Analysis of Corridor Traffic Peaking

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In the past, many traffic forecasting efforts have focused on estimating 24-hr volumes, leaving the determination of design-hour volumes up to highway designers. These days, planners are becoming more involved in estimating peak-hour volumes, but the available techniques for calculating peak-hour traffic are somewhat limited. In particular, it is often assumed that the percentage of daily traffic occurring in the peak hour will not change in the future. In addition, planners sometimes forget that, for a given link, the peak-hour volume cannot exceed the link's capacity, regardless of the increase in daily volume. It is hypothesized that, as 24-hr traffic volumes continue to increase, peak-hour volumes will not increase at the same rate. In fact, future roadway capacity limitations (as well as other factors) will force drivers to modify their trip departure times, most likely so as to travel in the shoulders of the peak. Other researchers have also hypothesized this, but a literature review disclosed no practical methods for forecasting the future flattening or shifting of the peak hour on a link-specific basis in response to increased congestion. A methodology is presented for projecting such a change in temporal patterns, on the basis of research conducted in the I-80 corridor in northern New Jersey. This technique uses a modified Poisson distribution to describe the spread of 4-hr volumes across each 15-min period. Although the resulting model structure is not free of flaws, it represents a reasonable attempt to estimate future changes in peaking and will hopefully stimulate further research into this important subject.

Many efforts to predict future traffic volumes have focused on estimating the travel demand during the single peak hour. Early planning studies only provided values for 24-hr demand, leaving it up to highway designers to apply their own *K* and *D* factors to estimate peak-hour volumes by direction. More recently, planners are themselves taking on the task of estimating peak-hour traffic.

The ratio of future peak-hour volume to roadway capacity is commonly used as a key indicator of highway system performance and has always been an important part of the justification for expanding facilities. However, planners are often accused of ignoring a simple rule of transportation systems: demand cannot exceed capacity for a given time period. As a theoretical illustration of travelers' desire to use a particular facility, volume/capacity (V/C) ratios over 1 can be useful. However, design-hour volumes in excess of capacity have no basis in reality.

In the present era, in which inexorable future increases in traffic run hard up against the immovable budget limitations of many jurisdictions, the planner is still called on to provide realistic predictions of peak-hour traffic. It is theorized that, in locations where volumes are already at capacity, such volumes cannot increase on the existing roadway system. Without new capacity, increased travel demand must occur outside the peak hour, most likely in the shoulders of the peak.

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This concept has interesting implications for roadway design. If roadways are to be sized on the basis of peak-hour demand, and the peak volume does not increase but merely spreads over a longer period, why should roads be widened? Indeed, shouldn't peak spreading be encouraged, as a better use of existing facilities? Of course, this logic is circular: it is not necessary to expand roads because the peak volume will spread itself out due to congestion, which occurs because the decision not to expand the roads was made in the first place.

However, a philosophical discourse on the need for new roadways is not intended. Rather, a practical question at the heart of this dilemma is investigated: How can planners forecast the degree to which traffic growth might be served in the shoulders of the peak? A methodology for predicting this phenomenon is examined, using data from a recent study of the I-80 corridor in northern New Jersey.

STUDY OVERVIEW

Interstate 80 stretches across the entire northern part of New Jersey, from the Delaware Water Gap at the Pennsylvania line to the Hudson River at the New York line (see Figure 1). It is a major commuting and trucking route, serving both intraregional and intercity travel needs. The study reported on was concerned with the outer section of the road, from the Delaware River eastward to a point just west of I-287, in the middle of New Jersey. Due mainly to recent development patterns, traffic congestion has become as big a problem in this outer section as in the section closer to New York City. On many days, some of the most congested points along I-80 are those located 35 mi from the city.

In 1989 the New Jersey Department of Transportation (NJDOT) commissioned a study of methods that might be used to reduce congestion now and in the future, given the tremendous growth in population and employment expected to occur in the study area. Some parts of the road are currently being widened, but NJDOT anticipates that the options for reconstruction will be more restricted in the future, due to limitations in funding and available right-of-way. Thus, the study investigated alternative means of reducing vehicle trip demand, including expansion of existing bus and rail services and ways to increase high-occupancy vehicle (HOV) usage.

At the outset of the study, it was decided that a key measure of the success of these alternatives should be the degree to which they reduce the peak-hour vehicular volume at the major choke points along I-80. However, a parallel objective was to determine to what extent drivers might voluntarily shift their travel away from the peak 60 min in response to future conditions. Thus, a two-phase approach was taken to forecasting traffic volumes. The first phase is to estimate the total

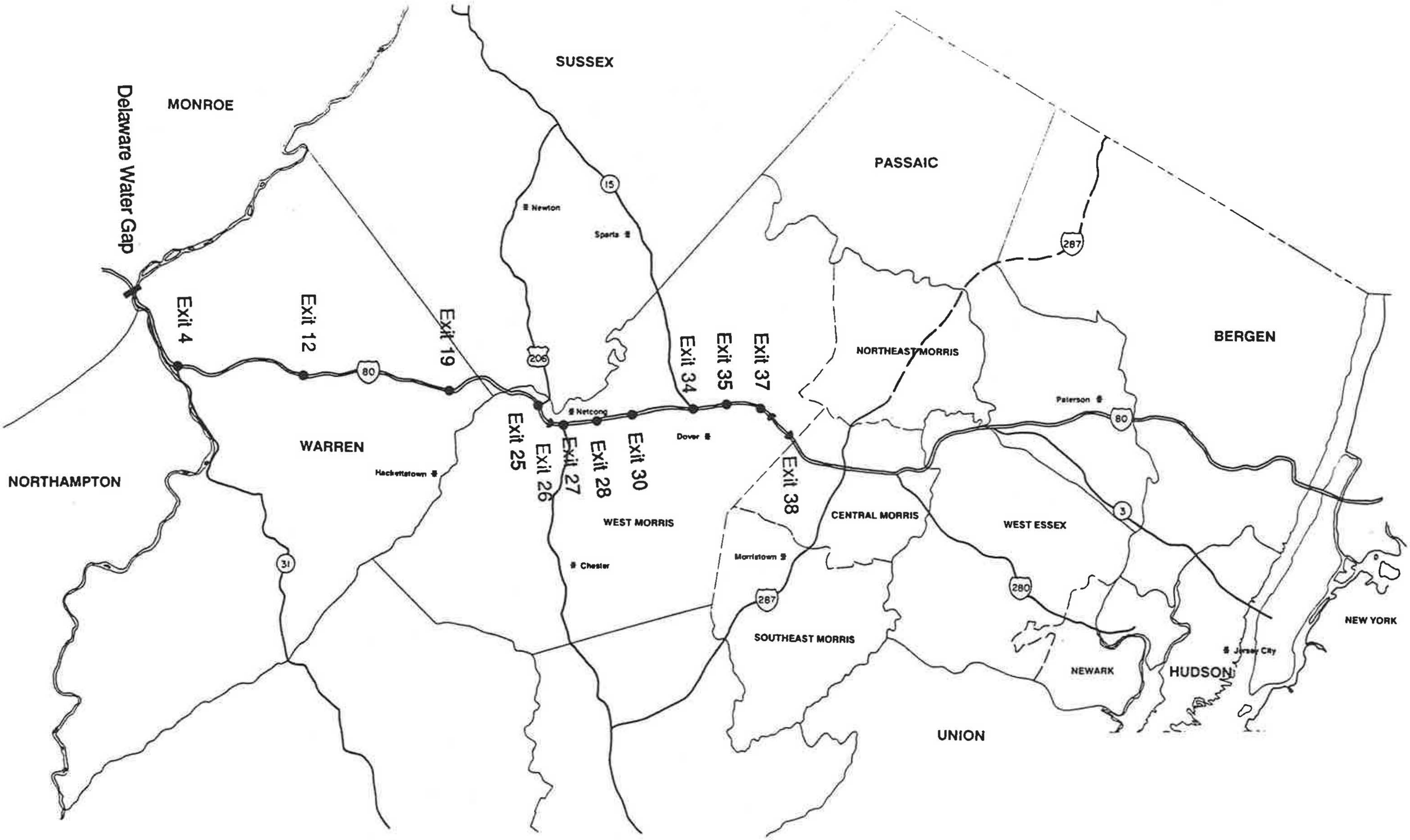

FIGURE 1 Study area.

volume from 6:00 to 10:00 a.m., which is the 4-hr period when the greatest level of traffic activity is seen in the corridor. The second phase is to estimate the highest consecutive 60-min volume during that time.

Many studies of future traffic assume that current peaking patterns will remain stable in the future. However, this study's investigators wanted to examine the premise that the future could bring noticeable changes in the temporal variation in a.m. traffic volumes. Increased peak-period congestion, changing lifestyles, and expanded use of flexible working hours might well lead to changes in peaking patterns over time. In addition, the changing development pattern along the corridor could by itself lead to a change in when the peak 60 min will occur at any given point (i.e., the rolling peak). It was thus necessary to develop a model that could be used to estimate the possible extent of the shift in peak traffic flow in the future.

To better understand corridor traffic flow and to support the development of this model, an extensive roadside survey was undertaken in the summer of 1989. Over 33,000 postage-paid postcards were distributed to all vehicles entering I-80 between 6:00 and 10:00 a.m., at all points between the Delaware Water Gap (Exit 0) and US-46 (Exit 38). Over 41 percent of these postcards were returned, providing information on trip origin, destination, purpose, vehicle occupancy, and other roadways used. Sufficient survey controls were maintained so that the time each driver entered I-80 was known by 15-min period. From this information, a data base was assembled providing trip origin and destination by 15-min period for each exit-to-exit segment along I-80. This information was then used to develop both future estimates of eastbound a.m. 4-hr traffic as well as the volume during the peak 60-min period, for each roadway link in the study area.

THEORY

It was hypothesized that highway peaking patterns in the I-80 corridor are influenced mainly by two things: the extent to which employees have flexible working hours (flex-time) and the level of traffic congestion. The extent of flex-time is a policy variable that indicates how many employees have enough variability in their work start time that they can travel to work at nonstandard times. Although this variable is intuitively important, little information is available to describe the extent of flex-time in this corridor. Moreover, it is not known if future values of this variable could be predicted on a corridorwide basis with any suitable accuracy.

The use of flex-time by employers is probably related to the type of employment, location, competition with other businesses for employees, individual employer philosophies, and other factors that are basically outside the realm of transportation planning. Thus, this variable was considered essentially unavailable for use.

Because of these and other related reasons, attention was focused on the other key factor, traffic congestion. Congestion has increased sharply in recent years in northern New Jersey and is frequently identified as a major problem in surveys of the motoring public. Thus, it was hypothesized that a prime reason individuals in the I-80 corridor vary their commuting times is to try to avoid the most congested periods. After all, even without flex-time, most employees do have the option of arriving at their jobs before their normal start time. Also, many morning nonwork travelers can delay the start of their trip until after the main peak period.

Another reason for focusing on congestion is that it is readily quantified for existing conditions and can reasonably be forecast for the target future year. These attributes make it suitable for use as a key independent variable in this model structure.

At first glance, this reasoning may seem like circular logic: peaking is dependent on congestion, which is a function of peaking. However, careful definition of the variables ensures that this situation will not be the case. At a micro level, congestion is a function of 5- or 10-min flow rates and the presence of accidents or other roadside incidents. But in the context of this study, congestion was defined in a larger sense as a function of the total traffic activity on I-80 during the 4-hr peak period. The dependent variable is the percentage of that 4-hr volume that occurs in each 15-min segment. This method is analogous to the long-standing practice among highway engineers of estimating the ratio of peak hour to average daily traffic (ADT) as a function of the ADT itself. Various ways of measuring this congestion effect were investigated.

In addition, the analysis considered other variables that are related to the relative distribution of households and jobs in the corridor. It was hypothesized that, in the future, the concentration of jobs will likely move westward, following the growth in housing in the western part of the corridor. This shift, in turn, would affect the timing of work trips on I-80. Thus, it was judged that a variable measuring this distribution effect might explain some of the base-year variation from link to link. (Ideally, this kind of analysis would benefit greatly from a longitudinal data base, which would capture the same information at two or more points in time. However, the study schedule did not permit this flexibility in data collection.)

The candidate independent variables are described in more detail in the following sections.

MODEL STRUCTURE

Previous Research

A brief literature review disclosed little previous research on this subject, although four recent papers were found that considered the problem. Replogle (*1*) estimated peaking patterns in terms of population and employment density, as a surrogate for congestion. This approach was interesting, and it highlighted the difficulty of measuring and forecasting peaking due to the variety of causal factors. Marshment and Sulsky (*2*) assumed different levels of peak curve flattening by freeway ramp location and travel direction. Although they stated the problem in clear terms, their analysis made rather broad assumptions about the nature of future peaking patterns, rather than trying to mathematically estimate such patterns. Kroes and Hamerslag (*3*) described a theoretical approach to modeling the impact of congestion inside the assignment process, using a logit model of trip departure times. They limited themselves to discussing how their approach could be accomplished in conceptual terms and did not attempt to provide a workable mechanism for its implementation.

In a particularly relevant effort, Loudon et al. (*4*) examined this phenomenon to the extent of developing a model of the ratio of the peak-hour volume to be 3-hr peak-period volume. Their research did conclude that peak-period congestion clearly affects this ratio and that a recognizable pattern of peak spreading does occur. One limitation of the model is that it is applied on a facility-type basis, so that discontinuities can (and probably will) occur in the predicted peak volumes for adjacent links on a roadway. The model also does not consider that the peaking on any one link might be influenced by congestion effects that occur elsewhere on the roadway. In any event, Loudon et al. advance the state of research into peak-hour spreading and provide some of the motivation for the more ambitious methodology examined here.

All of these efforts acknowledged the need to better understand peaking patterns as they affect the accuracy of traffic forecasts, but they also all fell somewhat short of providing a practical approach that could be readily adapted to produce usable results for the I-80 study.

Model Formulation

The model to be developed in this study was intended to calculate the change in the temporal distribution of a.m. peak-period traffic, given estimated changes in total peak-period traffic volume and other measurable traffic characteristics. In its most basic sense, the model must determine how the base year curve (dashed line) in Figure 2 might change over time into the future curve (solid line).

The unknown variable to be estimated in this case is not a single number, but rather the distribution described by the dashed line itself. Once the distribution is determined, the peak 60-min volume can be readily calculated. The literature review disclosed little about the inherent form of this curve, only that it should be expected to become flatter over time in response to increased flex-time and increased congestion.

In addition to becoming flatter, it is of interest to investigate to what extent the curve might also shift to the right or left in response to these changes.

Upon reviewing the observed data and other sources of information, the Poisson function was identified as being sufficiently descriptive of the distribution of 1989 traffic volumes by time period. The basic Poisson distribution is described as follows:

$$P(x) = \frac{m^x * e^{-m}}{x!} \tag{1}$$

where

m = arithmetic mean of the distribution,
x = time period under study, and
$P(x)$ = probability of an event occurring in Period x.

The selection of this function is further supported by observing that the Poisson distribution is often used in traffic engineering to represent a variety of phenomena, such as queueing and distribution of speeds. Thus, it was judged appropriate for use. To use the Poisson function in this context, Equation 1 was modified as shown below:

$$P(s,x) = \frac{z * m^{(x+y)} * e^{-m}}{(x + y)!} + a \tag{2}$$

where

s = eastbound highway segment (from 1 to 13);
x = time period (1 = 6:00–6:15, 2 = 6:15–6:30, . . . , 16 = 9:45–10:00);
m = primary shape factor;
a, z, y = other scale coefficients; and
$P(s,x)$ = proportion of 4-hr traffic occurring in Period x on Link s.

This structure is described further in the following sections.

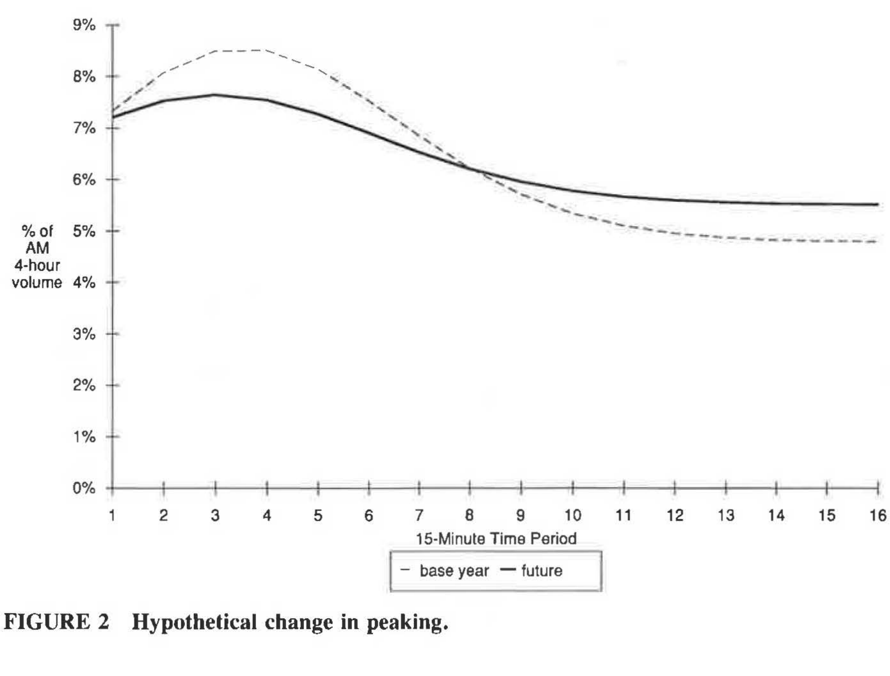

FIGURE 2 Hypothetical change in peaking.

CALIBRATION METHOD

Data Structure

The detailed traffic counts and the highway survey conducted for this project provided the base calibration data for the model. Because of the need to estimate specific peak-hour volumes for each segment of eastbound I-80 from the Delaware Water Gap to Denville, a link-based calibration file was established. The file includes values for the dependent and all available independent variables and consisted of the following data for each of the 13 highway segments under study:

- Milepost of segment's on-ramp (i.e., miles from the Pennsylvania line);
- Segment length;
- Counted 4-hr volume by 15-min period;
- Segment hourly capacity [Level of Service (LOS) E];
- Peak V/C ratio (highest 15-min period);
- Average V/C ratio (averaged over 4 hr);
- Average speed difference (free-flow minus congested, averaged for 4 hr);
- Delay on this link (based on average speed difference);
- Cumulative delay on this link, plus all links 2 mi downstream (i.e., eastbound);
- Cumulative delay on this link, plus all links 5 mi downstream (i.e., eastbound);
- Cumulative delay eastbound from the Delaware Water Gap;
- Average total trip time for all vehicles on this link;
- Average total trip time for all vehicles entering I-80 at the start of this link;
- Average access time to I-80 for all vehicles entering I-80 at the start of this link;
- Four-hour volume of vehicles entering I-80 at the start of this link;
- Percent trucks;
- Percent work trips;
- Percent of jobs within 5, 7, 10, 15, or 20 mi of this link's entry point; and
- Relative location of this link with respect to the whole trip, averaged across all trips using the link.

For these data, times are measured in minutes, distances in miles, and speed in miles per hour. All but the first two items represent data that exist only for the eastbound a.m. peak direction. Some of these variables are explained further in the following paragraphs.

Speed Difference

The standard Bureau of Public Roads (BPR) formula relating speed to V/C ratio was judged unacceptable. The BPR equation is as follows:

$$S = \frac{S(0)}{1 + 0.15 * (V/C)^4} \quad (3)$$

This equation did not provide congested speed values that were consistent with the investigators' observations and travel time runs. An alternative approach was developed on the basis of Figure 16.1 in the *ITE Transportation and Traffic Engineering Handbook* (*5*), using the following equation:

$$S = 65 - (4 * 10^{(V/C)}) \quad (4)$$

(valid only for V/C values of 1.2 or lower)

where V/C is the average a.m. 4-hr V/C ratio, based on capacity at LOS E.

Thus, the difference in speed is imply $4 * 10^{(V/C)}$.

Downstream Delay

It is hypothesized that motorists may adjust their travel times not only in response to congestion on any one link, but by considering congestion downstream from that link as well. This factor is reflected in two statistics: 2-mi and 5-mi downstream delay, which is the sum of the delay on each link plus the links that are 2 or 5 mi east of that link.

Average Trip Time

Average trip time is simply the total trip time (from the study area network), weighted by highway person trips and averaged over all the trips using each highway segment (or all the trips entering the highway at each on-ramp). Thus, it is a function of the origins and destinations of the trips using each link, as well as overall congestion in the corridor.

Average Relative Location

The highway survey contains information on each trip's origin zone, point of entry to and exit from I-80, and destination zone. By examining the relative lengths of the access, line-haul, and egress trip segments, it can be determined whether the I-80 portion is at the beginning, middle, or end of the trip. Averaging this information across all trips on the link might provide a statistic with power to explain the peaking phenomenon.

As noted, the dependent variable in this process consists of 13 sets of temporal distributions, one for each segment. Each distribution consists of the observed proportion of the total 4-hr volume occurring in each of sixteen 15-min periods. By definition, these fractions total to 1 for each link.

The calibration process assumes that the 13 highway segments exhibit some variation in temporal patterns and that this variation can somehow be explained by the available independent variables in a way that is both logical and mathematically appropriate. A priori examination of the observed distributions disclosed that they do, in fact, display intuitively rational patterns. The far western segments show a very early peaking (6:00–6:30 a.m.), which occurs later in time as one moves eastward. Also, the more congested segments tend to have a slightly flatter distribution than the less congested ones.

Technique

The sixteen 15-min traffic counts for each link were tabulated and graphed as a proportion of the total 4-hr volume. Next, a modified Poisson curve (Equation 2) was hand-fitted to each of the 13 graphs. This procedure was done by adjusting the *m*, *a*, *y*, and *z* values until the best fit was obtained. The adequacy of this fit was confirmed by checking the chi-square values for each link. The hand-fitted curves were defined in terms of 13 sets of coefficients.

These curves then became the observed data, to be fitted to one Poisson model for all links. The calibration file is shown in Table 1. It consists of the Poisson coefficients *m*, *a*, *y*, and *z* and the available independent variables for each link. The next step was to use those variables to estimate the coefficients.

A correlation table was calculated for all variables to provide a better understanding of the relationships that exist. Numerous two-dimensional graphs were prepared to display the relationships between the coefficients and the independent variables. Most important, the theories described previously were invoked to hypothesize the types of relationships that should exist among the variables.

Regression analysis was used to estimate each coefficient (*m*, *a*, *y*, and *z*) in terms of the available independent variables. Because there was no a priori judgment about these functions and the literature offered little guidance as to the forms, several regression models were tried. These models included linear, exponential, power, logarithmic, and 2nd- and 3rd-order polynomial. Certain transformations of the independent variables were tested as well.

The regression results were evaluated on the basis of the square of the correlation coefficient (r^2), examination of residuals, proper sign on the coefficients, and simplicity. Because the greatest congestion occurs on the segments from Exit 34 to 35 and Exit 35 to 37, those segments were assigned a slightly greater importance in determining the final coefficients.

RESULTS

Coefficients

Table 2 presents the final equations for estimating the four coefficients of Equation 2. These relationships are described in more detail in the following paragraphs. Overall, the functions used to estimate the coefficients are logical, use input variables that can be forecast, and have the proper signs on the coefficients.

m

The factor *m* is the most critical because it determines the basic shape of the curve. Lower values both increase the slope

TABLE 2 FINAL COEFFICIENT FUNCTIONS

Coefficient	Function	r^2
m =	$7.4613 + 0.2382 * \Delta S - 0.5593 * D_5 + 0.1930 * MP$	0.93
	ΔS = speed difference (mph) $= 4 * [10^{(V/C)}]$ for $V/C \le 1.2$ D_5 = total 5-mile downstream delay MP = 27 - entry milepost number, for mileposts 25-34 only (else 0)	
a =	$\dfrac{\Delta V}{20.9558 * \Delta V - 507.8082}$	0.55
	ΔV = ABS(4-hour volume - 15000)	
y =	$INT(4.4367 + 0.0622 * \Delta S)$	0.88
z =	$0.0794 * (\Delta V^{0.1399})$	0.70

TABLE 1 CALIBRATION FILE

Exit Number	0	4	12	19	25	26	27	28	30	34	35	37	38
Model Coefficients:													
a	0.047	0.050	0.047	0.048	0.048	0.048	0.049	0.049	0.050	0.055	0.051	0.046	0.045
y	5	5	5	5	5	5	5	5	5	6	6	6	6
z	0.290	0.260	0.300	0.265	0.265	0.250	0.240	0.250	0.240	0.140	0.210	0.275	0.295
m	8.9	8.0	8.8	9.3	9.4	9.9	9.9	9.2	8.9	9.6	10.5	12.3	12.0
Distance to Next Exit (mi.)	4.90	7.42	7.81	5.34	1.00	0.97	1.80	1.68	3.47	1.26	2.38	1.22	0.69
Speed Difference (mph)	7.8	6.2	7.1	8.2	10.2	10.4	13.8	11.5	11.7	21.3	23.3	28.2	22.2
Average V/C	0.29	0.19	0.25	0.31	0.41	0.42	0.54	0.46	0.47	0.73	0.77	0.85	0.74
Peak V/C	0.40	0.28	0.34	0.42	0.54	0.55	0.70	0.60	0.62	0.88	0.95	1.02	0.91
Average Delay (min.)	0.62	0.73	0.88	0.71	0.17	0.17	0.45	0.33	0.71	0.57	1.23	0.86	0.33
Delay for 5 Mi. Downstream	0.62	0.73	0.88	0.71	1.12	1.66	1.49	1.04	1.27	2.99	4.15	2.92	2.06
Delay for 2 Mi. Downstream	0.62	0.73	0.88	0.71	0.34	0.62	0.45	0.33	0.71	1.79	1.23	1.19	2.06
Average Trip Time (min.)	66.9	70.6	66.5	62.3	59.5	54.8	54.1	53.4	53.4	54.7	55.0	53.9	53.9
4-Hour Link Volume	4320	3923	5026	6382	8328	11317	10959	10331	10549	14819	15613	19120	16798
4-Hr. On-Ramp Volume	0	809	1260	1989	2336	2989	2057	2601	1217	6702	1893	4926	1462
Total Delay (min.)	2667	2847	4411	4555	1438	1936	4899	3437	7448	8404	19169	16455	5555
Tot. Cumulative Delay	2667	5514	9925	14479	15917	17853	22752	26189	33637	42041	61209	77664	83220
% Truck Trips	10.4%	9.8%	7.8%	6.4%	5.2%	4.2%	4.5%	4.7%	4.7%	4.2%	4.2%	3.6%	4.0%
% Work Trips	85.5%	85.1%	85.9%	87.2%	88.3%	88.7%	88.6%	88.2%	87.8%	88.7%	89.7%	90.2%	89.9%
Avg. Time This On-Ramp	51.5	49.5	52.0	47.4	49.1	40.6	44.2	42.7	39.3	51.2	44.6	46.9	39.4
Avg. Time to Access Ramp	8.3	15.8	22.7	17.4	16.5	13.1	14.2	12.3	11.1	22.8	16.1	18.9	9.4
% Jobs 15 Min. From Link	0.2	0.8	1.0	1.2	3.4	4.2	4.3	4.0	5.9	6.0	6.8	7.4	11.5
% Jobs 20 Min. From Link	0.8	1.2	3.9	4.7	4.7	6.7	6.4	7.7	10.9	13.2	14.5	15.7	19.0

of the curve and shift it to the left (i.e., earlier), whereas larger values flatten the distribution and shift it to the right. The best fit for this coefficient is as a function of the link's 4-hr average speed difference, the cumulative 5-mi downstream delay, and a dummy variable indicating the link's location. As the speed difference increases (i.e., greater congestion), m increases, flattening the peak demand, which is logical.

The negative coefficient on 5-mi total delay is somewhat less intuitive. This factor suggests that, as the 5-mi delay increases, m decreases, making the curve more peaked. However, it also means that increasing delay shifts the curve to the left, reflecting a desire to travel earlier to avoid the most congested periods.

Because the average speed difference and the 5-mi total delay measure essentially the same phenomenon and have opposite signs in the equation, it must be concluded that the effect of one variable is partially canceling out the effect of the other. According to the regression results, the speed difference has a much greater beta weight and t value than the 5-mi delay variable. Thus, the speed difference is the more significant of the two variables. A model using speed difference alone was also tested, but it did not give quite as good results as when 5-mi delay was added. Apparently, the 5-mi delay exerts a beneficial influence in counteracting some of the effect of the speed difference.

The final variable in the m equation is a dummy term representing the link's location in the corridor. In the calibration, it was discovered that something other than congestion or the available independent variables was affecting the model's fit from Exit 25 to Exit 34. The correction was a variable that is calculated as 27 minus the milepost of the link's entry point, for link entry points between Exits 25 and 34. It is assumed that this term represents something unique to the geography or physical layout of the road in this area and is thus not projected to change in the future.

y

The only variable in Equation 2 is x, the 15-min time period. In this analysis, the x values range simply from 1 (6:00–6:15) to 16 (9:45–10:00). However, these values do not result in the peak hump being in the proper location. An additional peak-shift factor is needed to help locate the hump in the correct place mathematically. As with m, y is related to the link's speed difference, for the same reasons. As the speed difference increases, y becomes larger, slightly flattening the curve and shifting it to the left. However, because y is in both the numerator and the denominator of the equation, the effect is somewhat muted. The equation for y in Table 2 includes the integer function because y must be an integer value.

z

The factor z can be thought of as a curve scale factor, needed to modify the Poisson function so as to produce values in the 5 to 9 percent range. This factor adjusts the vertical height of the distribution. It is somewhat difficult to attach a precise physical interpretation to this parameter, but it can best be thought of as describing the steepness of the curve. The observed data disclosed a distinct, but unusual, pattern for z: it is high for low-volume links and decreases with increasing volume, up to 15,000 vehicles (in 4 hr). Above that point, it increases again. The regression analysis indicated a fairly good power fit against the absolute value of the 4-hr volume minus 15,000.

The significance of the 15,000 figure could be related to the fact that this volume (and the cusp of the z versus volume curve) occurs on Segment 10 (Exit 34). This segment is the point at which NJ-15, the principal north-south arterial in northwest New Jersey, loads a large volume of eastbound traffic onto I-80 in the morning. It may be that the discontinuity in volume from this point eastward creates some kind of fundamental change in temporal patterns.

a

The coefficient a represents the asymptotic tail end of the distribution (i.e., 9:00–10:00). It is thus the minimum value that any 15-min proportion can take. The statistical analysis indicated that this value is closely, but inversely, related to the z coefficient. Thus, it was decided to use the same independent variable (absolute value of the difference between the 4-hr volume and 15,000) to estimate it.

The resulting nonlinear function is an inverse relationship that says, as the 4-hr volume increases, the value of a decreases. This decrease is sharp until the 4-hr volume reaches 20,000, then it levels off asymptotically to a value of about 0.048.

Observed and Estimated Comparisons

Three of the 13 observed distributions and the final estimated curves (one for each link) are shown in Figures 3–5. These three represent the link with the best fit, the worst fit, and an average fit, respectively. In general, these figures indicate a reasonable overall fit of the model. Table 3 presents the observed and estimated peak 60-min volumes. There is a slight (less than 5 percent) overestimation in the western part of the corridor, balanced by a slight underestimation (less than 5 percent) at the eastern end. However, this bias is not systematic enough to warrant additional external correction. In general, these estimates should be sufficiently accurate for design purposes.

Another comparison is presented in Table 4—the time when the peak 60-min period begins. This time matches exactly in the critical section from Exit 28 to Exit 35, but in general, the estimated start of the peak hour is 15 min later than the observed start. This difference may be due to discontinuities in the observed data.

On the basis of these comparisons, the model is judged to provide an accurate estimation of the temporal distribution of traffic in the I-80 corridor.

Sensitivity Analysis

Although base year observed and estimated comparisons are important in calibration, a true test of any model's forecasting

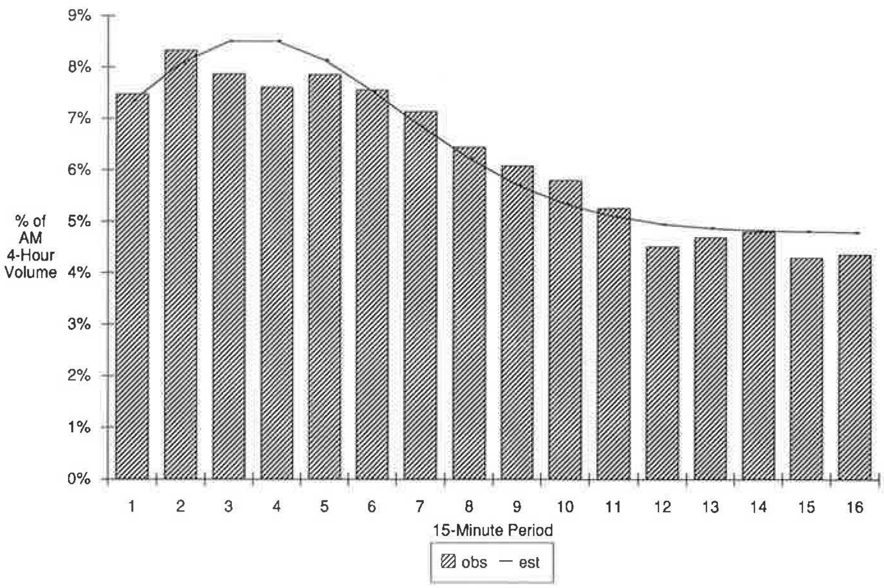

FIGURE 3 Observed versus estimated distribution: best fit.

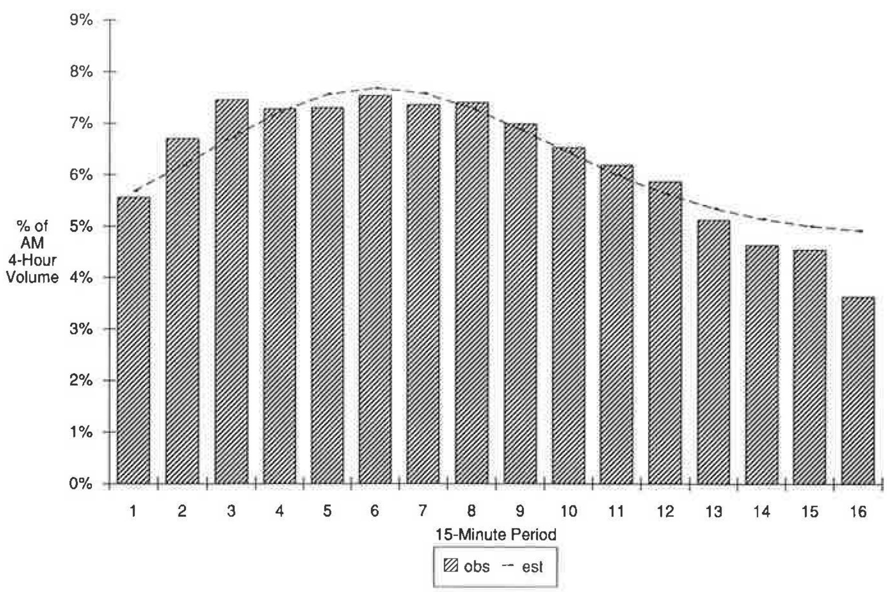

FIGURE 4 Observed versus estimated distribution: worst fit.

adequacy requires an assessment of its sensitivity to future changes in the independent variables. Thus, an analysis was performed to determine how potential changes in these variables would affect future estimates of the temporal distribution.

Because the link from Exit 34 to Exit 35 is an important one, it was selected to demonstrate how changes in the highway system would affect future traffic peaking patterns. The model was tested by varying the values of its key variables: link speed difference, average downstream 5-mi delay, and total 4-hr volume. Tests were done using 10 and 25 percent decreases in each of these values, and then 10 and 25 percent increases in each of them. Although the three variables are interrelated, these tests were performed by changing only one variable at a time so that the net effect of each could be more clearly seen.

Figures 6–8 show the results of this analysis. The model is clearly most sensitive to the difference between average congested and free speed on each link (Figure 6). From this curve it is apparent that most of the drivers on this link want to be there at 6:15–6:45. But increasing the speed difference from its base value of 21.3 mph strongly flattens the curve and shifts it slightly to the right; decreasing this value leads to a higher peak that occurs earlier. This finding suggests that drivers would use the link later in an attempt to avoid the increased congestion. If congestion were to abate, drivers might travel closer to their desired time (i.e., earlier), which is consistent with the basic theory of the model.

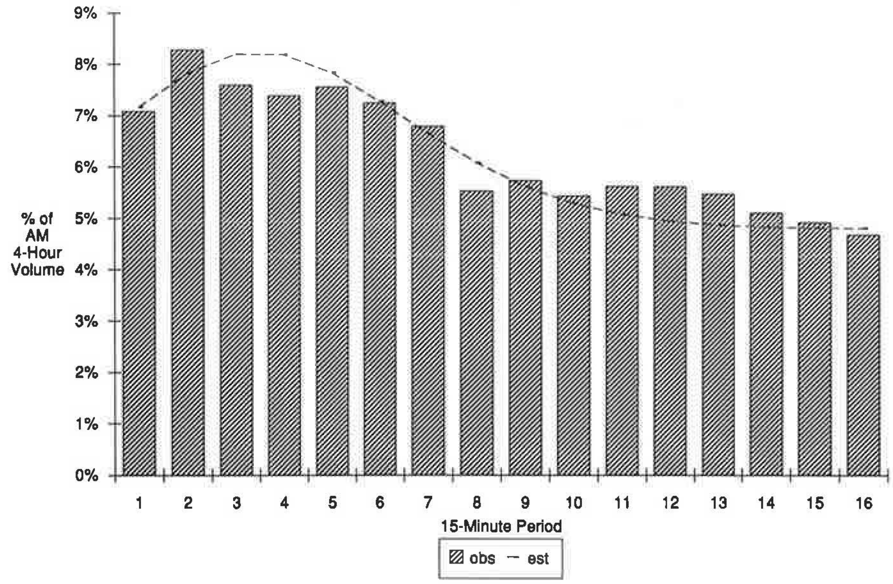

FIGURE 5 Observed versus estimated distribution: average fit.

TABLE 3 OBSERVED AND ESTIMATED PEAK-HOUR VOLUMES

Segment (Entry - Exit Milepost)	Observed	Estimated	Percent Difference
0 - 4	1,405	1,454	4%
4 - 12	1,303	1,332	2%
12 - 19	1,616	1,694	5%
19 - 25	2,019	2,119	5%
25 - 26	2,595	2,689	4%
26 - 27	3,497	3,570	2%
27 - 28	3,301	3,435	4%
28 - 30	3,258	3,277	1%
30 - 34	3,250	3,381	4%
34 - 35	4,299	4,443	3%
35 - 37	4,678	4,507	-4%
37 - 38	5,652	5,740	2%
38 - 39	5,091	4,939	-3%

TABLE 4 OBSERVED AND ESTIMATED START OF A.M. PEAK 60-min PERIOD

Segment (Entry - Exit Milepost)	Observed	Estimated	Difference
0 - 4	6:00	6:15	:15
4 - 12	6:00	6:15	:15
12 - 19	6:00	6:15	:15
19 - 25	6:15	6:15	0
25 - 26	6:15	6:30	:15
26 - 27	6:15	6:30	:15
27 - 28	6:15	6:30	:15
28 - 30	6:15	6:15	0
30 - 34	6:15	6:15	0
34 - 35	6:15	6:15	0
35 - 37	6:15	6:30	:15
37 - 38	6:30	7:00	:30
38 - 39	6:30	6:45	:15

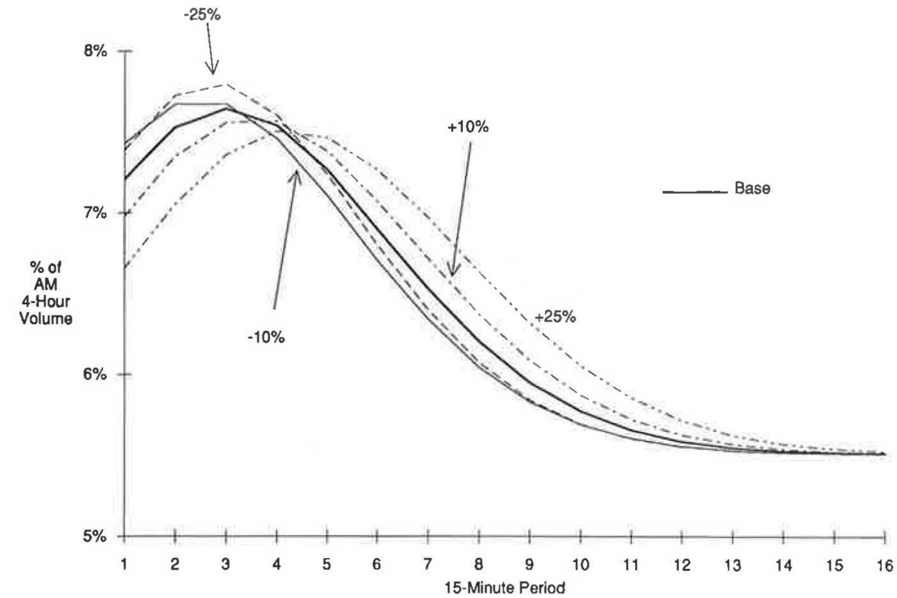

FIGURE 6 Sensitivity to change in speed difference, Exit 34 to 35.

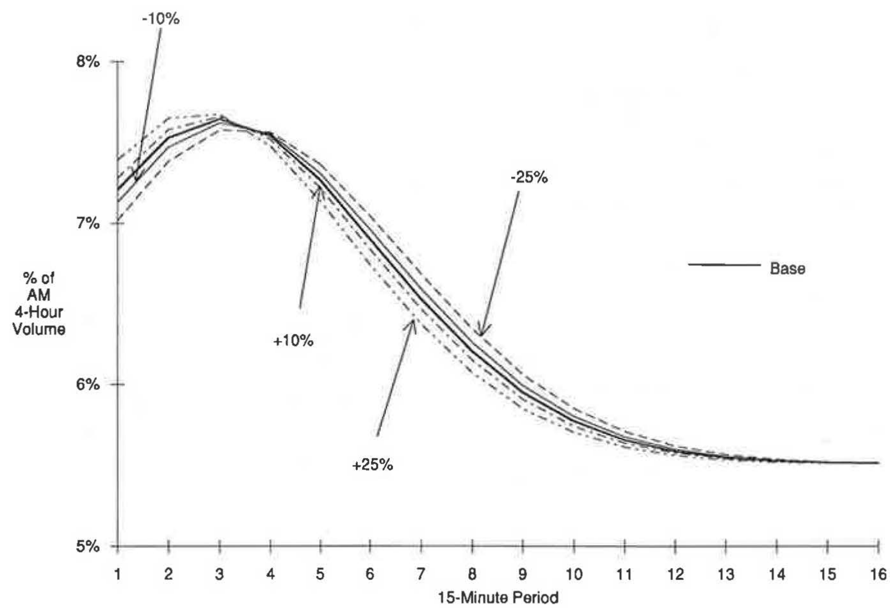

FIGURE 7 Sensitivity to change in average downstream 5-mi delay, Exit 34 to 35.

As Figure 6 shows, the model is indeed sensitive to changes in speed difference, although this sensitivity appears to be within reasonable bounds.

In examining Figure 7, however, the opposite effect is demonstrated, although to a much smaller degree. As downstream delay increases, the curve becomes sharper and peaks earlier. As noted previously, this effect serves to moderate the influence of changes in speed difference and helps ensure that the model does not give results that are too extreme in any direction.

Figure 8 shows the effect of changes in the total 4-hr volume. Because two of the model's coefficients are based on the absolute difference between this volume and 15,000, it can be seen that a 25 percent increase produces a similar effect as a 25 percent decrease. No apparent physical interpretation can readily be attached to this phenomenon; it is possibly a statistical aberration of this particular calibration data set. However, it is not a cause of great concern: as noted earlier, the total 4-hr volume is related to the other two variables, whose effect would moderate any unusual influence of this variable.

CONCLUSIONS

The question of future spread of peak-hour traffic volumes affects most current traffic forecasting efforts. Research into this topic asserts that it will become an increasingly important

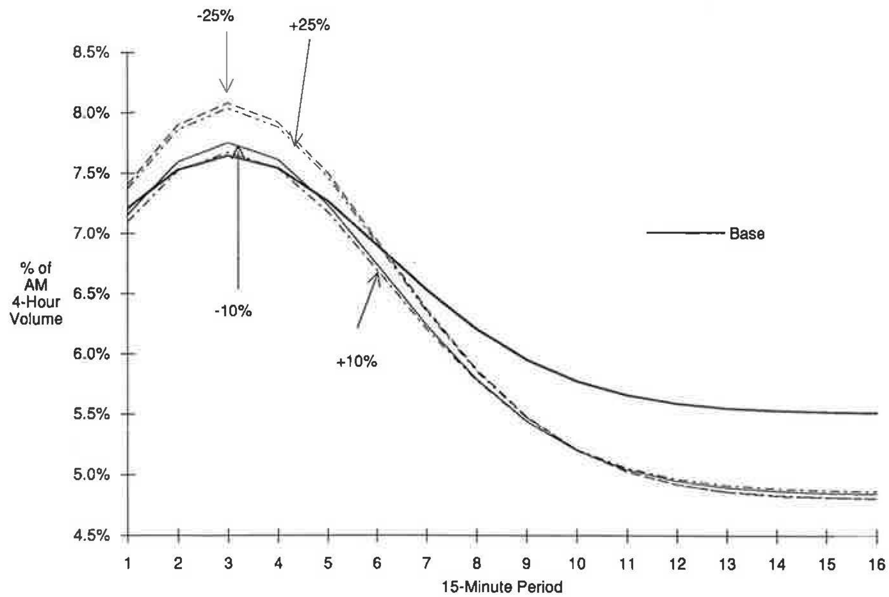

FIGURE 8 Sensitivity to change in total 4-hr volume, Exit 34 to 35.

issue that must be addressed by transportation planners. The research effort described is a somewhat awkward attempt to quantify and forecast peaking; as such, it should be viewed as merely an early step in what should become an expanding area of transportation planning research.

Although the I-80 corridor is, in many respects, not unlike other Interstate corridors around the country, the results described may be difficult to generalize for use elsewhere. It is explicitly recognized that the survey data and the planning framework of the I-80 study did not readily lend themselves to the kind of analysis that the investigators might have liked to pursue. The resulting model is thus perhaps not as theoretically sound or usable on other projects as hoped. Notwithstanding the model's shortcomings, it is important to expose this issue to a wide audience of planners and researchers in the hope of stimulating others to develop more rigorous procedures with which to forecast the traffic peaking phenomenon.

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