TRAFFIC IMPACT MODELS

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Chapter 7 - Frequently used Symbols

Φ	=	fuel consumption per unit distance
ω	=	parameter
a	=	instantaneous acceleration rate (a>0, km/hr/sec)
A	=	acceleration rate
AADT	=	average annual daily traffic (vehicles/day)
ACCT	=	acceleration time
ADT	=	average daily traffic (vehicles/day)
C1	=	average peak hour concentration
C8	=	annual maximum 8-hour concentration
CLT	=	Deceleration time
d_s	=	average stopped delay per vehicle (secs)
E	=	Engine size (cm^3)
EFI	=	idle emission factor
EFL	=	intersection link composite emission factor
f	=	instantaneous fuel consumption (mL/sec)
F	=	fuel consumed (lit/km) [Watson, et al model]
F	=	average fuel consumption per roadway section (mL) [Akcelik model]
f_1	=	fuel consumption rate while cruising (mL/km)
f_2	=	fuel consumption rate while idling (mL/sec)
f_3	=	excess fuel consumption per vehicle stop (mL)
f_c	=	steady-state fuel consumption rate at cruising speed (mL/km)
h	=	average number of stops per vehicle
K_1	=	parameter representing idle flow rate (mL/sec)
K_2	=	parameter representing fuel consumption to overcome rolling resistance
K_4,K_5	=	parameters related to fuel consumption due to positive acceleration
L	=	payload (Kg)
LQU	=	queue length
m	=	expected number of single-vehicle accidents per unit time
NDLA	=	vehicles delayed per cycle per lane
P	=	probability of a single-vehicle accident
PKE	=	positive kinetic energy
q	=	flow (vph)
q_i	=	flow rate of type i vehicles (vph)
S	=	speed
SPD	=	cruise speed
T	=	average travel time per unit distance
u	=	model parameter related to driving conditions
V	=	average speed (km/hr) [Elemental model]
V	=	instantaneous speed (km/hr) [Akcelik and Bayley model]
V_c	=	steady-state cruising speed (km/hr)
V_f	=	final speed (km/hr)
V_i	=	initial speed (km/hr)
VMT	=	vehicle miles of travel
V_s	=	space mean speed (km/hr)
VSP	=	at-rest vehicle spacing

7. TRAFFIC IMPACT MODELS

7.1 Traffic and Safety

7.1.1 Introduction

This section ought to be about how traffic flow, speed and the like are related to accident frequency and severity. However, due to limitation of space, only the relationship between accident frequency and traffic flow will be discussed. The terminology that pertains to characteristics of the traffic stream has already been established and only a few definitions need to be added. The '**safety**' of an entity is defined as '**the number of accidents by type, expected to occur on the entity in a certain period, per unit of time**'. In this definition, 'accident types' are categories such as rear-end, sideswipe, single-vehicle, multi-vehicle, injury, property damage only, etc. The word 'expected' is as in probability theory: what would be the average-in-the-long-run if it was possible to freeze all the relevant circumstances of the period at their average, and then repeat it over and over again. The word 'entity' may mean a specific road section or intersection, a group of horizontal curves with the same radius, the set of all signalized intersections in Philadelphia, etc. Since the safety of every entity changes in time, one must be specific about the period. Furthermore, to facilitate communication, safety is usually expressed as a frequency. Thus, eg., one might speak about the expected number of fatal accidents/year in 1972-1976 for a certain road section. To standardize further one often divides by the section length; now the units may be, say, accidents/(year × km).

As defined, the safety of an entity is a string of expected frequencies, m_1, m_2, . . . ,m_i, . . . , one for each accident type chosen. However, for the purpose of this discussion it will suffice to speak about one (unspecified) accident type, the expected accident frequency of which is m_i.

7.1.2 Flow and Safety

The functional relationship between m_i and the traffic flow which the entity serves, is a **'safety performance function'**. A safety performance function is depicted schematically in Figure 7.1. For the moment its shape is immaterial. It tells how for some entity the expected frequency of accidents of some type would be changing if traffic flow on the entity changed while all other conditions affecting accident occurrence remained fixed. While the flow may be in any units, it is usually understood that it pertains to the same period of time which the accident frequency represents. Thus, eg., if the ordinate shows the expected number of fatal accidents/year in 1972-1976 for a certain road section, then the AADT is the average for the period 1972-1976.

Naturally, m_i can be a function of more than one traffic flow. Thus, eg., head-on collisions may depend on the two opposing flows; collisions between pedestrians and left-turning traffic depend on the flow of pedestrians, the flow of straight-through vehicles, and the flow of left-turning vehicles etc.. In short, the arguments of the safety performance function can be several flows.

In practice it is common to use the term '**accident rate**'. The accident rate is proportional to the slope of the line joining the origin and a point of the safety performance function. Thus, at point A of Figure 7.1, where AADT is 3000 vehicles per day and where the expected number of accidents for this road section is 1.05 accidents per year, the accident rate is $1.05/(3000\times365)=0.96\times10^{-6}$ accidents/vehicle. At point B the accident rate is $1.2/(4000\times365)=0.82\times10^{-6}$ accidents/vehicle. If the road section was, say, 1.7 km long, the same accident rates could be written as $1.05/(3000\times365\times1.7)=0.56\times10^{-6}$ accidents/vehicle-km and $1.2/(4000\times365\times1.7)=0.48\times10^{-6}$ accidents/vehicle-km.

The safety performance function of an entity is seldom a straight line. If so, the accident rate is not constant but varies with traffic flow. As a consequence, if one wishes to compare the safety of two or more entities serving different flows, one can not use the accident rate for this purpose. The widespread habit of using accident rates to judge the relative safety of different entities or to assess changes in safety of the same entity is inappropriate and often harmful. To illustrate, suppose that the AADT on the road section in Figure 7.1 increased from 3000 'before pavement resurfacing' to 4000 'after pavement resurfacing' and that the average accident frequency increased from 1.05 'before' to 1.3 'after'. Note that 1.2 accidents/year would be expected at AADT=4000 had the road surface remained unchanged (see Figure 7.1). Since 1.3 > 1.2 one must conclude that following resurfacing there was a **deterioration** of 0.1 accidents/year. But

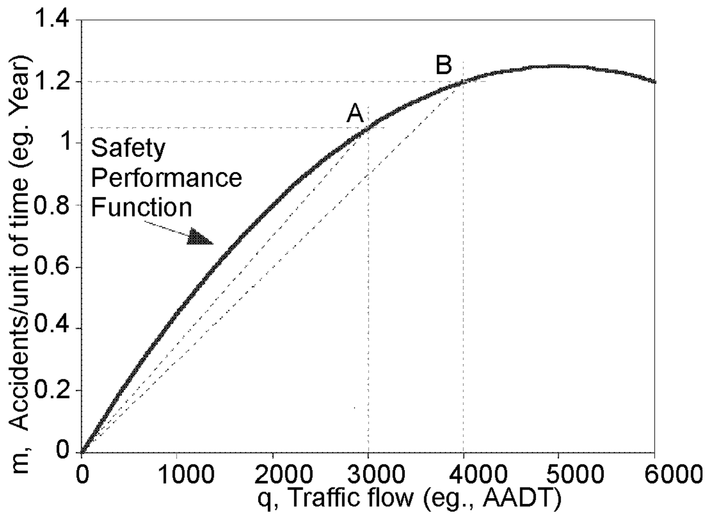

Figure 7.1
Safety Performance Function and Accident Rate.

less than the accident rate 'before resurfacing' ($1.05/3000\times365= 0.96\times10^{-6}$) which erroneously suggests that there has been an improvement. Similar arguments against the use of accident rates can be found in Pfundt (1969), Hakkert et al. (1976), Mahalel (1986), Brundell-Freij & Ekman (1991), Andreassen (1991).

To avoid such errors, the simple rule is that safety comparisons are legitimate only when the entities or periods are compared **as if they served the same amount of traffic**. To accomplish such equalization, one needs to know the safety performance function. Only in the special case when the performance function happens to be a straight line, may one divide by traffic flow and then compare accident rates. However, to judge whether the safety performance function is a straight line, one must know its shape, and when the shape of the safety performance function is known, the computation of an accident rate is superfluous. It is therefore best not to make use of accident rates. For this reason, the rest of the discussion is about expected accident frequencies, not rates.

Knowledge of safety performance functions is an important element of rational road safety management. The nature and shape of this function is subject to some logical considerations. However, much of the inquiry must be empirical.

7.1.3 Logical Considerations

It stands to reason that there is some kind of relationship between traffic flow and safety. For one, without traffic there are no traffic accidents. So, the safety performance function must go through the origin. Also, the three interrelated characteristics of the traffic stream - flow, speed and density - all influence the three interrelated aspects of safety - the frequency of opportunities for accidents to occur, the chance of accident occurrence given an opportunity, and the severity of the outcome given an accident. However, while a relationship may be presumed to exist, it is rather difficult to learn much about its mathematical form by purely deductive reasoning.

Using logic only, one could argue as follows: "If, as in probability theory, the passage of a vehicle through a road section or an intersection is a 'trial' the 'outcome' of which can be 'accident' or 'no-accident' with some fixed probability. Assume further that vehicle passages are so infrequent that this probability is not influenced by the frequency at which the 'trials' occur. Under such conditions the expected number of single-vehicle accidents in a fixed time period must be proportional to the number of trials in that time period - that is to flow." In symbols, $m_{\text{single-vehicle}}=qp$, where q is flow and p is the probability of a single-vehicle accident in one passage of a vehicle. In this, p is a constant that does not depend on q. Thus, one may argue, that the number of single-vehicle accidents ought to be proportional to flow, but only at very low flows.

As flow and density increase to a point where a driver can see the vehicle ahead, the correspondence between the mental picture of independent trials and between reality becomes strained. The probability of a 'trial' to result in a single-vehicle accident now depends on how close other vehicles are; that is $p = p(q)$. Should $p(q)$ be an increasing function of q, then $m_{single\text{-}vehicle}$ would increase more than in proportion with flow. Conversely, if an increase in flow diminishes the probability that a vehicle passing the road section will be in a single-vehicle accident, then $m_{single\text{-}vehicle}$ would increase less than in proportion to traffic flow; indeed, $m_{single\text{-}vehicle} = qp(q)$ can even decrease as traffic flow increases beyond a certain point. Thus, by logical reasoning one can only conclude that **near the origin**, the safety performance function for single-vehicle accidents ought to be a straight line.

If the safety performance function depends on two conflicting flows (car-train collisions at rail-highway grade crossings, car-truck collisions on roads, car-pedestrian collision at intersections etc.) then, near the origin, m_i should be proportional to the **product** of the two flows. One could also use the paradigm of probability theory to speculate that (at very low flows) the expected number of collisions with vehicles parked on shoulders is proportional to the **square** of the flows: in the language of 'trials', 'outcomes', the number of vehicles parked on the shoulder ought to be proportional to the passing flow and the number of vehicles colliding with the parked cars ought to be proportional to the same flow. From here there is only a small step to argue that, say, the number of rear-end collisions should also be proportional to q^2. Again, this reasoning applies only to very low flows. How m depends on q when speed choice, alertness and other aspects of behavior are also a function of flow, cannot be anticipated by speculation alone.

This is as far as logical reasoning seems to go at present. It only tells us what the shape of the safety performance function should be near the origin. Further from the origin, when p changes with q, not much is gained thinking of m as the product $qp(q)$. Since the familiar paradigm of 'trials' and 'outcomes' ceased to fit reality, and the notion of 'opportunity to have an accident' is vague, it might be better to focus directly on the function $m = m(q)$ instead of its decomposition into the product $qp(q)$.

Most theoretical inquiry into the relationship between flow and safety seems to lack detail. Thus, eg., most researchers try to relate the frequency of right-angle collisions at signalized intersections to the two conflicting flows. However, on reflection, the second and subsequent vehicles of a platoon may have a much lesser chance to be involved in such a collision than the first vehicle. Therefore it might make only a slight difference whether 2 or 20 vehicles have to stop for the same red signal. For this reason, the total flow is likely to be only weakly and circuitously related to the number of situations which generate right angle collisions at signalized intersections. There seems to be scope and promise for more detailed, elaborate and realistic theorizing. In addition, most theorizing to date attempted to relate safety to flow only. However, since flow, speed and density are connected, safety models could be richer if they contained all relevant characteristics of the traffic stream. Thus, eg., a close correspondence has been established between the number of potential overtakings derived from flow and speed distribution and accident involvement as a function of speed (Hauer 1971). Welbourne (1979) extends the ideas to crossing traffic and collisions with fixed objects. Ceder (1982) attempts to link accidents to headway distributions (that are a function of flow) through probabilistic modeling.

There is an additional aspect of the safety performance function which may benefit from logical examination. The claim was that it is reasonable to postulate the existence of a relationship between the traffic triad 'flow, speed and density' and between the safety triad 'frequency of opportunities, chance of accident given opportunity, and severity of the outcome given accident'. However, if there is a cause-effect relationship, it must be between accidents and the traffic characteristics near the time of their occurrence. One must ask whether there still is some meaningful safety performance function between accidents and traffic flow when flow is averaged over, say, a year. Whether the habit of relating accidents to AADTs (that is, averages over a year) materially distorts the estimated safety performance function is at present unclear. In a study by Quaye et al. (1993) three separate models were estimated from 15 minute flows, which then were aggregated into 1 hour flows and then into 7 hour flows. The three models differed but little. Persaud and Dzbik (1993) call models that relate hourly flows to accidents "microscopic" and models that relate AADT to yearly accident counts "macroscopic".

As will become evident shortly, empirical inquiries about safety performance functions display a disconcerting variety of results.

A part of this variety could be explained by the fact that the most ubiquitous data for such inquiries consist of flow estimates which are averages pertaining to periods of one year or longer.

7.1.4 Empirical Studies

Empirical studies about the association of traffic flow and accident frequency seldom involve experimentation; their nature is that of fitting functions to data. What is known about safety performance functions comes from studies in which the researcher assembles a set of data of traffic flows and accident counts, chooses a certain function that is thought to represent the relationship between the two, and then uses statistical techniques to estimate the parameters of the chosen function. Accordingly, discussion here can be divided into sections dealing with: (1) the kinds of study and data, (2) functional forms or models, (3) parameter estimates.

7.1.4.1 Kinds Of Study And Data

Data for the empirical investigations are accident counts and traffic flow estimates. A number-pair consisting of the accident count for a certain period and the estimated flow for the same period is a 'data point' in a Cartesian representation such as Figure 7.1. To examine the relationship between traffic flow and accident frequency, many such points covering an adequate range of traffic flows are required.

There are two study prototypes (for a discussion see eg., Jovanis and Chang 1987). The most common way to obtain data points for a range of flows is to choose many road sections or intersections that are similar, except that they serve different flows. In this case we have a '**cross-section'** study. In such a study the accident counts will reflect not only the influence of traffic flow but also of all else that goes with traffic flow. In particular, facilities which carry larger flows tend to be built and maintained to higher standards and tend to have better markings and traffic control features. This introduces a systematic bias into 'cross-section' models. If a road that is built and maintained to a higher standard is safer, then accident counts on high-flow roads will tend to be smaller than had the roads been built, maintained and operated in the same way as the lower flow roads. Thus, in a cross-section study, it is difficult to separate what is due to traffic flow, and what is due to all other factors which depend on traffic flow. It is therefore questionable whether the result of a cross section study enables one to anticipate how safety of a certain facility would change as a result of a change in traffic flow.

Less common are studies that relate different traffic flows on the same facility to the corresponding accident counts. In this case we have a '**time-sequence**' study. In such a study one obtains the flow that prevailed at the time of each accident and the number of hours in the study period when various flows prevailed. The number of accidents in a flow group divided by the number of hours when such flows prevailed is the accident frequency (see, eg., Leutzbach et al. 1970, Ceder and Livneh 1978, Hall and Pendelton 1991). This approach might obviate some of the problems that beset the cross-section study. However, the time-sequence study comes with its own difficulties. If data points are AADTs and annual accident counts over a period of many years, then the range of the AADTs is usually too small to say much about any relationship. In addition, over the many years, driver demography, norms of behavior, vehicle fleet, weather, and many other factors also change. It is therefore difficult to distinguish between what is due to changes in traffic flow and what is due to the many other factors that have also changed. If the data points are traffic flows and accidents over a day, different difficulties arise. For one, the count of accidents (on one road and when traffic flow is in a specified range) is bound to be small. Also, low traffic flows occur mostly during the night, and can not be used to estimate the safety performance function for the day. Also, peak hour drivers are safer en-route to work than on their way home in the afternoon, and off-peak drivers tend to be a different lot altogether.

7.1.4.2 Models

The first step of an empirical study of the relationship between traffic flow and accident frequency is to assemble, plot and examine the data. The next step is to select the candidate model equation(s) which might fit the data and serve as the safety performance function. Satterthwaite (1981, section 3) reviews the most commonly used models. Only those that are plausible and depend on traffic flow only are listed below. Traffic flow, while important, is but one of the many variables on which the expected accident frequency depends. However, since the monograph is devoted to traffic flow, the dependence on other variables will not be pursued here. The Greek letters are the unknown parameters to be estimated from data.

When only one traffic stream is relevant the power function and polynomial models have been used:

$$m = \alpha q^{\beta} \tag{7.1}$$
$$m = \alpha q + \beta q^2 + ... \tag{7.2}$$

At times the more complex power form

$$m = \alpha q^{\beta + \gamma log(q)} \tag{7.1a}$$

is used which is also akin to the polynomial model 7.2 when written in logarithms

$$log(m) = log(\alpha) + \beta log(q) + \gamma [log(q)]^2 \tag{7.2a}$$

When two or more traffic streams or kinds of vehicles are relevant, the product of power functions seems common:

$$m = \alpha q_1^{\beta} q_2^{\gamma} ... \tag{7.3}$$

The common feature of the models most often used is that they are linear or can be made so by logarithmization. This simplifies statistical parameter estimation. The shapes of these functions are shown in Figure 7.2.

The power function (Equation 7.1) is simple and can well satisfy the logical requirements near the origin (namely, that when q=0 m=0, and that β=1 when one flow is involved or β=2 when two flows are involved). However, its simplicity is also its downfall. If logic dictates, eg., that near the origin β=1 (say, for single-vehicle accidents), then the safety performance function has to be a straight line even for the higher flows where $p(q)$ is not constant any more. Similarly, if logic says that β=2, the quadratic growth applies for all q. In short, if β is selected to meet requirements of logic, the model may not fit the data further from the origin. Conversely, if β is selected to fit the data best, the logical requirements will not be met.

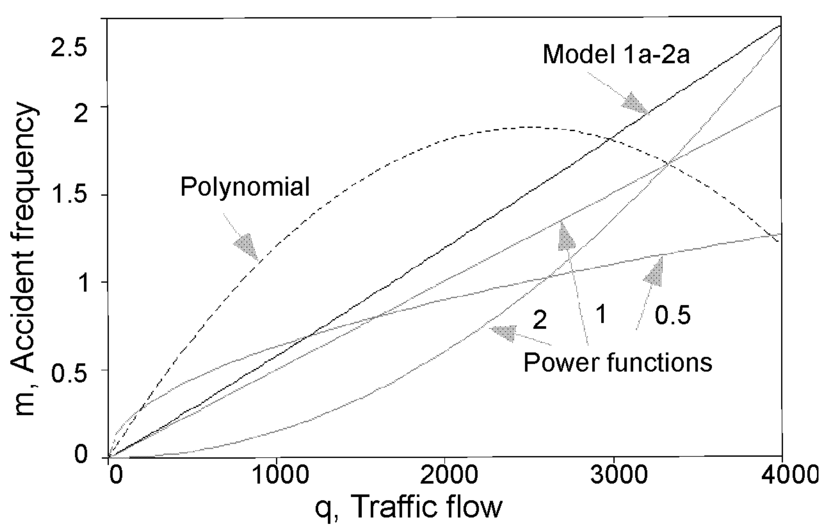

Figure 7.2
***Shapes of Selected Model Equations*.**

The popularity of the power function in empirical research derives less from its suitability than from it being 'lazy user friendly'; most software for statistical parameter estimation can accommodate the power function with little effort. The polynomial model (Equation 7.2) never genuinely satisfies the near-origin requirements. Its advantage is that by using more terms (and more parameters) the curve can be bent and shaped almost at will. This is achieved at the expense of parsimony in parameters.

If the data suggest that as flow increases beyond a certain level, the slope of *m(q)* is diminishing, perhaps even grows negative, an additional model that is parsimonious in parameters might deserve consideration:

$$m = \alpha q^k e^{\beta q} \tag{7.4}$$

where k = 1 or 2 in accord with the near-origin requirements. When $\beta<0$ the function has a maximum at $q = -k/\beta$. Its form when k=1 and k=2 is shown in Figure 7.3. The advantage of this model is that it can meet the near-origin requirements and still can follow the shape of the data.

A word of caution is in order. In the present context the focus is on how accident frequency depends on traffic flow. Accordingly, the models were written with flow (q) as the principal independent variable. However, traffic flow is but one of the many causal factors which affect accident frequency. Road geometry, time of day, vehicle fleet, norms of behaviour and the like all play a part. Therefore, what is at times lumped into a single parameter (α in equation 7.1, 7.1a, 7.3 and 7.4) really represents a complex multivariate expression. In short, the modeling of accident frequency is multivariate in nature.

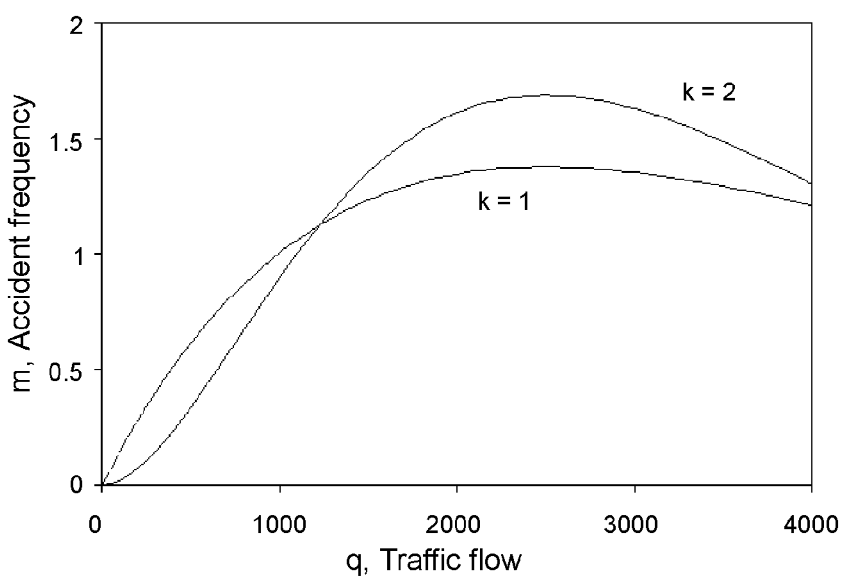

Figure 7.3
***Two Forms of the Model in Equation 7.4*.**

7.1.4.3 Parameter Estimates

With the data in hand and the functional form selected, the next step is to estimate the parameters (α, β,...). In earlier work, estimation was often by minimization of squared deviations. This practice now seems deficient. Recognizing the discrete nature of accident counts, the fact that their variance increases with their mean, and the possible existence of over-dispersion, it now seems that more appropriate statistical techniques are called for (see eg., Hauer 1992, Miaou and Lum 1993).

Results of past research are diverse. Part of the diversity is due to the problems which beset both the cross-section and the time sequence of studies; another part is due to the use of AADT and similar long-period averages that have a less than direct tie to accident occurrence; some of the diversity may come from various methodological shortcomings (focus on accident rates, choice of simplistic model equations, use of inappropriate statistical techniques); and much is due to the diversity between jurisdictions in what counts as a reportable accident and in the proportion of reportable accidents that get reported. Hauer and Persaud (1996) provide a comprehensive review of safety performance functions and their parameter values (for two-lane roads, multi-lane roads without access control, freeways, intersections and interchanges) based on North American data. A brief summary of this information and some international results are given below.

A. Road Sections. In a cross-section study of Danish *rural roads* Thorson (1967) estimated the exponent of ADT to be 0.7. In a similar study of German rural roads Pfundt (1968) estimated the exponent of ADT to be 0.85. Kihlberg and Tharp (1968) conducted an extensive cross-section study using data from several states. For sections that are 0.5 miles long, they estimate the parameters for a series of road types and geometric features. The model used is an elaborated power function $m=\alpha(ADT)^{\beta}(ADT)^{\gamma \log(ADT)}$. The report contains a rich set of results but creates little order in the otherwise bewildering variety. Ceder and Livneh (1982) used both cross-sectional and time-sequence data for interurban road sections in Israel, using the simple power function model (Equation. 1). The diverse results are difficult to summarize. Cleveland et al. (1985) divide low-volume rural two-lane roads into 'bundles' by geometry and find the ADT exponents to range from 0.49 to 0.93 for off-road accidents Recent studies in the UK show that on urban road sections, for single-vehicle accidents the exponent of AADT in model 7.1 is 0.58; for rear-end accidents the exponent of AADT is 1.43. In a time-sequence study, Hall and Pendelton (1990) use ten mile 2 and 4-lane road segments surrounding permanent counting stations in New Mexico and provide a wealth of information about accident rates in relations to hourly flows and time of day. In an extensive cross-section study, Zegeer et al. (1986) find that the exponent of ADT is 0.88 for the total number of accidents on rural two-lane roads. Ng and Hauer (1989) use the same data as Zegeer and show that the parameters differ from state to state and also by lane width. For non-intersection accidents on rural two lane roads in New York State, Hauer et al. (1994) found that when m is measured in [accidents/(mile-year)] and AADT is used for q, then in model 1, in 13 years α varies from 0.0024-0.0028 and β=0.78. Persaud (1992) using data on rural roads in Ontario finds the exponent of AADT to vary between 0.73 and 0.89, depending on lane and shoulder width. For urban two-lane roads in Ontario the exponent is 0.72 For urban multi-lane roads (divided or undivided) β=1.14, for rural multi-lane divided roads it is 0.62 but for undivided roads it is again 1.13.

For California freeways Lundy (1965) shows that the accidents per million vehicle miles increase roughly linearly with ADT. This implies the quadratic relationship of model 2. Based on the figures in Slatterly and Cleveland (1969), with m measured in [accidents/day], $m = (5.8\times10^{-7})ADT + (2.4\times10^{-11})ADT^2$ for four-lane freeways, $m = (6.6\times10^{-7})ADT + (.94 \times 10^{-11})ADT^2$ for six-lane freeways and $m = (5.4 \times 10^{-7}) \times ADT + (.78 \times 10^{-11}) \times ADT^2$ for eight-lane freeways. Leutzbach (1970) examines daytime accidents on a stretch of an autobahn. Fitting a power function to his Figure (1c) and with m measured in accidents per day, $m =(3\times10^{-11})\times(\text{hourly flow})^3$. However, there is an indication in this and other data that as flow increases, the accident rate initially diminishes and then increases again. If so a third degree polynomial might be a better choice. Jovanis and Chang (1987) fit model 7.3 to the Indiana Toll Road and find the exponents to be 0.25 and 0.23 for car and truck-miles. Persaud and Dzbik (1993) find that when yearly accidents are related to AADT, $m=0.147\times(AADT/1000)^{1.135}$ for 4-lane freeways but, when hourly flows are related to accidents/hour, $m = 0.00145 \times (\text{hourly flow}/1000)^{0.717}$. Huang et al. (1992) report for California that Number of accidents = 0.65 + 0.666 × million-vehicle-miles.

B. Intersections. Tanner (1953) finds for rural T intersections in the UK the exponents to be 0.56 and 0.62 for left turning traffic and main road traffic respectively. Roosmark (1966) finds for similar intersections in Sweden the corresponding exponents to be 0.42 and 0.71. For intersections on divided highways McDonald (1953) gives m in accidents per year as $m = 0.000783 \times (\text{major road ADT})^{0.455} \times (\text{cross road ADT})^{0.633}$. For signalized intersections in California, Webb (1955) gives m in accidents per year, as $m = 0.00019 \times (\text{major road ADT})^{0.55} \times (\text{cross road ADT})^{0.55}$ in urban areas with speeds below 40 km/h ; $= 0.0056 \times (\text{major road ADT})^{0.45} \times (\text{cross road ADT})^{0.38}$ in semi-urban areas with speeds in the 40-70 km/h range; and $= 0.007 \times (\text{major road ADT})^{0.51} \times (\text{cross road ADT})^{0.29}$ in rural areas with speeds above 70 km/h. For rural stop-controlled intersections in Minnesota, Bonneson and McCoy (1993) give m [in accidents/year] $= 0.692 \times (\text{major road ADT}/1000)^{0.256} \times (\text{cross road ADT}/1000)^{0.831}$. Recent studies done in the UK show that at signalized intersections for single vehicle accidents the exponent of AADT in model 1 is 0.89; for right angle accidents (model 7.3) the exponents of AADT are 0.36 and 0.60 and for accident to left-turning traffic (their right turn) the exponents are 0.57 and 0.46. Using data from Quebec, Belangér finds that the expected annual number of accidents at unsignalized rural intersections is $0.002 \times (\text{Major road ADT})^{0.42} \times (\text{Cross road ADT})^{0.51}$.

C. Pedestrians. Studies in the UK show that for nearside pedestrian accidents on urban road sections the exponents for vehicle and pedestrian AADTs in model 7.3 are 0.73 and 0.42. With m measured in [pedestrian accidents/year] Brüde and Larsson (1993) find that, at intersections, $m = (7.3 \times 10^{-6})(\text{incoming traffic/day})^{0.50}$ $(\text{crossing pedestrians/day})^{0.7}$. With m measured in (pedestrian accidents/hour), Quaye et al. (1993) find that if left-turning vehicles at signalized intersections do not face opposing vehicular traffic then $m = 1.82 \times 10^{-8}(\text{hourly flow of left-turning cars})^{1.32} \times (\text{hourly pedestrian flow})^{0.34}$; when the left turning vehicles have to find gaps in the opposing traffic, $m = 1.29 \times 10^{-7}(\text{hourly flow of left-turning cars})^{0.36} \times (\text{hourly pedestrian flow})^{0.86}$.

7.1.5 Closure

Many aspects of the traffic stream are related to the frequency and severity of accidents; only the relationship with flow has been discussed here. How safety depends on flow is important to know. The relationship of traffic flow to accident frequency is called the 'safety performance function'. Only when the safety performance function is known, can one judge whether one kind of design is safer than another, or whether an intervention has affected the safety of a facility. Simple division by flow to compute accident rates is insufficient because the typical safety performance function is not linear.

Past research about safety performance functions has led to diverse results. This is partly due to the use of flow data which are an average over a long time period (such as AADT), partly due to the difficulties which are inherent in the cross-sectional and the time-sequence studies, and partly because accident reporting and roadway definitions vary among jurisdictions. However, a large part of the diversity is due to the fact that accident frequency depends on many factors in addition to traffic flow and that the dependence is complex.Today, some of the past difficulties can be overcome. Better information about traffic flows is now available (eg. from freeway-traffic-management-systems, permanent counting stations, or weight-in-motion devices); also better methods for the multivariate analysis of accident counts now exist. However, in addition to progress in statistical modeling, significant advances seem possible through the infusion of detailed theoretical modeling which makes use of all relevant characteristics of the traffic stream such as speed, flow, density, headways and shock waves.

7.2 Fuel Consumption Models

Substantial energy savings can be achieved through urban traffic management strategies aimed at improving mobility and reducing delay. It is conservatively estimated, for example, that if all the nearly 250,000 traffic signals in the U.S. were optimally timed, over 19 million liters (5 million gallons) of fuel would be saved each day. It is further estimated that 45 percent of the total energy consumption in the U.S. is by vehicles on roads. This amounts to some 240 million liters (63 million gallons) of petroleum per day, of which nearly one-half is used by vehicles under urban driving conditions.

Fuel consumption and emissions have thus become increasingly important measures of effectiveness in evaluating traffic management strategies. Substantial research on vehicular energy consumption has been conducted since the 1970's, resulting in an array of fuel consumption models. In this section, a number of such models which have been widely adopted are reviewed.

7.2.1 Factors Influencing Vehicular Fuel Consumption

Many factors affect the rate of fuel consumption. These factors can be broadly categorized into four groups: vehicle, environment, driver, and traffic conditions. The main variables in the traffic category include speed, number of stops, speed noise and acceleration noise. Speed noise and acceleration noise measure the amount of variability in speed and acceleration in terms of the variance of these variables. The degree of driver aggressiveness also manifests itself in speed and acceleration rates and influences the fuel consumption rate.

Factors related to the driving environment which could affect fuel consumption include roadway gradient, wind conditions, ambient temperature, altitude, and pavement type (for example, AC/PCC/gravel) and surface conditions (roughness, wet/dry). Vehicle characteristics influencing energy consumption include total vehicle mass, engine size, engine type (for example, gasoline, diesel, electric, CNG), transmission type, tire type and size, tire pressure, wheel alignment, the status of brake and carburetion systems, engine temperature, oil viscosity, gasoline type (regular, unleaded, etc.), vehicle shape, and the degree of use of auxiliary electric devices such as air-conditioning, radio, wipers, etc. A discussion of the degree of influence of most of the above variables on vehicle fuel efficiency is documented by the Ontario Ministry of Transportation and Communications (TEMP 1982).

7.2.2 Model Specifications

Fuel consumption models are generally used to evaluate the effectiveness and impact of various traffic management strategies. As such these models are developed using data collected under a given set of vehicle fleet and performance characteristics such as weight, engine size, transmission type, tire size and pressure, engine tune-up and temperature conditions, etc. The variation in fuel consumption due to other factors such as driver characteristics, ambient temperature, pavement roughness, grades, wind, altitude, etc. is hoped to be small due to collective effects if data points represent aggregate values over sufficiently long observation periods. Alternatively, model estimates may be adjusted for known effects of roadway grade, ambient temperature, altitude, wind conditions, payload, etc.

Given a fixed set of vehicle and driver characteristics and environmental conditions, the influence of traffic-related factors on fuel consumption can be modeled. A number of studies in Great Britain (Everall 1968), Australia (Pelensky et al. 1968), and the United States (Chang et al. 1976; Evans and Herman 1978; Evans et al. 1976) all indicated that the fuel consumption per unit distance in urban driving can be approximated by a linear function of the reciprocal of the average speed. One such model was proposed by Evans, Herman, and Lam (1976), who studied the effect of sixteen traffic variables on fuel consumption. They concluded that speed alone accounts for over 70 percent of the variability in fuel consumption for a given vehicle. Furthermore, they showed that at speeds greater than about 55 km/h, fuel consumption rate is progressively influenced by the aerodynamic conditions. They classified traffic conditions as urban (V < 55 km/h) versus highway (V > 55 km/h) traffic showing that unlike the highway regime, in urban driving fuel efficiency improves with higher average speeds (Figure 7.4).

7.2.3 Urban Fuel Consumption Models

Based on the aforementioned observations, Herman and co-workers (Chang and Herman 1981; Chang et al. 1976; Evans and Herman 1978; Evans et al. 1976) proposed a simple theoretically-based model expressing fuel consumption in urban conditions as a linear function of the average trip time per unit distance (reciprocal of average speed). This model, known as the Elemental Model, is expressed as:

$$\Phi = K_1 + K_2 T, \quad V < 55 \text{ km/hr} \tag{7.5}$$

where,

Φ : fuel consumption per unit distance

T : average travel time per unit distance

and

$V(=1/T)$: average speed

K_1 and K_2 are the model parameters. K_1 (in mL/km) represents fuel used to overcome the rolling friction and is closely related

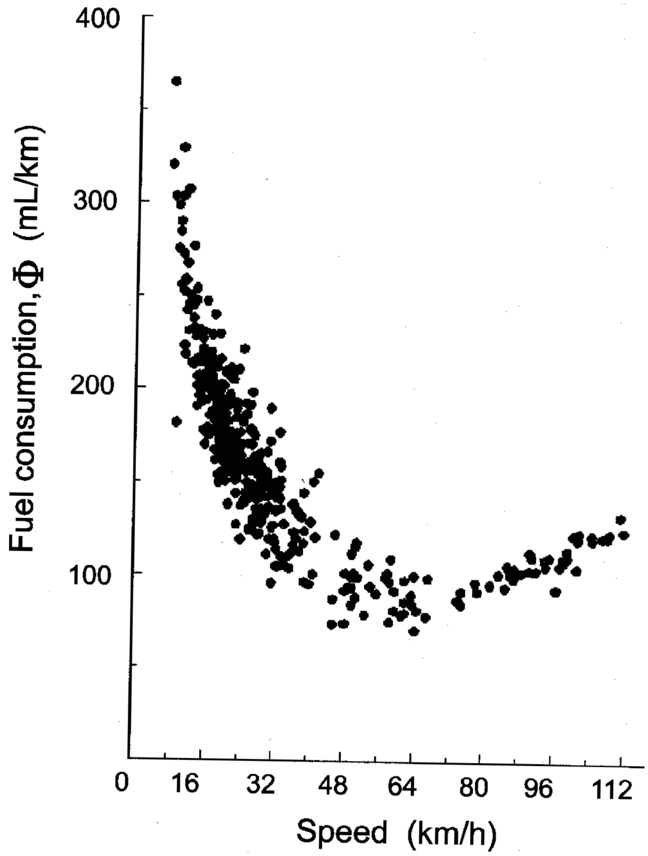

Figure 7.4
Fuel Consumption Data for a Ford Fairmont (6-Cyl.)
Data Points represent both City and Highway Conditions.

to the vehicle mass (Figure 7.5). K_2 (in mL/sec) is a function In an effort to improve the accuracy of the Elemental Model, other researchers have considered additional independent variables. Among them, Akcelik and co-workers (Akcelik 1981; Richardson and Akcelik1983) proposed a model which separately estimates the fuel consumed in each of the three portions of an urban driving cycle, namely, during cruising, idling, and deceleration-acceleration cycle. Hence, the fuel consumed along an urban roadway section is estimated as:

$$F = f_1 X_s + f_2 d_s + f_3 h, \tag{7.11}$$

where,

F = average fuel consumption per roadway section (mL)
X_s = total section distance (km)
d_s = average stopped delay per vehicle (secs)
h = average number of stops per vehicle
f_1 = fuel consumption rate while cruising (mL/km)
f_2 = fuel consumption rate while idling (mL/sec)
f_3 = excess fuel consumption per vehicle stop (mL)

The model is similar to that used in the TRANSYT-7F simulation package (Wallace 1984).

Herman and Ardekani (1985), through extensive field studies, have shown that delay and number of stops should not be used

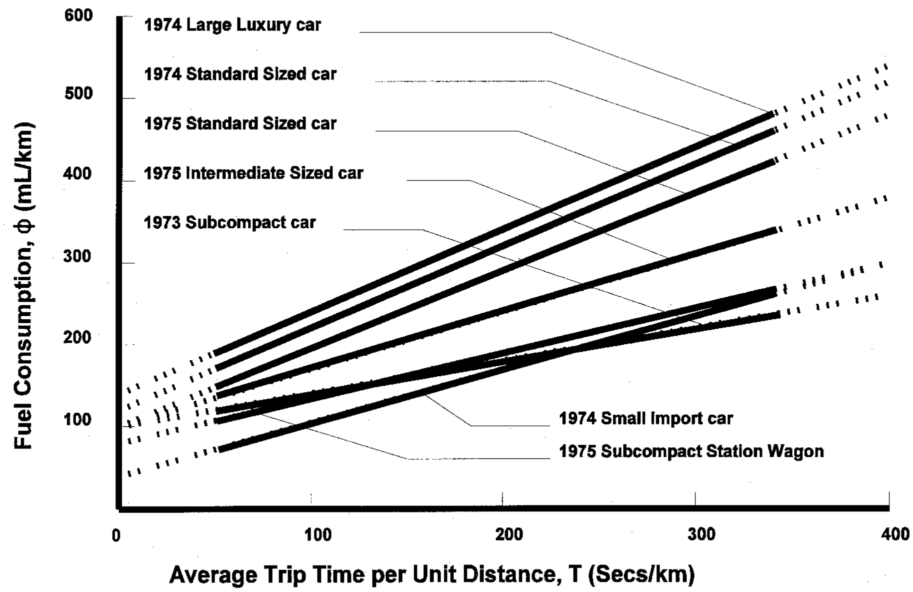

Figure 7.5 Fuel Consumption Versus Trip Time per Unit Distance for a Number of Passenger Car Models.

together in the same model as estimator variables. This is due to the tendency for the number of stops per unit distance to be highly correlated with delay per unit distance under urban traffic conditions. They propose an extension of the elemental model of Equation 7.5 in which a correction is applied to the fuel consumption estimate based on the elemental model depending on whether the number of stops made is more or less than the expected number of stops for a given average speed (Herman and Ardekani 1985).

A yet more elaborate urban fuel consumption model has been set forth by Watson et al. (1980). The model incorporates the changes in the positive kinetic energy during acceleration as a predictor variable, namely,

$$F = K_1 + K_2/V_s + K_3 V_s + K_4 PKE, \qquad (7.12)$$

where,

F = fuel consumed (Lit/km)
V_s = space mean speed (km/hr)

The term *PKE* represents the sum of the positive kinetic energy changes during acceleration in m/sec^2, and is calculated as follows:

$$PKE = \Sigma(V_f^2 - V_i^2)/(12{,}960\ X_s) \qquad (7.13)$$

where,

V_f = final speed (km/hr)
V_i = initial speed (km/hr)
X_s = total section length (km)

A number of other urban fuel consumption models are discussed by other researchers, among which the work by Hooker et al. (1983), Fisk (1989), Pitt et al. (1987), and Biggs and Akcelik (1986) should be mentioned.

7.2.4 Highway Models

Highway driving corresponds to driving conditions under which average speeds are high enough so that the aero-dynamic effects on fuel consumption become significant. This occurs at average speeds over about 55 km/h (Evans et al. 1976). Two highway models based on constant cruising speed are those by Vincent et al. (1980) and Post et al. (1981). The two models are valid at any speed range, so long as a relatively constant cruise speed can be maintained (steady state speeds). The steady-state speed requirement is, of course, more easily achievable under highway driving conditions.

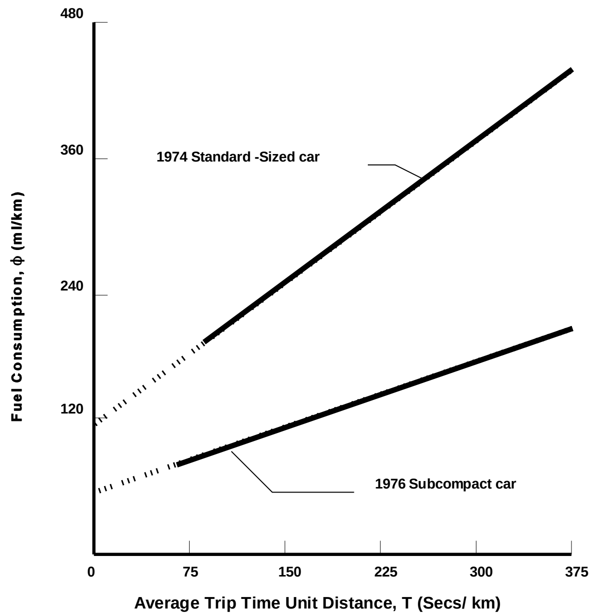

Figure 7.6
Fuel Consumption Data and the Elemental Model Fit
for Two Types of Passenger Cars (Evans and Herman 1978).

The model by Vincent, Mitchell, and Robertson is used in the TRANSYT-8 computer package (Vincent et al. 1980) and is in the form:

$$f_c = a + b\,V_c + c\,V_c^2, \tag{7.14}$$

where,

V_c = steady-state cruising speed (km/hr)
f_c = steady-state fuel consumption rate at cruising speed (mL/km), calibration of this model for a mid-size passenger car yields

a = 170 mL/km,
b = -4.55 mL-hr/km^2; and
c = 0.049 mL-hr^2/km^3 (Akcelik 1983).

A second steady-state fuel model formulated by Post et al. (1981) adds a V^2 term to the elemental model of Equation 7.5 to account for the aero-dynamic effects, namely,

$$f_c = b_1 + b_2/V_c + b_3\,V_c^2. \tag{7.15}$$

Calibration of this model for a Melbourne University test car (Ford Cortina Wagon, 6-Cyl, 4.1L, automatic transmission) yields (Akcelik 1983) the following parameter values (Also see Figure 7.7):

b_1 = 15.9 mL/km
b_2 = 2,520 mL/hr
b_3 = 0.00792 mL-hr^2/km^3.

Instantaneous fuel consumption models may also be used to estimate fuel consumption for non-steady-state speed conditions under both urban or highway traffic regimes. These models are used in a number of microscopic traffic simulation packages such as NETSIM (Lieberman et al. 1979) and MULTSIM (Gipps and Wilson 1980) to estimate fuel consumption based on instantaneous speeds and accelerations of individual vehicles.

By examining a comprehensive form of the instantaneous model, Akcelik and Bayley (1983) find the following simpler form of the function to be adequate, namely,

$$f = K_1 + K_2\,V + K_3\,V^3 + [K_4\,aV + K_5\,a^2V] \tag{7.16}$$

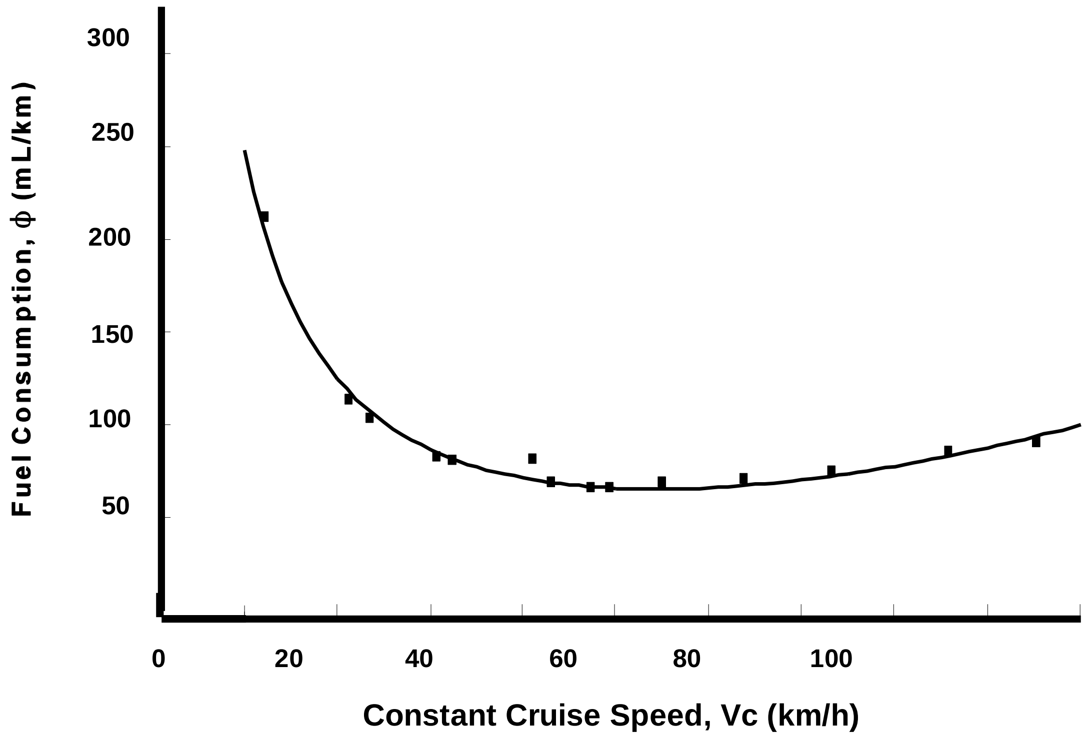

Figure 7.7
Constant-Speed Fuel Consumption per Unit Distance for the Melbourne University Test Car (Akcelik 1983).

where,

f	=	instantaneous fuel consumption (mL/sec)
V	=	instantaneous speed (km/hr)
a	=	instantaneous acceleration rate ($a > 0$)(km/hr/sec)
K_1	=	parameter representing idle flow rate (mL/sec)
K_2	=	parameter representing fuel consumption to overcome rolling resistance
K_3	=	parameter representing fuel consumption to overcome air resistance
K_4, K_5	=	parameters related to fuel consumption due to positive acceleration

The above model has been used by Kenworthy et al. (1986) to assess the impact of speed limits on fuel consumption.

7.2.5 Discussion

During the past two decades, the U.S. national concerns over dependence on foreign oil and air quality have renewed interest in vehicle fuel efficiency and use of alternative fuels. Automotive engineers have made major advances in vehicular fuel efficiency (Hickman and Waters 1991; Greene and Duleep 1993; Komor et al. 1993; Greene and Liu 1988).

Such major changes do not however invalidate the models presented, since the underlying physical laws of energy consumption remain unchanged. These include the relation between energy consumption rate and the vehicle mass, engine size, speed, and speed noise. What does change is the need to recalibrate the model parameters for the newer mix of vehicles. This argument is equally applicable to alternative fuel vehicles (DeLuchi et al. 1989), with the exception thatthere may also be a need to redefine the variable units, for example, from mL/sec or Lit/km to KwH/sec or KwH/km, respectively.

7.3 Air Quality Models

7.3.1 Introduction

Transportation affects the quality of our daily lives. It influences our economic conditions, safety, accessibility, and capability to reach people and places. Efficient and safe transportation satisfies us all but in contrast, the inefficient and safe use of our transportation system and facilities which result in traffic congestions and polluted air produces personal frustration and great economic loss.

The hazardous air pollutants come from both mobile and stationary sources. Mobile sources include passenger cars, light and heavy trucks, buses, motorcycles, boats, and aircraft. Stationary sources range from oil refineries to dry cleaners and iron and steel plants to gas stations.

This section concentrates on mobile source air pollutants which comprise more than half of the U.S. air quality problems. Transportation and tailpipe control measure programs in addition to highway air quality models are discussed in the section.

Under the 1970 U.S. Clean Air Act, each state must prepare a State Implementation Plan (SIP) describing how it will controll emissions from mobile and stationary sources to meet the requirements of the National Ambient Air Quality Standards (NAAQS) for six pollutants: (1) particulate matter (formerly known as total suspended particulate (TSP) and now as PM10 which emphasizes the smaller particles), (2) sulfur dioxide (SO2), (3) carbon monoxide (CO), (4) nitrogen dioxide (NO2), (5) ozone (O3), and (6) lead (Pb).

The 1990 U.S. Clean Air Act requires tighter pollution standards especially for cars and trucks and would empower the Environmental Protection Agency (EPA) to withhold highway funds from states which fail to meet the standards for major air pollutants (USDOT 1990). The 1970 and 1990 federal emission standards for motor vehicles are shown in Table 7.1.

This section concentrates on mobile source air pollutants which comprise more than half of the U.S. air quality problems. Exhaust from these sources contain carbon monoxide, volatile organic compounds (VOCs), nitrogen oxides, particulates, and lead. The VOCs along with nitrogen oxides are the major elements contributing to the formation of "smog".

Carbon monoxide which is one of the main pollutants is a colorless, and poisonous gas which is produced by the incomplete burning of carbon in fuels. The NAAQS standard for ambient CO specifies upper limits for both one-hour and eight-hour averages that should not be exceeded more than once per year. The level for one-hour standard is 35 parts per million (ppm), and for the eight-hour standard is 9 ppm. Most information and trends focus on the 8-hour average because it is the more restrictive limit (EPA 1990).

7.3.2 Air Quality Impacts of Transportation Control Measures

Some of the measures available for reducing traffic congestion and improving mobility and air quality is documented in a report prepared by Institute of Transportation Engineers (ITE) in 1989. This "toolbox" cites, as a primary cause of traffic congestion, the increasing number of individuals commuting by automobile in metropolitan areas, to and from locations dispersed throughout a wide region, and through areas where adequate highway capacity does not exist. The specific actions that can be taken to improve the situation are categorized under five components as follows:

1) Getting the most out of the existing highway system
    - Intelligent Transportation Systems (ITS)
    - urban freeways (ramp metering, HOV's)
    - arterial and local streets (super streets, parking management)
    - enforcement
2) Building new capacity (new highway, reconstruction)
3) Providing transit service (paratransit service, encouraging transit use)
4) Managing transportation demand
    - strategic approaches to avoiding congestion (road pricing)
    - mitigating existing congestion (ridesharing)
5) Funding and institutional measures
    - funding (fuel taxes, toll roads)
    - institutional measures (transportation management associations)

Table 7.1
Federal Emission Standards

	1970 Standards (grams/km)	1990 Standards (grams/km)
Light Duty Vehicles[1] (0-3, 340 Kgs)		
Carbon Monoxide (CO)	2.11	2.11
Hydrocarbons (HC)	0.25	0.16
Oxides of Nitrogen (NOx)	0.62	0.25[2]
Particulates	0.12	0.05
Light Duty Vehicles[1] (1,700-2,600 Kgs)		
Carbon Monoxide (CO)	2.11	2.73
Hydrocarbons (HC)	0.25	0.20
Oxides of Nitrogen (NOx)	0.62	0.44[2]
Particulates	0.12	0.05

	1970 Standards (grams/km)	1990 Standards (grams/km)
Light Duty Trucks (over 2,600 Kgs GVWR)		
Carbon Monoxide (CO)	6.22	3.11
Hydrocarbons (HC)	0.50	0.24
Oxides of Nitrogen (NOx)	1.06	0.68
Particulates	0.08	0.05

[1] ***Light duty vehicles include light duty trucks.***
[2] ***The new emission standards specified in this table are for useful life of 5 years or 80,000 Kms whichever first occurs.***

7.3.3 Tailpipe Control Measures

It is clear that in order to achieve the air quality standards and to reduce the pollution from the motor vehicle emissions, substantial additional emission reduction measures are essential. According to the U.S. Office of Technology Assessment (OTA), the mobile source emissions are even higher in most sources - responsible for 48 percent of VOCs in non-attainment U.S. cities in 1985, compared to other individual source categories (Walsh 1989).

The following set of emission control measures, if implemented, has the potential to substantially decrease exhaust emissions from major air pollutants:

- limiting gasoline volatility to 62.0 KPa (9.0 psi) RVP;
- adopting "onboard" refueling emissions controls;
- "enhanced" inspection and maintenance programs; requiring "onboard diagnostics" for emission control systems;
- adopting full useful life (160,000 kms) requirements;

- requiring alternative fuel usage;
- utilizing oxygenated fuels/reformulated gasoline measures;
- adopting California standards (Tier I and II).

Some of the measures recommended in California standards include: improved inspection and maintenance such as "centralized" inspection and maintenance programs, heavy duty vehicle smoke enforcement, establishing new diesel fuel quality standards, new methanol-fueled buses, urban bus system electrification, and use of radial tires on light duty vehicles.

In the case of diesel-fueled LDT's (0-3,750 lvw) and light-duty vehicles, before the model year 2004, the applicable standards for NOx shall be 0.62 grams/km for a useful life as defined above.

7.3.4 Highway Air Quality Models

As discussed earlier, federal, state, and local environmental regulations require that the air quality impacts of transportation - related projects be analyzed and be quantified. For this purpose, the Federal Highway Administration (FHWA) has issued guidelines to ensure that air quality effects are considered during planning, design, and construction of highway improvements, so that these plans are consistent with State Implementation Plans (SIPs) for achieving and maintaining air quality standards .

For example, the level of CO associated with a given project is a highway - related air quality impact that requires evaluation. In general, it must be determined whether the ambient standards for CO (35 ppm for 1 hour and 9 ppm for 8 hours, not to be exceeded more than once per year) will be satisfied or exceeded as a result of highway improvements. This requirement calls for estimating CO concentrations on both local and areawide scales.

A number of methods of varying sophistication and complexity are used to estimate air pollutant levels. These techniques include simple line-source-oriented Gaussian models as well as more elaborate numerical models (TRB 1981). The databases used in most models could be divided into the following categories:

- meteorological data
  - wind speed and direction
  - temperature
  - humidity
  - wind and temperature fluctuations
- traffic data
  - traffic volume
  - vehicle speed
  - vehicle length or type
- site types
  - at-grade sites
  - elevated sites
  - cut sites
- period of measurement

Some of these models which estimate the pollutant emissions from highway vehicles are discussed in more detail in the following sections.

7.3.4.1 UMTA Model

The most simple of these air quality models is the one which relates vehicular speeds and emission levels (USDOT 1985). The procedure is not elaborate but is a quick-response technique for comparison purposes. This UMTA (now Federal Transit Administration) model contains vehicular emission factors related to speed of travel for freeways and surfaced arterials.

The model uses a combination of free flow and restrained (peak period) speeds. It assumes that one-third of daily travel would occur in peak hours of flow reflecting restrained (congested) speeds, while two-thirds would reflect free-flow speed characteristics. For a complete table of composite emission factors categorized by autos and trucks for two calendar years of 1987 and 1995 refer to Characteristics of Urban Transportation System, U.S. Department of Transportation, Urban Mass Transportation Administration, October 1985.

7.3.4.2 CALINE-4 Dispersion Model

This line source air quality model has been developed by the California Department of Transportation (FHWA 1984). It is based on the Gaussian diffusion equation and employs a mixing zone concept to characterize pollutant dispersion over the roadway.

The model assesses air quality impacts near transportation facilities given source strength, meteorology, and site geometry. CALINE-4 can predict pollution concentrations for receptors located within 500 meters of the roadway. It also has special options for modeling air quality near intersections, street canyons, and parking facilities.

CALINE-4 uses a composite vehicle emission factor in grams per vehicle-mile and converts it to a modal emission factor. The Environmental Protection Agency (EPA) has developed a series of computer programs, the latest of which is called Mobile4.1 (EPA 1991), for estimating composite mobile emission factors given average route speed, percent cold and hot-starts, ambient temperature, vehicle mix, and prediction year. These emission factors are based on vehicle distribution weighted by type, age, and operation mode, and were developed from certification and surveillance data, mandated future emission standards, and special emission studies.

Composite emission factors represent the average emission rate over a driving cycle. The cycle might include acceleration, deceleration, cruise, and idle modes of operation. Emission rates specific to each of these modes are called modal emission factors. The speed correction factors used in composite emission factor models, such as MOBILE4, are derived from variable driving cycles representative of typical urban trips. The Federal Test Procedures (FTP) for driving cycle are the basis for most of these data.

Typical input variables for the CALINE-4 model are shown in Table 7.2. In case of an intersection, the following assumptions are made for determining emission factors:

- uniform vehicle arrival rate;
- constant acceleration and deceleration rates; constant time rate of emissions over duration of each mode;
- deceleration time rate of emissions equals 1.5 times the idle rate;
- an "at rest" vehicle spacing of 7 meters; and
- all delayed vehicles come to a full stop.

In addition to composite emission factor at 26 km/hr (EFL), the following variables must be quantified for each intersection link:

- arrival volume in vehicles per hour;
- departure volume in vehicles per hour;
- average number of vehicles per cycle per lane for the dominant; movement
- average number or vehicles delayed per cycle per lane for the dominant movement (NDLA);
- distance from link endpoints to stopline;
- acceleration and deceleration times (ACCT, DCLT);
- idle times at front and end of queue;
- cruise speed (SPD); and
- idle emission rate (EFI).

The following computed variables are determined for each link from the input variables:

- acceleration rate;
- deceleration rate;
- acceleration length;
- deceleration length;
- acceleration-speed product;
- FTP-75 (BAG2) time rate emission factor;
- acceleration emission factor;
- cruise emission factor;
- deceleration emission factor; and
- queue length (LQU=NDLA*VSP), where VSP is the "at rest" vehicle spacing.

The cumulative emission profiles (CEP) for acceleration, deceleration, cruise, and idle modes form the basis for distributing the emissions. These profiles are constructed for each intersection link, and represent the cumulative emissions per cycle per lane for the dominant movement. The CEP is developed by determining the time in mode for each vehicle during an average cycle/lane event multiplied by the modal emission time rate and summed over the number of vehicles. CALINE4 can predict concentrations of relatively inert pollutants such as carbon monoxide (CO), and other pollutants like nitrogen dioxide (NO2) and suspended particles.

7.3.4.3 Mobie Source Emission Factor Model

MOBILE4.1 is the latest version of mobile source emission factor model developed by the U.S. Environmental Protection Agency (EPA). It is a computer program that estimates hydrocarbon (HC), carbon monoxide (CO), and oxides of nitrogen (NOx) emission factors for gasoline-fueled and diesel-fueled highway motor vehicles.

MOBILE4.1 calculates emission factors for eight vehicle types in two regions (low- and high-altitude). Its emission estimates depend on various conditions such as ambient temperature, speed, and mileage accrual rates. MOBILE4.1 will estimate emission factors for any calendar year between 1960 and 2020.

Table 7.2
Standard Input Values for the CALINE4

I. Site Variables	
Temperature	©
Wind Speed	(m/s)
Wind Direction	(deg)
Directional Variability	(deg)
Atmospheric Stability	(F)
Mixing Height	(m)
Surface Roughness	(cm)
Settling Velocity	(m/s)
Deposition Velocity	(m/s)
Ambient Temperature	©
II. Link Variables	
Traffic Volume	(veh/hr)
Emission Factor	(grams/veh-mile)
Height	(m)
Width	(m)
Link Coordinates	(m)
III. Receptor Locations	(m)

The 25 most recent model years are considered to be in operation in each calendar year. It is to be used by the states in the preparation of the highway mobile source portion of the 1990 base year emission inventories required by the Clean Air Act Amendments of 1990.

MOBILE4.1 calculates emission factors for gasoline-fueled light-duty vehicles (LDVs), light-duty trucks (LDTs), heavy-duty vehicles (HDVs), and motorcycles, and for diesel LDVs, LDTs, and HDVs. It also includes provisions for modeling the effects of oxygenated fuels (gasoline-alcohol and gasoline- ether blends) on exhaust CO emissions. Some of the primary input variables and their ranges are discussed below.

Speed correction factors are used by the model to correct exhaust emissions for average speeds other than that of the FTP (32 km/hr). MOBILE4.1 uses three speed correction models: low speeds (4-32 km/hr), moderate speeds (32-77 km/hr), and high speeds (77-105 km/hr). The pattern of emissions as a function of vehicle speed is similar for all pollutants, technologies, and model year groups. Emissions are greatest at the minimum speed of 4 km/hr, decline relatively rapidly as speeds increase from 4 to 32 km/hr, decline more slowly as speeds increase from 32 to 77 km/hr, and then increase with increasing speed to the maximum speed of 105 km/hr.

The vehicle miles traveled (VMT) mix is used to specify the fraction of total highway VMT that is accumulated by each of the eight regulated vehicle types. The VMT mix is used only to calculate the composite emission factor for a given scenario on the basis of the eight vehicle class-specific emission factors. Considering the dependence of the calculated VMT mix on the annual mileage accumulation rates and registration distributions by age, EPA expects that states develop their own estimates of VMT by vehicle type for specific highway facility, sub-zones, time of day, and so on.

Many areas of the country have implemented inspection and maintenance (I/M) programs as a means of further reducing mobile source air pollution. MOBILE4.1 has the capability of modeling the impact of an operating I/M program on the calculated emission factors, based on user specification of certain parameters describing the program to be modeled. Some of the parameters include:

- program start year and stringency level;
- first and last model years of vehicles subject to program;
- program type (centralized or decentralized);
- frequency of inspection (annual or biennial); and
- test types.

MOBILE4.1 (EPA 1991) has the ability to model uncontrolled levels of refueling emissions as well as the impacts of the implementation of either or both of the major types of vehicle recovery systems. These include the "Stage II" (at the pump) control of vehicle refueling emissions or the "onboard" (on the vehicle) vapor recovery systems (VRS).

The minimum and maximum daily temperatures are used in MOBILE4.1 in the calculation of the diurnal portion of evaporative HC emissions, and in estimating the temperature of dispensed fuel for use in the calculation of refueling emissions. The minimum temperature must be between -18 C to 38 C (0 F and 100 F), and the maximum temperature must be between -12 C to 49 C (10 F and 120 F) inclusive.

The value used for calendar year in MOBILE4.1 defines the year for which emission factors are to be calculated. The model has the ability to model emission factors for the year 1960 through 2020 inclusive. The base year (1990) inventories are based on a typical day in the pollutant season, most commonly Summer for ozone and Winter for CO. The base year HC inventories are based on interpolation of the calendar year 1990 and 1991 MOBILE4.1 emission factors.

One important determinant of emissions performance is the mode of operation. The EPA's emission factors are based on testing over the FTP cycle, which is divided into three segments or operating modes: cold start, stabilized, and hot start. Emissions generally are highest when a vehicle is in the cold-start mode: the vehicle, engine, and emission control equipment are all at ambient temperature and thus not performing at optimum levels. Emissions are generally somewhat lower in hot start mode, when the vehicle is not yet completely warmed up but has not been sitting idle for sufficient time to have cooled completely to ambient temperature. Finally, emissions generally are lowest when the vehicle is operating in stabilized mode, and has been in continuous operation long enough for all systems to have attained relatively stable, fully "warmed-up" operating temperatures.

The EPA has determined through its running loss emission test programs that the level of running loss emissions depends on several variables: the average speed of the travel, the ambient temperature, the volatility (RVP) of the fuel, and the length of the trip. "Trip length" as used in MOBILE4.1 refers to the duration of the trip (how long the vehicle has been traveling), not on the distance traveled in the trip (how far the vehicle has been driven). Test data show that for any given set of conditions (average speed, ambient temperature, and fuel volatility), running loss emissions are zero to negligible at first, but increase significantly as the duration of the trip is extended and the fuel tank, fuel lines, and engine become heated.

7.3.4.4 MICRO2

MICRO2 is an air quality model which computes the air pollution emissions near an intersection. The concentration of the pollutants in the air around the intersection is not computed. In order to determine the pollution concentration, a dispersion model which takes weather conditions such as wind, speed, and direction into account should be used (Richards 1983).

MICRO2 bases its emissions on typical values of the FTP performed for Denver, Colorado in the early 1980s. They are:

- FTP (1) HC 6.2 grams/veh/km
- FTP (2) CO 62.2 grams/veh/km

- FTP (3) NOx 1.2 grams/veh/km

For lower than Denver altitudes or years beyond 1980's, emission rates may be lower and should change from these initial values.

The emission formulas as a function of acceleration and speeds are as follow:

HC Emission (gram/sec) =
$$0.018 + 5.668*10^{-3} (A*S) + 2.165*10^{-4} (A*S^2) \quad (7.17a)$$

CO Emission (gram/sec) =
$$0.182 - 8.587*10^{-2} (A*S) + 1.279*10^{-2} (A*S^2) \quad (7.17b)$$

NOx Emission (gram/sec) =
$$3.86*10^{-3} + 8.767*10^{-3}(A*S) \text{ (for } A*S>0) \quad (7.17c)$$

NOx Emission (gram/sec) =
$$1.43*10^{-3} - 1.830*10^{-4}(A*S) \text{ (for } A*S<0) \quad (7.17d)$$

where,

A = acceleration (meters/sec^2) and
S = speed (meters/sec).

7.3.4.5 The TRRL Model

This model has been developed by the British Transport and Road Research Laboratory (TRRL) and it predicts air pollution from road traffic (Hickman and Waterfield 1984). The estimations of air pollution are in the form of hourly average concentrations of carbon monoxide at selected locations around a network of roads. The input data required are the configuration of the road network, the location of the receptor, traffic volumes and speeds, wind speed, and wind direction.

The concentration of carbon monoxide may be used as to approximate the likely levels of other pollutants using the following relations:

$$HC\ (ppm) = 1.8\ CO\ (ppm) * R + 4.0 \quad (7.18a)$$

$$NOx\ (ppm) = CO\ (ppm) * R + 0.1 \quad (7.18b)$$

where R is the ratio of pollutant emission rate to that of carbon monoxide for a given mean vehicle speed,

Mean Speed (km/hr)	NOx (ppm)	HC (ppm)
20	0.035	0.205
30	0.050	0.240
40	0.070	0.260
50	0.085	0.280
60	0.105	0.290
70	0.120	0.305

A quick but less accurate estimate of the annual maximum 8 hour CO concentration can also be obtained from the average peak hour CO concentration estimate, as follows:

$$C_8 = 1.85\ C_1 + 1.19 \quad (7.19)$$

where C_8 is the annual maximum 8 hour concentration, and C_1 is the average peak hour concentration.

A graphical screening test is introduced by which any properties likely to experience an air pollution problem are identified. The procedure first reduces the network to a system of long roads and roundabouts (if any). Then from a graph, the concentration of carbon monoxide for standard traffic conditions for locations at any distance from each network element may be determined. Factors are then applied to adjust for the traffic conditions at the site and the sum of the contributions from each element gives an estimate of the likely average peak hour concentration. An example of the graphical screening test results is shown in Table 7.3.

7.3.5 Other Mobile Source Air Quality Models

There are many other mobile source models which estimate the pollutant emission rates and concentrations near highway and arterial streets. Most of these models relate vehicle speeds and other variables such as vehicle year model, ambient temperature, and traffic conditions to emission rates. A common example of this type of relation could be found in Technical Advisory #T6640.10 of EPA report, "Mobile Source Emission Factor Tables for MOBILE3." Other popular models include HIWAY2 and CAL3QHC. HIWAY2 model has been developed by U.S. EPA to estimate hourly concentrations of non-reactive pollutants, like CO, downwind of roadways. It is usually used

Table 7.3
Graphical Screening Test Results for Existing Network

Distance from Centerline (m)	13	45	53	103
CO for 1000 vehs/hr at 100 km/h (ppm)	1.08	0.49	0.40	0.11
Traffic Flow (vehs/hr)	2,600	1,400	800	400
Speed (km/hr)	40	20	20	50
Speed Correction Factor	2.07	3.59	3.59	1.73
CO for Actual Traffic Conditions (ppm)	5.81	2.46	1.15	0.08
Total 1-Hour CO = 9.50(ppm) Equivalent Annual Maximum 8-Hour = 18.76(ppm)				

for analyzing at- grade highways and arterials in uniform wind conditions at level terrain as well as at depressed sections (cuts) of roadways. The model cannot be used if large obstructions such as buildings or large trees hinder the flow of air. The simple terrain requirement makes this model less accurate for urban conditions than CALINE4 type of models.

The CAL3QHC model has the ability to account for the emissions generated by vehicles traveling near roadway intersections. Because idling emissions account for a substantial portion of the total emissions at an intersection, this capability represents a significant improvement in the prediction of pollutant concentrations over previous models. This EPA model is especially designed to handle near-saturated and/or over-capacity traffic conditions and complex intersections where major roadways interact through ramps and elevated highways. The model combines the CALINE3 line source dispersion model with an algorithm that internally estimates the length of the queues formed by idling vehicles at signalized intersections. The inputs to the model includeinformation and data commonly required by transportation models such as roadway geometries, receptor locations, vehicular emissions, and meteorological conditions. Emission factors used in the model should be obtained from mobile source emission factor models such as MOBILE4.

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