in southern Finland. It is an arch bridge with a stiffened deck and has a span of 22.5 m.

Early Independence, 1918–1939

At first, most bridge construction was aimed at repairing bridges that were destroyed during the war. New bridges were not built until 1924. During the 1920s there was tough competition between choosing the timber and reinforced concrete bridges. Not until the end of the 1930s did concrete gain the upper hand, and the use of timber was almost discontinued. The number of stone bridges grew until the mid-1930s. Bridges with stone piers and supports were also considered stone bridges.

Many reinforced concrete bridges were built during the beginning years of independence. An arch bridge over the Kokemäki river was built in 1918, using timber truss bridge supports left over from the war. Its free spans measured 24.9 × 24.7 × 23.8 m. The Savukoski Bridge was built in 1926–1927; it is the first cantilevered, one jointed arch bridge in the world. During the construction of the bridge a rope lift was made to help the transportation of building materials and scaffoldings. Both of these bridges are road museum sites. The Hessund bridge on the Archipelago Road was built in 1937. Presumably it is still the largest stiffened reinforced concrete arch bridge of that type in the world. The main arch is 81.6 m long. No scaffoldings were needed to build this bridge.

World War II, 1940–1945

During the war, bridge construction was almost at a standstill. When the Germans pulled out of northern Finland, they destroyed almost 1,000 bridges. Because there was a shortage of building materials, and only timber was available, they had to be replaced with temporary bridges. In order to replace large steel and reinforced concrete bridges, a "nailed board beam" became a standard, with span measurements of 8 to 24 m. Supports were made as elements in five prefab factories. One of these is a museum site.

Timber also had to be used elsewhere as bridge material. As a curiosity, it may be mentioned that the Russians built the 139-m timber bridge in Porkkala, an area that they rented from Finland at that time. The bridge is also a museum site.

FERRY DEVELOPMENT

Finland is known as the land of thousands of lakes, and ferries have been used since the 1500s. The rivers of Pohjanmaa got their ferries around the 1600s. The ferryman got his salary from the persons or instances responsible for the ferry, and he also had the right to collect a ferry fee.

The development of ferries in Finland has gone through many different stages. When horse traffic was common, ferries were like large rowboats. In 1924 a new type of ferry, consisting of steel cylinders with wooden decks on top of them, was built. They moved by pulling the ferry line with wooden dowels and could carry up to 6 to 12 tons. One of these has been chosen as a museum ferry. In 1931 this kind of ferry was given a motorized winch and a few years later 12-ton propeller ferries came into use. The propeller powered ferries were useful during the winter, because they kept the route open much longer. Ferries made on pontoons covered with a deck and powered by a PT-boat motor replaced some of the bigger bridges which were destroyed in northern Finland during the war. They crossed large, strong rivers without the use of a cable.

CONCLUSION

"Historia vitae magistra." History is life's teacher. History is a show of how something has evolved through the ages, its development and associated research. Everything which makes development clearer is historical. Age is not a factor. The written history of Finland is relatively young; nothing written before the middle of the 12th century has survived. But according to archeological diggings, there has been life in Finland for at least 9,000 years.

Even though road and bridge building techniques and skills are thousands of years old, their history in Finland is short and modest; this is especially true for bridges. Diggings done near roads have not been of very much help. This is because only wood was used until the end of the 18th century to build bridges, although stone churches were built in Finland during the 13th century. The history of road and bridge design, construction and maintenance in Finland should not be looked at by comparing it to the accomplishments of ancient civilizations.

Historically significant road sections and bridges can and should be kept in original form if possible; they should be objects of historical studies. This is a concrete way of preserving a country's historical memory—similar to the eagles, which are said to have led the Hungarians from the Ural mountains to present-day Hungary, in the pylons of old Donau bridges in Budapest. The road museum also shows future generations the history in its social setting and how roads have evolved from little paths in the woods to freeways, how bridges have evolved from chopped down trees to suspension bridges, and how highways have throughout the ages been an integral part of a country's social development.

The road museum also gives a new, humanistic dimension to the highway agency. It relates the highways and the highway agency directly with the history of the country. Appreciation of history and culture enables the highway agency to face today's issues and accomplish today's tasks better because it is seen to stand also for other goals besides relieving congestion and improving traffic safety and to represent also other values than the narrow economic issues and interest groups of today.

ACKNOWLEDGMENT

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PART 5

Noise

Field Evaluation of Acoustical Performance of Parallel Highway Noise Barriers in California

RUDOLF W. HENDRIKS

The effect of multiple reflections between two parallel highway noise barriers on the acoustical performance of each of the barriers has been a subject of considerable controversy. Mathematical and scale modeling predict possible large reductions (degradations) in the effectiveness of noise barriers with smooth, hard surfaces (such as masonry or concrete) due to parallel configurations. However, noise measurements under conditions of actual highway traffic, typical barrier heights, and separations have yet to confirm the large degradations. The methods and results of a parallel noise barrier research project performed by the California Department of Transportation are presented. Field measurements of noise, traffic, and meteorology were made in three stages: before barrier construction, after construction of the near barrier, and after construction of the barrier on the opposite side of a highway. The selected site was typical of many parallel barrier configurations in California. More than 100 simultaneous, A-weighted, 15-min energy-averaged noise levels were measured at 11 microphones from 5 to 23 ft high and 15 to 200 ft behind the near barrier. The noise data were matched by crosswind vector wind velocities and normalized for differences in traffic. Analysis results showed degradations of 0 to 1.9 dBA, independent of wind. Vector wind velocities of −3 to +11 mph caused variations in noise levels of up to 9 dBA at 200 ft behind the near barrier.

This paper presents the results of a FHWA-funded research project entitled *Field Evaluation of Acoustical Performance of Parallel Highway Noise Barriers Along Route 99 in Sacramento, California*. The project investigated the effects of multiple noise reflections between two parallel masonry sound walls on the acoustical performance of one of the sound walls. The study was performed by the California Department of Transportation (Caltrans), Division of New Technology, Materials and Research (NT, M&R).

BACKGROUND

By far the most common method of mitigating traffic noise in California is to construct noise barriers between highways and critical receivers. About 330 mi of noise barriers have been built so far in California. These barriers intercept the direct noise path between noise sources and receivers and provide adequate noise attenuation when properly designed.

One of the consequences of interrupting the noise path with a barrier is the possibility of reflecting noise to the opposite side of a highway. The amount of reflected noise depends on the surface of the barrier. A hard, smooth surface such as a concrete wall will reflect almost all of the noise that strikes it. Although other materials may be used for noise barriers, the vast majority of Caltrans barriers are made of noise-reflective masonry blocks. Reflections off such barriers may be classified as single or multiple reflections depending on whether a single barrier is present on one side of the highway or parallel barriers are present on both sides.

Single-barrier reflections can theoretically increase noise levels by 3 dBA on the unprotected opposite side of a highway. This maximum value is associated with a 100 percent increase in acoustical energy at the receiver, due to a perfect specular reflection of the noise source. In rare cases in which direct noise is shielded and reflected noise is not, noise increases may be more than 3 dBA.

In practice, however, single reflections off barrier surfaces rarely increase noise levels more than 1 to 2 dBA on the opposite side of the highway; often the increases are not even measurable (*1*).

Of greater interest are multiple reflections between parallel sound walls and their potential negative effects on the performance of each individual sound wall. Mathematical calculations and measurements in laboratories indicate substantial degradations in parallel barrier field insertion losses. Degradations measured under real-world conditions, however, are generally considerably less (*2*).

The reason most often cited for this lack of agreement is the difficulty in finding suitable sites for studying multiple reflections under actual traffic conditions. A parallel barrier study should ideally include extensive measurements before barrier construction, after construction of one barrier, and after construction of both barriers. Such a three-staged approach requires careful scheduling of construction activities, allowing adequate time for noise measurements under free-flowing traffic and a variety of meteorological conditions. All pertinent variables must be meticulously documented during the measurements.

In 1987 and 1988, Caltrans District 7 initiated a demonstration project for retrofitting one of two existing parallel masonry noise barriers with noise-absorptive panels along Route I-405 in Los Angeles. The project was done in response to noise complaints by some residents of the nearby community of Brentwood. The residents specifically addressed reflective noise as the major cause of a perceived noise increase. A thoroughly documented filed study involving before and after noise measurements, meteorological observations, and traffic counts revealed that the absorptive panels reduced the noise

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levels about 1 dBA. Treating the opposite wall with absorptive material in essence simulated a "no wall" condition, so it was inferred that the opposite wall had a negligible effect on the performance of the near wall (*3*).

The study also showed a considerable influence of wind speed and direction on the noise readings within 250 ft of the highway. The data clearly showed that noise level differences of 4 dBA could be attributed to relatively minor wind shifts (*3*). This made it necessary to study before and after measurements for the same wind conditions. Measurements performed during various wind speeds and directions made this possible.

The geometry of the site studied was very complicated and considerably less than ideal for application to other site locations. A site that was representative of many parallel barrier locations needed to be studied.

An excellent site was found in 1988. Two parallel masonry sound walls were to be constructed along State Route 99 in south Sacramento. The site was fairly typical of parallel barrier locations throughout California.

With cooperation of the design engineers, special provisions directed the contractor to build the wall nearest to the site first, cease construction operations for 1 week, construct the opposite (far) wall, and once again cease operations for 1 week. This sequence in construction would enable NT, M&R personnel to measure the acoustical performance (insertion loss) of the near wall without and with the opposite (far) sound wall. The difference between the two would give an indication of insertion loss degradation due to multiple reflections between the parallel walls.

OBJECTIVES

The objectives of this research project were to

1. Measure the reduction in acoustical performance of a noise barrier on one side of a highway due to multiple reflections caused by the presence of another barrier on the opposite side of the highway—and do so at a site that incorporates a barrier configuration and geometry that is representative of most parallel barrier locations in California; and
2. Develop recommendations and guidelines ("do nothing" or mitigate) depending on the findings of this research project and define the parallel barrier projects to which they apply.

LITERATURE SEARCH

A literature search was conducted for previous research on parallel barriers (*4–15*).

Of particular interest was a recent report on measurements behind two experimental parallel noise barriers at Dulles Airport, near Washington, D.C. Using a combination of noise-absorptive material and tilting walls, the researchers reported barrier insertion loss improvements of 2 to 6 dBA over the reflective barrier configuration (*14*). However, the walls were separated by a distance of 87 ft, far less than the typical separation of parallel noise barriers along California highways.

WORK PLAN AND APPROACH

Experimental Design

The measurements, criteria, and analysis methods used in this project were generally consistent with ANSI S12.8 (1987). The Los Angeles I-405 data pointed out the importance of matching noise measurements by crosswind vector component wind speed ranges (*3*). The data also suggested that the wind velocity criteria set forth in ANSIS12.8 (1987) are too lenient. The standard recommends a maximum allowable range of 2.4 m/sec (5.4 mph) in average velocity as an equivalent crosswind criterion. In this study, the range was held to only 2.0 mph for the purposes of establishing atmospheric equivalency for comparing noise measurements.

In this study the measured crosswind components spanned a range of -2.9 to $+11.3$ mph. A negative crosswind blows from noise receiver to noise source; a positive wind blows from source to receiver. The winds were measured at a reference height of 20 ft, about 300 ft behind the near noise barrier. The purpose of the wind measurements was to establish atmospheric equivalency throughout the three stages and not necessarily to investigate atmospheric effects in detail. Hence, one height was deemed sufficient for comparative measurements during the study periods.

Extensive noise, traffic, and meteorological measurements were designed for the following three stages of the project:

1. Before construction,
2. After construction of the near barrier, and
3. After construction of near and far barriers.

Noise measurements for Stages 1, 2, and 3 were normalized for differences in traffic via a primary control microphone in a location that was not influenced acoustically by the sound walls. This microphone was about 0.8 mi south of the site, at the same distance and height as a secondary control microphone at the site. The secondary control microphone was assumed to be influenced by reflections off the far sound wall on the opposite side of the highway during Stage 3 measurements.

In this study, the term "control microphone" is synonymous with "reference microphone," as defined by ANSI S12.8 (1987).

After the data were normalized, comparisons of Stage 1 and 2 data within the same meteorological regimes gave a measure of near-wall insertion loss (reduction of noise levels due to inserting the wall between the freeway and the receivers) at each microphone.

Comparisons of Stage 2 and 3 measurements yielded a measure of insertion loss degradation, that is, reduction of near-wall performance due to the placement of a reflective wall on the opposite side of the freeway.

Noise Measurement Site and Microphone Locations

The noise measurement site was located along the east side of Route 99 between Florin Road and Mack Road at Trailhead Park, in south Sacramento. The highway consisted of two northbound (NB) and two southbound (SB) lanes during the three measurement stages. The lane profiles were approximately 4 ft above the average park ground elevation near the site.

Eleven microphones (mics) were used simultaneously: 10 at the Trailhead Park test site and 1 at the primary control location roughly 4,200 ft southerly along Route 99, as shown in Figures 1 and 2. The mics were designated C2 and Numbers 1 through 9 (Figure 1), and C1 (Figure 2). Figure 3 shows a cross section perpendicular to Route 99, along the mic line at Trailhead Park, with barrier locations and mic heights and distances.

Mic C1 was placed in the same three-dimensional position relative to the edge of traveled way (ETW) as Mic C2. Because of its location, noise levels measured at C1 were unaffected by the presence of sound walls during the Stage 2 and 3 measurements. Noise levels measured during Stage 3 at C2, however, were affected by reflections from the far sound wall on the opposite side of the freeway. They were presumed to be unaffected by the presence of the near wall.

Mic C1 and C2 were primary and secondary control mics, respectively. Both measured the Route 99 traffic noise levels at the same height (15 ft) above and same distance (31 ft) from the ETW. This height and distance corresponds with a position of 5 ft over the top of the near noise barrier at its intersection with the mic line at Trailhead Park.

Because of the 4,200-ft separation between Mics C1 and C2, the vehicles passing both locations would be different at any one instant of time. However, because of time-averaging of traffic and noise levels, these differences were expected to be small and mostly random, with perhaps a small systematic difference due to local site and pavement conditions. Mic C1, for instance, was near an on-ramp on which accelerating vehicles, passing closer to Mic C1 than the mainline traffic, could have introduced a small systematic difference between C1 and C2. Although the C1 location was less than ideal, no better locations were available. Average relationships between the noise levels measured at C1 and C2 during each stage could be used in the source normalization process necessary for stage comparisons. Mics 1 through 9 positions were selected to measure within barrier-shielded as well as unshielded zones.

Instrumentation

Noise

All sound-level meters (SLMs) used in this project conformed to ANSI S1.4 1971 (R1976) Type 1 requirements. Mics 1 through 9 were mounted with test-tube clamps to three telescoping aluminum swimming-pool-net poles (maximum length 23.5 ft) secured with adjustable guy lines. After the near barrier was constructed, Mic C2 was mounted on a 5-ft-tall removable bracket that slipped over the top of the 8-in.-thick masonry sound wall.

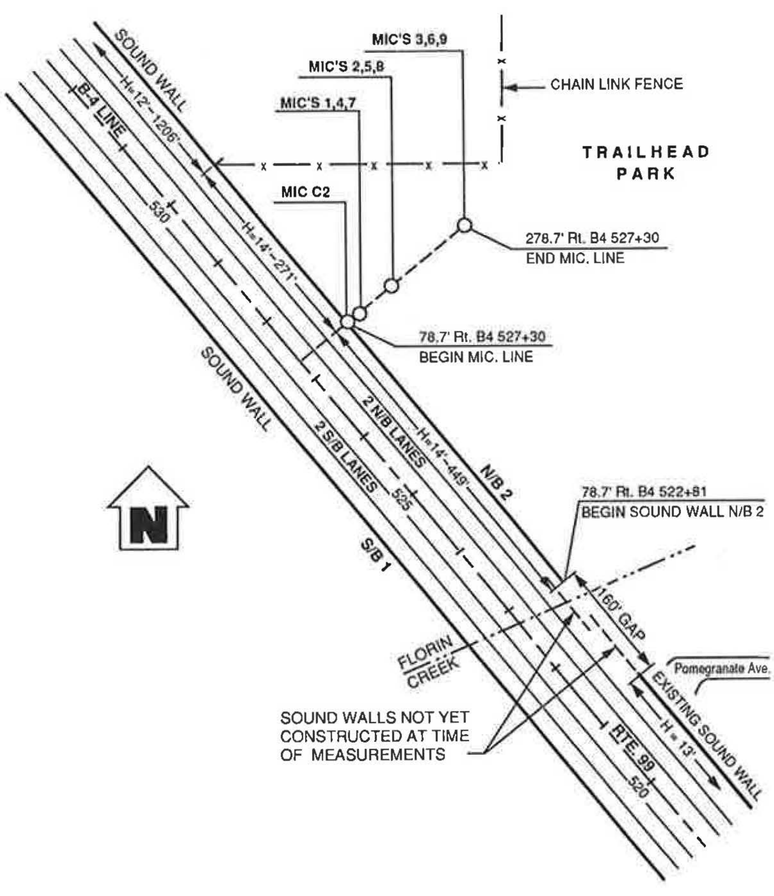

FIGURE 1 Site location, Mics C2 and 1 through 9.

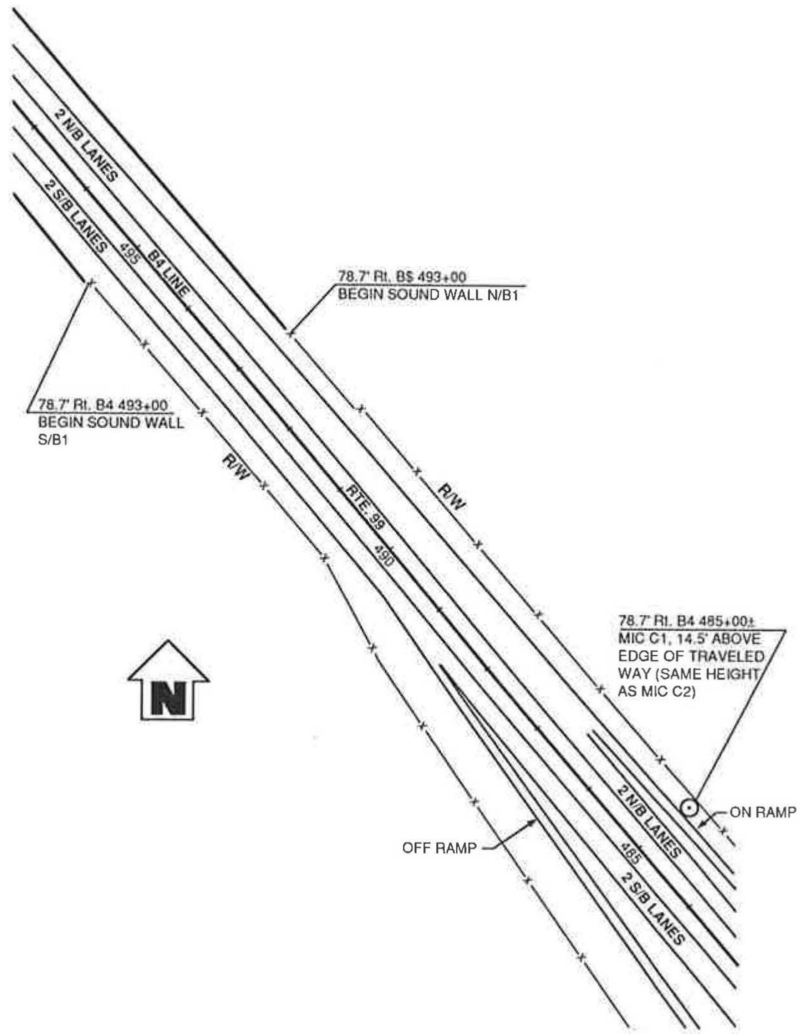

FIGURE 2 Site location, Mic C1.

The mics and preamplifiers were attached to SLMs by way of extension cables. The DC outputs of the SLMs were connected via long cables to specific channels on a 16-channel custom-made data logger inside a van. The data logger processed the signals into various descriptors such as Leq, L10, and L50 for each mic.

Mic C1 was mounted via a test tube clamp to a telescoping warning sign stand, extended to the same height as Mic C2, relative to the pavement. The display of the SLM at Mic C1 was read and recorded by the instrument operator.

Each mic–SLM–data logger system was calibrated before and after each day's measurement series. All systems (including the self-contained measuring system at Mic C1) were calibrated with the same master calibrator throughout all measuring stages.

Meteorology

Meteorological observations were taken at the van approximately 300 ft behind the near barrier. Wind speed and direction were taken at a height of approximately 20 ft. The data were fed into two channels of a data logger with a sampling rate of one per second in the van. The data logger calculated 1-min averages of wind speed and direction. These averages were later converted to 15-min "resultant winds" and crosswind vectors, coinciding with the noise measurement periods.

Air temperature and humidity were read from a combined thermometer and humidity indicator at the beginning and end of each 15-min measuring period. The two readings for each were recorded and later averaged.

The sky condition was observed at the beginning of each 15-min measurement period and classified as clear, partly cloudy, or overcast.

Traffic

Traffic was videotaped during the noise measurements on the Mack Road overcrossing (in northerly direction). Vehicles were later counted from the tapes and classified in three vehicle groups: heavy trucks, medium trucks, and automobiles (*16*). Vehicles were also grouped by direction (NB or SB). Traffic speeds were taken randomly at the same location with a radar gun.

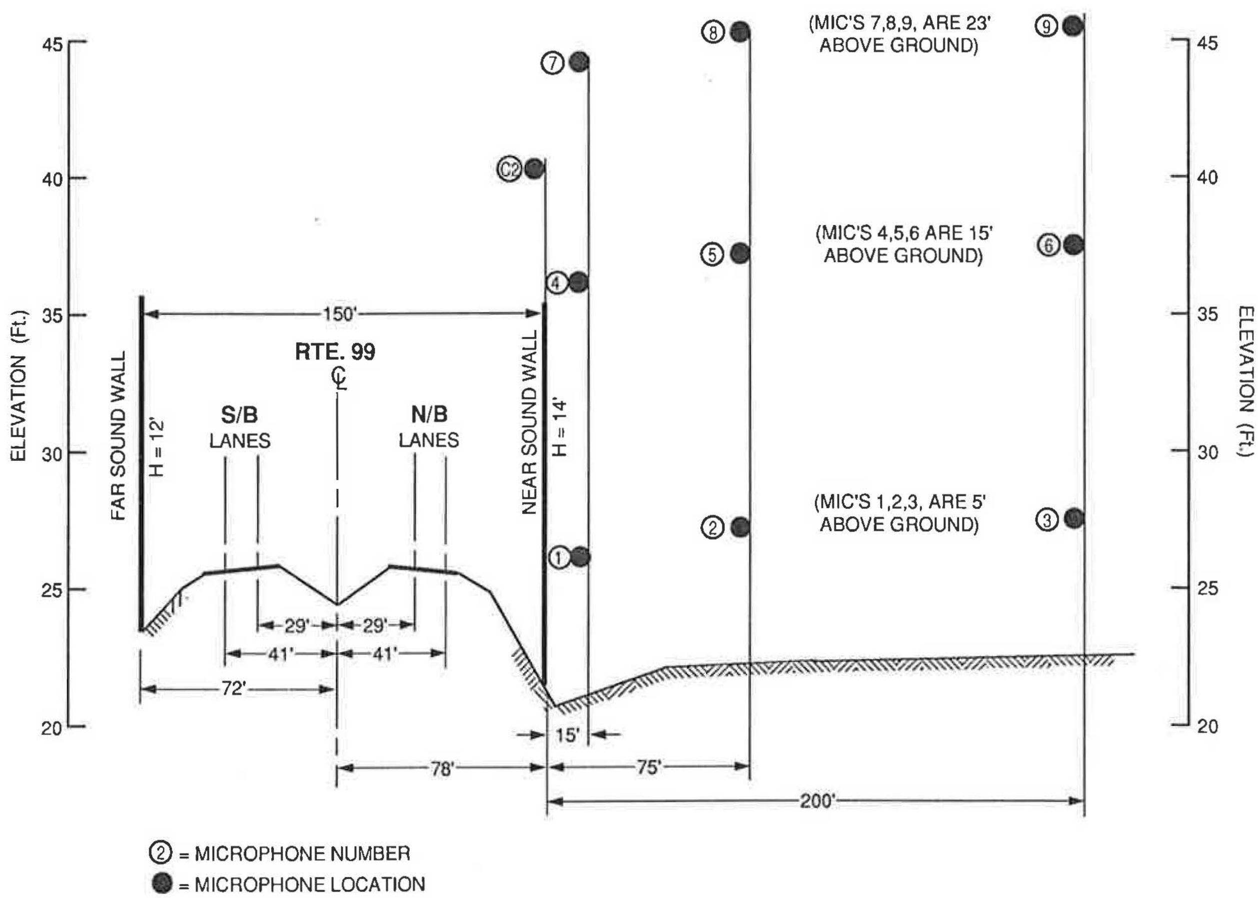

FIGURE 3 Cross section at Mics C2 and 1.

MEASUREMENTS

With few exceptions, all noise measurements were made during weekdays, between 10:00 a.m. and 3:00 p.m., under free-flowing traffic conditions. The noise descriptor used throughout the study was the Leq(15 min), in units of decibels on the A-weighted scale (dBA).

A total of 105 uncontaminated measurement runs were made: 27 during Stage 1, 45 during Stage 2, and 33 during Stage 3. Each run consisted of the 11 simultaneous noise levels recorded by the mics shown in Figures 1 through 3.

Spot checks in the park, away from the freeway and other activities, indicated maximum ambient Leq noise levels of 49 dBA. Since the minimum Leq(15min) measured during any stage was 58.9 dBA, none of the noise measurements was judged to be contaminated by more than 0.5 dBA.

DATA ANALYSIS METHODS

Normalization of Data

The Stage 1, 2, and 3 noise measurement data were normalized to account for variations in traffic. The differences between the primary and secondary control mics (C1 and C2) and the differences between Mic C2 and Mics 1 through 9 were calculated and compared for the three stages. Dealing with the differences had the effect of normalizing the data for traffic variations (*3*).

Relationships Between Control Mics by Stage

The next step of normalization was to determine the mean difference between Mics C1 and C2 for each stage. The field data showed the independence of the C1 minus C2 differences with crosswind velocities in all three stages. The differences could therefore be grouped by stage without concern for wind speed and direction. The differences are shown in Figure 4. Statistical *t*-tests (*17*) revealed that the means were statistically significant at a level of 0.05.

Calculation of Insertion Losses and Degradations

Once the average relationship between the two control mics was established for each stage, the Stage 2 and 3 insertion losses and the Stage 3 insertion loss degradations were calculated for each mic location and for each wind class. The calculation methods are shown in Figure 5.

ANALYSIS RESULTS

Insertion Losses and Degradations

The mean insertion losses for all wind classes are shown in Figure 6, and the mean insertion loss degradation due to the opposite wall are shown in Figure 7. Although the insertion

STAGE 1 (BEFORE BARRIERS)	STAGE 2 (NEAR BARRIER)
Mean Diff. = +0.14 dBA Standard Deviation = 0.29 No. of Measurements = 27	Mean Diff. = +0.35 dBA Standard Deviation = 0.23 No. of Measurements = 38

Difference of Mean Differences = -0.21 dBA

Results of "t" Test, Significance Level = 0.05:

Calculated "t" = -3.25
Critical "t" = -1.67 (one-tailed)

CONCLUSION: The Mic C1 - Mic C2 differences of Stage 1 and Stage 2 are statistically different by 0.2 dBA.

STAGE 2 (NEAR BARRIER)	STAGE 3 (BOTH BARRIERS)
Mean Diff. = +0.35 dBA Standard Deviation = 0.23 No. of Measurements = 38	Mean Diff. = -0.29 dBA Standard Deviation = 0.25 No. of Measurements = 33

Difference of Mean Differences = +0.64 dBA

Results of "t" Test, Significance Level = 0.05:

Calculated "t" = 11.23
Critical "t" = 1.67 (one-tailed)

CONCLUSION: The Mic C1 - Mic C 2 differences of Stage 2 and Stage 3 are statistically different by 0.6 dBA. It appears that this difference was caused by multiple reflections between the near barrier and the opposite (far) barrier, i.e. the Stage 3 noise levels increased by 0.6 dBA at Mic C2, compared to the same levels of Stage 2.

FIGURE 4 Analysis of Mic 1 – Mic 2 differences by stage.

I. MIC C1 - MIC C2 ADJUSTMENTS BY STAGE

A. Stage 1 (Before Barriers):
Mean Difference Mic C1 - Mic C2 = +0.14 dBA

B. Stage 2 (Near Barrier):
Mean Difference Mic C1 - Mic C2 = +0.35 dBA
Adjustment 1 at Mic C2, Stage 2 = +0.2 dBA

C. Stage 3 (Both Barriers):
Mean Difference Mic C1 - Mic C2 = -0.29 dBA
(Noise Level at Mic C2 increased 0.64 dBA during Stage 3 compared to Stage 2)
Adjustment 2 at Mic C2, Stage 3 = -0.6 dBA

II. STAGE 2 INSERTION LOSS CALCULATION

Mean $IL_{2(i,j)}$ =

= Mean $(C2\text{-}i)_{2(j)}$ - Mean $(C2\text{-}i)_{1(j)}$ + Adj.1

III. STAGE 3 INSERTION LOSS CALCULATION

Mean $IL_{3(i,j)}$ =

= Mean $(C2\text{-}i)_{3(j)}$ - Mean $(C2\text{-}i)_{1(j)}$ + Adj.1 + Adj.2

IV. INSERTION LOSS DEGRADATION

Mean $Deg_{(i,j)}$ =

= Mean $IL_{2(i,j)}$ - Mean $IL_{3(i,j)}$=

= Mean $(C2\text{-}i)_{2(j)}$ - Mean $(C2\text{-}i)_{3(j)}$ + Adj. 2

WHERE:		
	IL	= Insertion Loss
	Deg	= Degradation
	C1 and C2	= Noise Levels at Ref.Mic's C1 and C2
	i	= Noise Levels at Mic 1, 2,.....9
	j	= 2 Mph Wind Class j
	1,2,3	= Measurement Stage No.

FIGURE 5 Insertion loss and degradation calculation method.

losses showed a wind dependency, the degradations did not. With the exception of all wind classes at Mic 1, two wind classes at Mic 2, and one at Mic 3, the degradations were statistically significant. The Student *t*-test with a significance level of 0.05 was used to perform these analyses (*17*).

The mean degradations range from 0 dBA at Mic 1 to 1.4 dBA at Mics 8 and 9. These results are consistent with the findings in the Los Angeles study (*3*) and indicate that although the degradations were statistically significant at all mic locations except Mic 1, they were not significant in terms of human perception. It is generally recognized that humans cannot perceive noise increases of 1 to 2 dBA.

The standard errors of the Stage 2 and 3 insertion losses (*16*) and degradations averaged 0.8 dBA.

Effects of Wind

To study the effects of crosswind vector wind velocities on noise levels, the differences in noise levels between Mic C2 and each mic (Mics 1 through 9) for each run were paired with their associated crosswind components and grouped by stage. The data were then submitted to a regression analysis. Similar analyses had been done in the I-405 parallel barrier demonstration project in Los Angeles (*3*).

As was so in Los Angeles, the linear regression analyses in this project clearly showed that for the wind ranges tested, the data could be represented by linear regression lines in the general form of

$$y = a + bx$$

where

y = difference between Mic C2 and Mic 1, 2, . . . 9 (dBA);
x = crosswind vector wind velocity (a negative velocity indicates a wind blowing from receiver to source; a positive wind indicates a wind blowing from source to receiver);
a = a constant; in this case the difference between Mic C2 and Mic 1, 2, . . . 9 at a 0-mph crosswind; and
b = slope of the regression line, in this case the change in velocity (dBA/mph).

The slope of the regression line is the most useful parameter for showing the effects of vector wind velocity. A negative slope indicates that downwind from the source the differences between Mic C2 and the mic of interest become smaller as the vector wind velocity increases. The same differences become greater upwind from the source. If a is set at zero, the regression equation is normalized to a zero wind condition at the mic of interest, and the slope directly indicates the wind

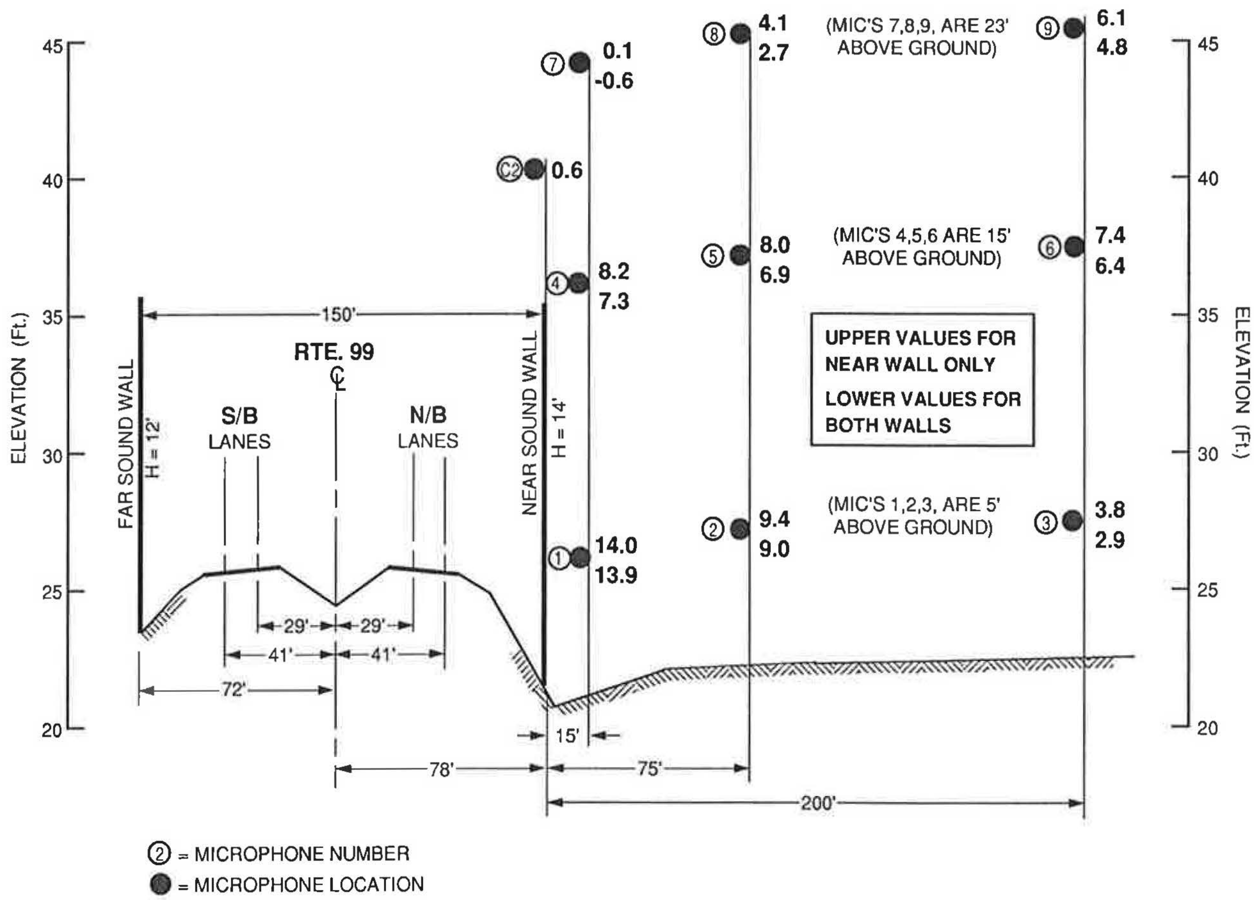

FIGURE 6 Mean insertion loss for near wall only and for both walls.

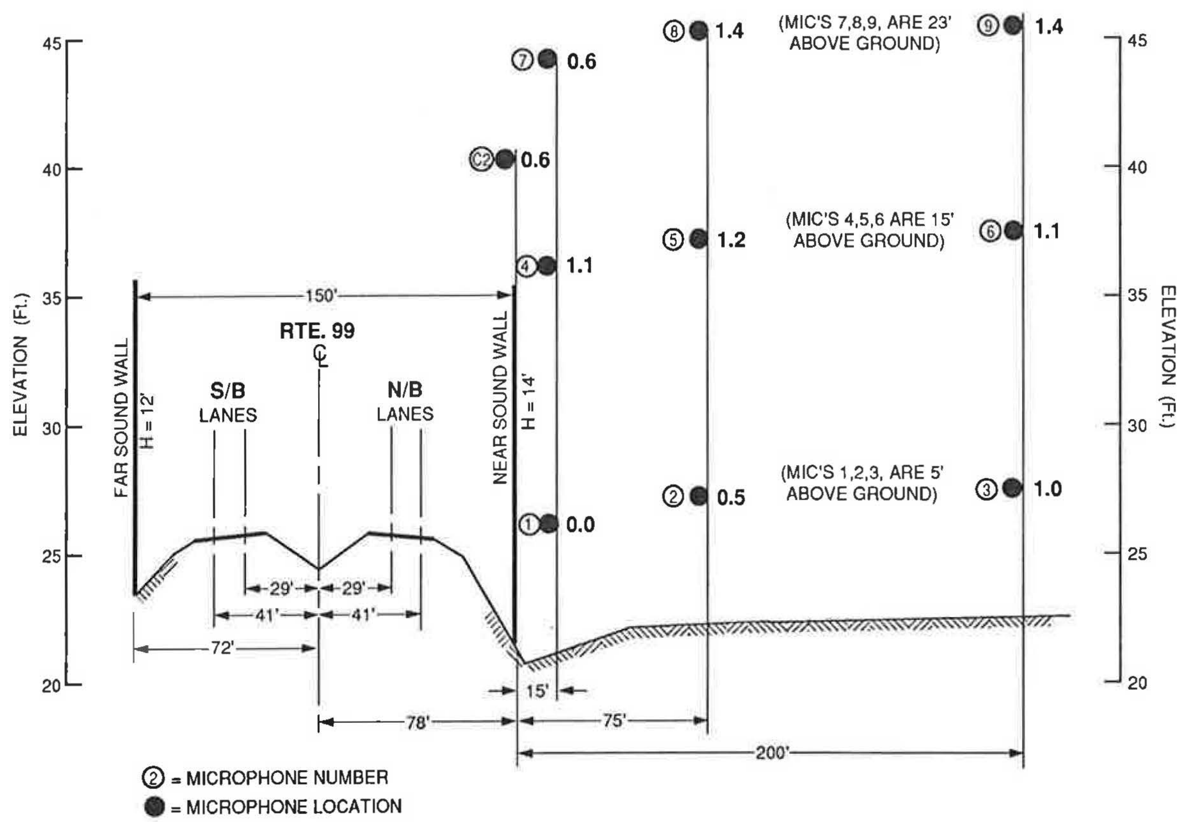

FIGURE 7 Mean insertion degradation due to far sound wall.

TABLE 1 Summary of Regression Slopes: C2 − *i* Differences Versus Crosswind Vector Velocity

Mic No.	Statistic	Stage 1	Stage 2	Stage 3
1	b=Slope (dBA/Mph) r = Coeff. of Corr.	-0.15* 0.57	-0.10* 0.65	-0.06* 0.51
2	b r	-0.21* 0.59	-0.38* 0.91	-0.37* 0.91
3	b r	-0.48* 0.78	-0.60* 0.93	-0.57* 0.95
4	b r	-0.03 0.15	-0.15* 0.92	-0.17* 0.85
5	b r	+0.07 0.23	-0.35* 0.92	-0.38* 0.95
6	b r	-0.03 0.13	-0.42* 0.95	-0.44* 0.96
7	b r	+0.03 0.11	+0.06* 0.71	+0.10* 0.86
8	b r	-0.02 0.07	-0.14* 0.78	-0.11* 0.69
9	b r	-0.02 0.08	-0.34* 0.91	-0.38* 0.96

Note: * Slope is significant ("F" test, level of sig.= 0.05)

effect in decibels per miles per hour at the mic location. To deal with noise levels, the signs of the slopes need to be reversed instead of the differences in noise levels.

Table 1 shows a summary of the regression line slopes and correlation coefficients of all data by mic and stage. Also shown are the results of statistical *F*-tests performed on the slopes of the regression lines (*18*). The *F*-test examines the validity of the hypothesis that the data do not show a regression, that is, that the slope of a regression line is zero.

The summary shows that Stage 1 wind effects were limited to the low mics (1, 2, and 3). After a barrier was constructed between the highway and receivers (Stages 2 and 3), the middle and high mics also showed significant wind effects within 200 ft behind the barrier.

Tables 2 and 3 were developed from Table 1. Shown are the wind effects on noise level during Stages 1, 2, and 3. The effects of Stages 2 and 3 were averaged because the regression slopes were very similar. No crosswind vectors of more than 6 mph were observed during Stage 1.

Other data in this project indicated the following observed extreme differences in noise levels at the Mic 3 location, by stage:

TABLE 2 Vector Wind Effects on Noise Levels, Stage 1

	---------------CROSSWIND VELOCITY---------------				
MIC.NO.	-2 MPH	0 MPH	+4 MPH	+8 MPH	+12 MPH
1	-0.2	0.0	+0.3	+0.6	+1.0 dBA
2	-0.8	0.0	+1.5	+3.0	+4.6 dBA
3	-1.2	0.0	+2.3	+4.7	+7.0 dBA
4	-0.3	0.0	+0.6	+1.3	+1.9 dBA
5	-0.7	0.0	+1.5	+2.9	+4.3 dBA
6	-0.9	0.0	+1.7	+3.4	+5.1 dBA
7	-0.2	0.0	+0.3	+0.6	+1.0 dBA
8	-0.3	0.0	+0.5	+1.0	+1.5 dBA
9	-0.7	0.0	+1.4	+2.9	+4.3 dBA

TABLE 3 Vector Wind Effects on Noise Levels, Stages 2 and 3

	---------------CROSSWIND VELOCITY---------------				
MIC.NO.	-2 MPH	0 MPH	+2 MPH	+4 MPH	+6 MPH
1	-0.3	0.0	+0.3	+0.6	+0.9 dBA
2	-0.4	0.0	+0.4	+0.8	+1.3 dBA
3	-1.0	0.0	+1.0	+1.9	+2.9 dBA
4-9	NO WIND EFFECTS				

- Stage 1, Mic 3, maximum noise level difference for 0- and 6-mph crosswind vector was +4 dBA.
- Stage 2, Mic 3, maximum noise level difference for −3 and 11-mph crosswind vector was +9 dBA.
- Stage 3, Mic 3, maximum noise level difference for 0- and 9-mph crosswind vector was +6 dBA.

The human ear can readily perceive these noise-level variations.

The ANSI S12.8 (1987) recommendation of comparing noise levels within a maximum allowable variation of average vector wind velocities of 2.4 m/sec (5.4 mph) appears to be too liberal, in light of the given data. As a member of the ANSI work group, the author has helped prepare a recommendation for revising the given standard on the basis of wind and noise data in this study.

CONCLUSIONS

The data in this report point toward two major findings:

1. The measured reductions in acoustical performance of one of two parallel noise barriers in this project are less than can be perceived by normal human ears.

2. Wind speed and direction can have a profound effect on noise measurements within 250 ft of a highway.

A summary of specific findings and conclusions in each of the two categories follows.

Reduction in Acoustical Performance (Insertion Loss Degradations)

- Degradations by wind class ranged from 0 to 1.9 dBA, depending on mic location.
- Mean degradations averaged over all wind classes ranged from 0 to 1.4 dBA, depending on mic location.
- Mean degradations increased with distance behind the barrier and height above the ground; the lowest mean of 0 dBA occurred 15 ft behind the near barrier, at a height of 5 ft (Mic 1); the highest mean of 1.4 dBA occurred at 200 ft behind the near barrier, at a height of 23 ft.
- Degradations were statistically significant at all mic locations, except for Mic 1 (*t*-test, level of significance = 0.05); in terms of human perception, however, none of the degradations was significant.
- With the results of the Los Angeles study and other data (*1*), this study concludes that reflective parallel barriers used in typical Caltrans configurations do not reduce the performance of each individual barrier perceptibly. However, there may be some unusual situations in which complex barrier or terrain configurations cause noise reflection problems.

Wind Effects

- Stage 2 and Stage 3 insertion losses tended to be wind-dependent; they generally decreased as the positive crosswind vector increased and increased as the negative crosswind vector increased.
- Degradations did not show any wind dependency.
- Within the observed ranges of crosswind vector wind speeds of -1 to $+6$ mph during Stage 1, -3 to $+11$ mph during Stage 2, and -1 to $+9$ mph during Stage 3, linear regression lines best described plots of noise levels normalized for source strengths versus crosswind vector velocity.
- Within these wind ranges, good correlations existed for Stage 1, low mics only, and all Stage 2 and 3 mic data. Noise levels increased downwind from the source and decreased upwind from the source.
- Wind effects increased with distance from the source, although not linearly.
- Construction of a noise barrier between source and receiver tended to enhance the wind effects.
- Without a barrier, wind effects at a normal receiver height of 5 ft above the ground were slightly less than the same receiver with a barrier. This accounted for the wind dependency of barrier insertion losses.
- Without a barrier, wind effects decreased rapidly with mic height. At a receiver height of 15 ft and above, no wind effects could be detected without a barrier.
- Comparing the "near barrier only" and "both barriers" conditions, the wind effects were nearly identical and extended to much greater heights than without barriers. At a receiver height of 23 ft, the wind effects were still large.
- These findings imply that second-story or higher dwellings could experience an increase in noise levels due to a noise barrier if they are located downwind from a highway. However, ground-level residences that are elevated above the highway by virtue of sloping terrain will not experience such an increase because the before-barrier wind effects are greater because of the proximity to the ground. By the same token, a second-story or higher dwelling upwind from the highway could benefit from a greater noise reduction due to the barrier.

RECOMMENDATIONS

This paper recommends that no mitigation of reflected noise will be attempted in California until controlled field studies indicate an actual problem with reflections, or until subsequent research, performed under real-world conditions, identifies under what conditions parallel barrier reflections will degrade the performance of a barrier by 3 dBA or more. On the basis of the data in this report and others (*14*), the author suspects that degradations of this amount occur when the ratio of the separation distance and height of the barriers is less than 10 to 1.

If such research is accomplished, a suitable FHWA-approved computer model needs to be available to predict the degradations and to include mitigation strategies in the future design of noise barriers where appropriate.

Future research in identifying parallel barrier problems should be done only under real-world conditions with extensive documentation of source, site, and atmospheric conditions. Such research is very expensive and should be done on a national scale.

The findings in this project also indicate an urgent need for more research in meteorological effects of noise levels with and without noise barriers, in the immediate vicinity of a highway as well as farther away. A better understanding of atmospheric effects on noise levels will help Caltrans districts address the scores of noise complaints received by the public each year. Many of these complaints may be caused by atmospheric phenomena.

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REFERENCES

1. M. M. Hatano. *Evaluation of Noise Barrier Reflection, 04-SCI-101-30.7.* Report CA-TL-7287-78-02. Office of Transportation Laboratory, California Department of Transportation, Sacramento, Jan. 1978.
2. W. Bowlby, L. F. Cohn, and R. A. Harris. A Review of Studies of Insertion Loss Degradation for Parallel Highway Noise Barriers. *Noise Control Engineering Journal*, March–April 1987.
3. R. Hendriks and J. Hecker. *Parallel Noise Barrier Report: A Noise Absorptive Demonstration Project—San Diego Freeway, Interstate Route 405 in the City of Los Angeles, Community of*

Brentwood. District 7 Environmental Investigations Section (and Transportation Laboratory), California Department of Transportation, July 1989.
4. C. Menge. Sound-Absorption Treatments for Highway Noise Barriers. In *Transportation Research Record 740*, TRB, National Research Council, Washington, D.C., 1980.
5. J. J. Hajek. Effects of Parallel Highway Noise Barriers. In *Transportation Research Record 933*, TRB, National Research Council, Washington, D.C., 1983.
6. W. Bowlby and L. F. Cohn. Insertion Loss Degradation For Parallel Traffic Noise Barriers. *Proc., Inter-Noise 84, Vol. I*, Noise Control Foundation, Inc., Poughkeepsie, N.Y., 1984.
7. D. A. Hutchins and H. W. Jones. Parallel Barriers in the Presence of Ground Surfaces. *Proc., Inter-Noise 84, Vol. I*, Noise Control Foundation, Inc., Poughkeepsie, N.Y., 1984.
8. C. G. Balachandran, R. G. Patching, and C. Andrew. Traffic Noise Attenuation Due to Single and Parallel Barriers. *Proc., Inter-Noise 84, Vol. I*, Noise Control Foundation, Inc., Poughkeepsie, N.Y., 1984.
9. W. Bowlby and L. F. Cohn. Image-3: Computer-Aided Design for Parallel Highway Noise Barriers. In *Transportation Research Record 933*, TRB, National Research Council, Washington, D.C., 1983.
10. *Effectiveness of Parallel Noise Barriers—An Iowa Study*. Planning and Research Division, Iowa Department of Transportation, Ames, Feb. 1984.
11. M. A. Staiano. Parallel Barrier Attenuation Degradation. Presented at Summer Meeting of TRB Committee A1F04, St. Petersburg, Fla., July 1985.
12. S. Slutsky and H. L. Bertoni. Some Insertion Loss Curves for Vertical and Tilted Barriers. Presented at Summer Meeting of TRB Committee A1F04, 1986.
13. W. Bowlby and L. F. Cohn. *Sound-Absorptive Highway Noise Barriers*. Report FHWA-TS-86-214. Office of Implementation (HRT-10), FHWA, U.S. Department of Transportation, May 1986.
14. G. G. Fleming and E. J. Rickley. *Parallel Barrier Effectiveness—Dulles Noise Barrier Project*. Report FHWA-RD-90-105. Office of Engineering and Highway Operations Research and Development, FHWA, U.S. Department of Transportation, May 1990.
15. R. L. Wayson and W. Bowlby. Atmospheric Effects on Traffic Noise Propagation. In *Transportation Research Record 1255*, TRB, National Research Council, Washington, D.C., 1990.
16. T. M. Barry and J. A. Reagan. *FHWA Highway Traffic Noise Prediction Model*. Report FHWA-RD-77-108. FHWA, U.S. Department of Transportation, Dec. 1978.
17. A. H. Bowker and G. J. Lieberman. *Engineering Statistics*, 2nd ed. Prentice-Hall, Inc., Englewood Cliffs, N.J., 1972.
18. N. R. Draper and H. Smith. *Applied Regression Analysis*. John Wiley & Sons, Inc., New York, N.Y., 1966.
19. R. W. Hendriks. *Field Evaluation of Acoustical Performance of Parallel Noise Barriers Along Route 99 in Sacramento, California*. Report FHWA/CA/TL-91/01. Division of New Technology, Materials and Research, California Department of Transportation, Sacramento, Jan. 1991.

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