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Analysis of Freeway Traffic Time-Series Data by Using Box-Jenkins Techniques

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This paper investigates the application of analysis techniques developed by Box and Jenkins to freeway traffic volume and occupancy time series. A total of 166 data sets from three surveillance systems in Los Angeles, Minneapolis, and Detroit were used in the development of a predictor model to provide short-term forecasts of traffic data. All of the data sets were best represented by an autoregressive integrated moving-average (ARIMA) (0,1,3) model. The moving-average parameters of the model, however, vary from location to location and over time. The ARIMA models were found to be more accurate in representing freeway time-series data, in terms of mean absolute error and mean square error, than moving-average, double-exponential smoothing, and Trigg and Leach adaptive models. Suggestions and implications for the operational use of the ARIMA model in making forecasts one time interval in advance are made.

In computer-supervised traffic-surveillance systems the control decisions are often based on forecasts of traffic-stream time-series data gathered in real time. One of the many applications of traffic time series in traffic surveillance and control is to urban freeways for determining control strategies for ramp metering, incident detection, and variable message advisory or warning signs. Most vehicle delay on arterial streets, for example, occurs at traffic signals. The sophisticated intersection control strategies that have been developed to alleviate such delay are based on traffic time-series data. These data can also be used to determine changes in traffic demand patterns, onset of peak-period conditions, and occurrence of traffic congestion during special events such as concerts and athletic events.

Computer control strategies usually require forecasts of the traffic variables long before implementation. These forecasts are based on past observations of the variable time series. In freeway surveillance and control systems, a forecast for the next minute is usually needed because changes in traffic flow can occur suddenly. Also, when this forecast is compared with the next observation of the traffic variable, it can signal a possible change in the traffic-stream behavior and can suggest a suitable control response.

The behavior of traffic time series has been the subject of much theoretical and experimental research work in recent years. Two analysis techniques have been commonly used: spectral analysis and discrete time-series analysis. Spectral analysis of time series as discussed by Jenkins and Watts (1) has been applied by Nicholson and Swann (2) to make short-term forecasts of traffic flow volumes in tunnels. Lam and Rothery (3) used the same technique to study the propagation of speed fluctuations on freeways. Also, Darroch and Rothery (4) used cross-spectral analysis of car-following data to explain the dynamic characteristics of a freeway traffic stream. Discrete time-series analysis has been used by Hillegas, Houghton, and Athol (5), who proposed a Markovian first-order autoregressive model when traffic occupancy exceeded 15 percent, and by Breiman and Lawrence (6), who explored short- and long-term fluctuations in traffic flow.

PURPOSE

The purpose of this paper is to investigate the application of the techniques developed by Box and Jenkins (7) to freeway traffic time series. Polhemus (8) previously applied them to a description of local fluctuations in air-traffic operations; Der (9) applied them to Chicago freeway occupancy data; and Eldor (10) applied them to Los Angeles freeway and ramp traffic data, although Eldor's data consisted of 5-min aggregations of volume time-series data.

In this paper, Box-Jenkins techniques are used to develop a forecasting model based on traffic volumes and occupancies by using data from three freeway surveillance systems in Los Angeles, Minneapolis, and Detroit. A total of 166 time series representing more than 27 000 min of observation were used in the development and evaluation of the model. Table 1 summarizes the data sources and types. The data from Los Angeles and Minneapolis are described by Payne and Heifenbein (11), while the data from Detroit are described by Cook and Cleveland (12). The Los Angeles data are 20-s volumes and occupancies per lane, and the data from Minneapolis and Detroit are volumes and occupancies aggregated over all lanes at 30- and 60-s intervals, respectively. Figure 1 shows representative plots of volume and occupancy time series at detector station 7 of I-35 in Minneapolis.

The performance of the model is tested and evaluated in comparison with three other ad hoc smoothing models: the moving-average model, the double-exponential smoothing model, and the Trigg and Leach adaptive model. Performance evaluations are based on the forecasting errors caused by each model.

BOX-JENKINS APPROACH TO TIME-SERIES ANALYSIS

The Box-Jenkins approach (7) is used here to construct a predictor model for freeway traffic-stream variables. Let X_t represent a nonseasonal time series of observations taken at equally spaced time intervals. X_t is either stationary or reducible to a stationary form Z_t by computing the difference for some integer number of times d such that

$$Z_t = (1 - B)^d X_t \tag{1}$$

where B is backshift operator defined as $BX_t = X_{t-1}$.

Mathematically, a stationary time series is one for which the probability distribution of any (K + 1) observations $(Z_t, \ldots, Z_{t-K})$ is invariant with respect to t. Any set of observations from a stationary series will have the same mean value, μ.

Many real-time series can be represented by the following general class of linear models:

$$\Phi_p(B)(1 - B)^d (X_t - \mu) = \Theta_q(B) a_t \tag{2}$$

where

p, d, q = nonnegative integers,
μ = mean of the series,
$\Phi_p(B)$ = autoregressive operator of order P or

Table 1. Data sources and types.

Freeway Location	Detection Hardware	Data Description: Type	Aggregation Interval (s)	No. of Intervals per Set	No. of Data Sets
Los Angeles	Induction loops	Volume[a]	60	175	10
		Volume[b]	20	175	30
		Volume[b]	60	525	30
		Occupancy[a]	60	175	10
		Occupancy[b]	20	175	30
		Occupancy[b]	60	525	30
Minneapolis	Induction loops	Volume[a]	30	150	10
		Occupancy[a]	30	150	10
Detroit	Ultrasonic	Volume[a]	60	260	2
		Occupancy[a]	60	121	2
				260	2

[a] Aggregated over lanes. [b] Per lane.

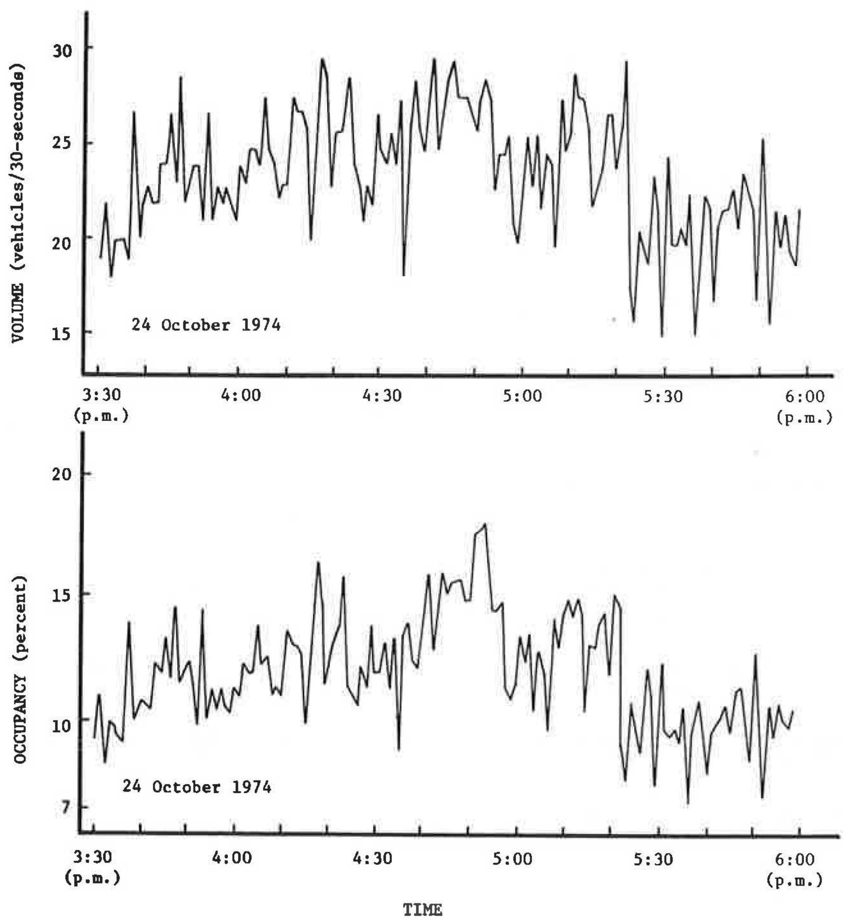


Figure 1. Freeway traffic volume and occupancy series, Minneapolis, I-35, station 7.

$1 - \Phi_1 B \ldots - \Phi_p B^p$, $|\phi| < 1$,

$\Theta_q(B)$ = moving-average operator of order q or $1 - \Theta_1 B - \Theta_2 B^2 \ldots - \Theta_q B^q$, $|\theta| < 1$, and

a_t = random disturbances (known as white noise) assumed to be independently distributed as $N(0, \sigma_a^2)$.

The models in Equation 2 are autoregressive integrated moving-average (ARIMA) models of order (p, d, q).

ARIMA models are fitted to a particular data set by a three-stage iterative procedure: preliminary identification, estimation, and diagnostic check. In preliminary identification, the values of p, d, and q are determined by inspecting the autocorrelations and partial autocorrelations of the series or its differences, or both, and by comparing them with those of some basic stochastic processes. The sample autocorrelation function is given by

$$r_K = \sum_{t=1}^{n-K} [(X_t - \bar{X})(X_{t+K} - \bar{X})] \Big/ \sum_{t=1}^{n} (X_t - \bar{X})^2, \; K = 1, 2, \ldots \quad (3)$$

where $\bar{X}$ is the sample mean and n is the number of observations. The autocorrelation function of a stochastic process provides a measure of how long a disturbance in the system affects the state of the system in the future.

In general, the autocorrelation function of a moving-average process of order q has a cutoff after lag q (memory of lag q), while its partial autocorrelation function tails off. Conversely, the autocorrelation function of an autoregressive process of order p tails off in

Figure 2. Sample autocorrelations and partial autocorrelations, volume data, Minneapolis, I-35, station 7.

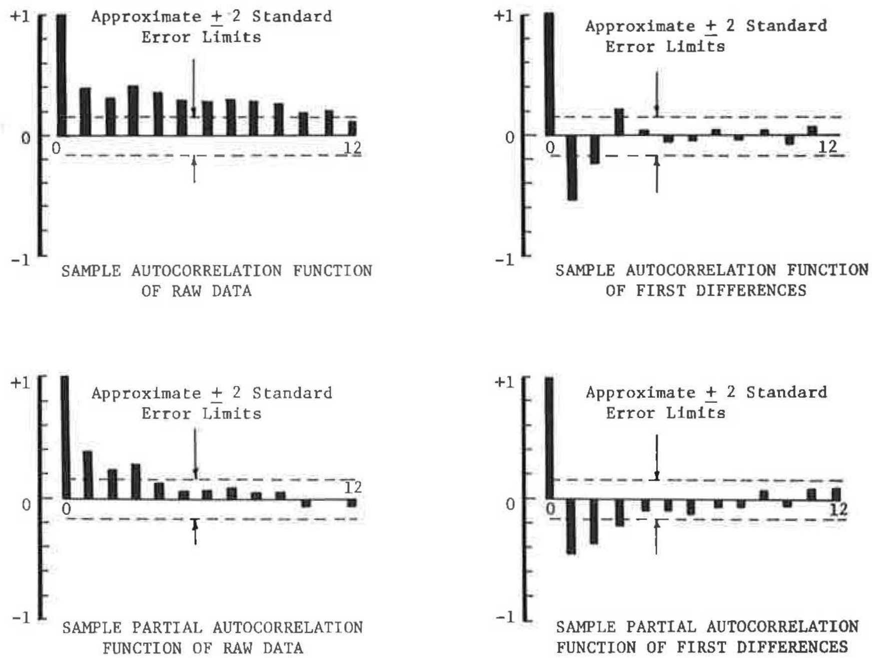
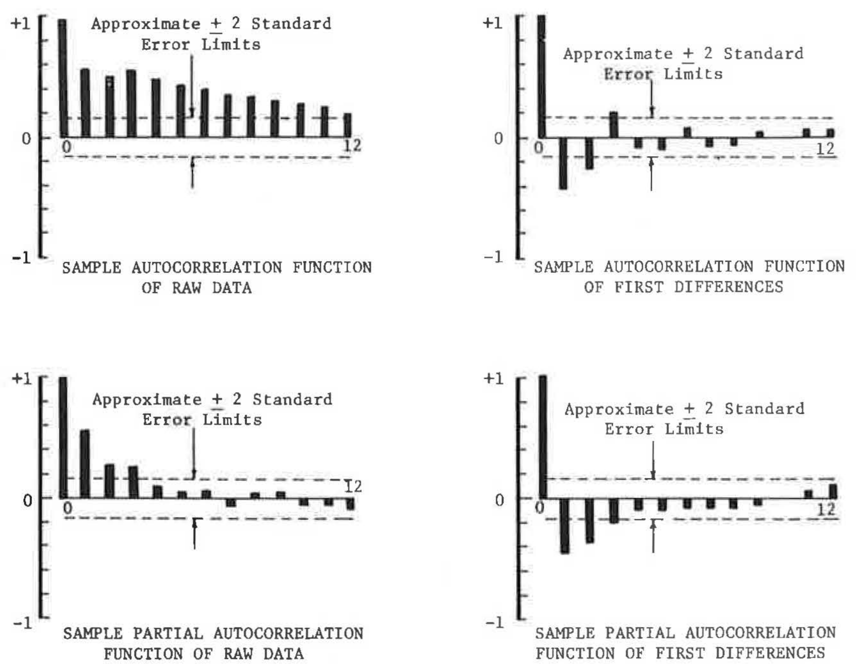


Figure 3. Sample autocorrelations and partial autocorrelations, occupancy data, Minneapolis, I-35, station 7.



the form of damped exponentials or damped sine waves, while its partial autocorrelation function has a cutoff after lag p. For mixed processes, both the autocorrelations and partial autocorrelations tail off. Failure of the autocorrelation function to die out rapidly suggests that differencing is needed ($d > 0$).

Once the values of p, d, and q have been determined, the autoregressive and moving-average parameters are estimated by using nonlinear least-squares techniques. Finally, the goodness of the model fit is checked. If the form of the chosen model is satisfactory, then the resulting residuals, $\hat{a}_t$, should be uncorrelated random deviations. To test for this, Box and Pierce (13) developed an overall test of residual autocorrelations for lags 1 through K. They found that the variable

$$Q = n \sum_{i=1}^{K} r_i^2(\hat{a}) \tag{4}$$

where n is the number of observations minus the degree of differencing and $r_i(\hat{a})$ is residual autocorrelation for lag i. Q is approximately distributed as a chi-square variable with (K-p-q) degrees of freedom.

MODELING FREEWAY TRAFFIC TIME-SERIES DATA

Three computer programs entitled PDQ, ESTIMATE, and FORECAST (14) were used in this research to perform the computations required by the Box-Jenkins technique. Application to all of the time series listed in

Table 1 resulted in the same ARIMA model, albeit with different coefficients.

Figures 2 and 3 show the sample autocorrelations and partial autocorrelations for the representative volume and occupancy series given in Figure 1 and for their first differences. The sample autocorrelations of the raw data damp off very slowly as lag increases. This suggests that differencing is needed.

The sample autocorrelations of the first differences, however, indicate that only the spikes at lags 1, 2, and 3 are large in relation to their standard error. The partial autocorrelations of the first differences gradually tail off. This confirms that the stochastic process generating the data is ARIMA (0, 1, 3); that is, the first differences of traffic data can be represented by a third-order moving-average model:

$$(1 - B)(X_t - \mu) = (1 - \theta_1 B - \theta_2 B^2 - \theta_3 B^3)a_t \quad , |\theta| < 1 \tag{5}$$

or simply

$$\begin{aligned} X_t - X_{t-1} &= Z_t \\ &= a_t - \theta_1 a_{t-1} - \theta_2 a_{t-2} - \theta_3 a_{t-3} \end{aligned} \tag{6}$$

The model in Equation 6 states that the series of differences $Z_1, Z_2, \ldots, Z_t, \ldots$ is a series of moving linear combinations of (a_0, a_1, a_2, a_3), (a_1, a_2, a_3, a_4), ..., and $(a_{t-3}, a_{t-2}, a_{t-1}, a_t)$, ..., with weight functions $(-\theta_3, -\theta_2, -\theta_1, 1)$. It is perhaps more meaningful, however, to view the model as showing that shock a_t coming into the system at time t will persist over (3 + 1) periods (t, t + 1, t + 2, t + 3) in proportion to $(1, -\theta_1, -\theta_2, -\theta_3)$ before dissipation. The vector $(1, -\theta_1, -\theta_2, -\theta_3)$, which is the mirror image of the weight function $(-\theta_3, -\theta_2, -\theta_1, 1)$, is called the shock-effect function. The coefficients of the volume and occupancy series shown in Figure 1 are

Data	Coefficient	Standard Error
Volume		
θ_1	0.7823	0.0825
θ_2	0.0557	0.0105
θ_3	0.0844	0.0082
Occupancy		
θ_1	0.6852	0.0825
θ_2	0.0627	0.0099
θ_3	0.0741	0.0082

Diagnostic checking was carried out by inspecting the residuals ($\hat{a}_t$). The autocorrelation functions of the residuals and the residual plots for volume and occupancy data are shown in Figures 4 and 5, where the autocorrelations exhibit no systematic pattern and are all quite small. For the volume series, the average of the residuals ($\bar{a}$) is 0.0221, and the estimated standard error of $\bar{a}$ is 0.2362. This strongly suggests that the a_t have zero mean. Similarly, the average of the residuals for the occupancy series is 0.0196 and has an estimated standard error of 0.1402, which supports the same conclusion.

The values of Q for K = 24 lags (a value set in the Box-Jenkins programs in this study) are 27.6 and 21.8 for the volume and occupancy series, respectively. When these values of Q are compared with tabulated chi-square values with 21 degrees of freedom, they indicate that the residuals are white noise at the 0.05 level of significance.

Der (9), in his analysis of two occupancy series from the Dan Ryan Expressway in Chicago, suggested an ARIMA (1, 0, 1) process to describe traffic occupancies. However, he reported that a higher-order ARIMA process such as (0, 1, 3) may be a possible candidate process. The problem with an ARIMA (1, 0, 1) process is that it assumes that the raw traffic time series is stationary, which is not always true. Eldor (10) evaluated the ARIMA series (0, 1, 1), (0, 1, 0), and (0, 2, 1).

Some freeway surveillance systems have detectors in all lanes, while other systems have detectors only in some lanes. Also, surveillance data are generally aggregated over different time intervals, usually 20, 30, or 60 s before processing. The transferability of the ARIMA (0, 1, 3) model under these conditions was studied by applying the model to different time series from the three different freeway systems in Table 1.

Tables 2 and 3 show the range of values of the moving-average parameters for 46 series of volume and occupancy aggregated over lanes at a detector station. Although there are some differences in the parameter estimates between or within the different freeway systems, it is emphasized that it is the form of the ARIMA model that is transferable. The differences in parameter estimates arise from variations in flow characteristics and, probably, variations in geometrics and similar factors. Eldor (10) also noted that no universal parameters could be identified with his data aggregated to 5-min intervals.

In addition, the data from Los Angeles, which consist of 20-s compilations of volume and occupancy per lane, provided an opportunity to compare individual lane data with data aggregated across all lanes at a detector station. The ARIMA (0, 1, 3) model was applied to 60 series of 20-s lane volumes and occupancies. The model process was found representative in all these cases. Tables 4 and 5 give the range of values of the moving-average parameters for lane volumes and occupancies. The effect of sampling interval was also investigated by aggregating the 20-s observations to 60-s observations, which also confirmed the model. Therefore, it is concluded that the model can be successfully used in a variety of freeway surveillance configurations to provide short-term forecasts of traffic volumes and occupancies.

COMPARATIVE EVALUATION OF FORECASTING PERFORMANCE

This section presents a comparative evaluation of the forecasting performance of the model in Equation 6 against three ad hoc smoothing models: the moving-average model, the double-exponential smoothing model, and the Trigg and Leach adaptive model. To facilitate the discussion, these smoothing models are briefly reviewed.

Moving-Average Model

The moving average at time t defined over the N previous observations is given by

$$m(t,N) = (1/N) \sum_{K=1}^{N} X_{t-K} \tag{7}$$

This model weights each of the previous N observations by 1/N, while other earlier observations have zero weight. The forecast of X_t is

$$\hat{X}_t = m(t,N) \tag{8}$$

Five values of N (N = 5, 10, 20, 50, and 100) were used in the evaluation of the moving-average model in this study.

Exponential Smoothing Model

It is assumed that the observation X_t can be described by a model of the form

$$X_t = F_t + \epsilon_t \tag{9}$$

where F_t is a deterministic function of time and ϵ_t is a stochastic component. Single exponential smoothing as proposed by Brown (15) assumes that F_t represents some equilibrium level; the corresponding smoothing function is given by

$$S_1(t) = \alpha x_t + (1 - \alpha) \times S_1(t - 1) \tag{10}$$

Figure 4. Residual plots and sample autocorrelations, volume data, Minneapolis, I-35, station 7.

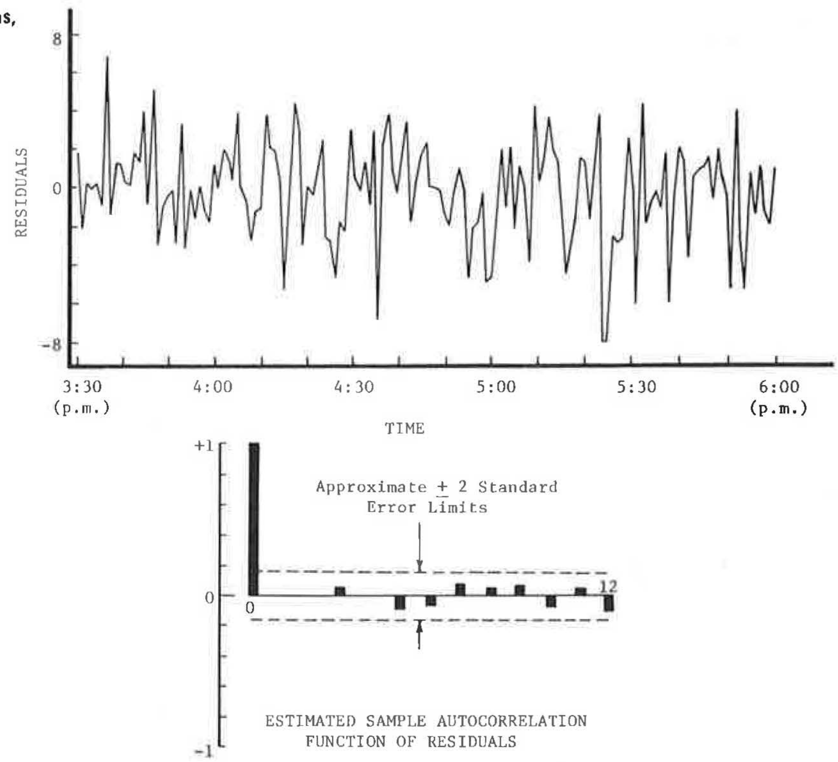


Figure 5. Residual plots and sample autocorrelations, occupancy data, Minneapolis, I-35, station 7.

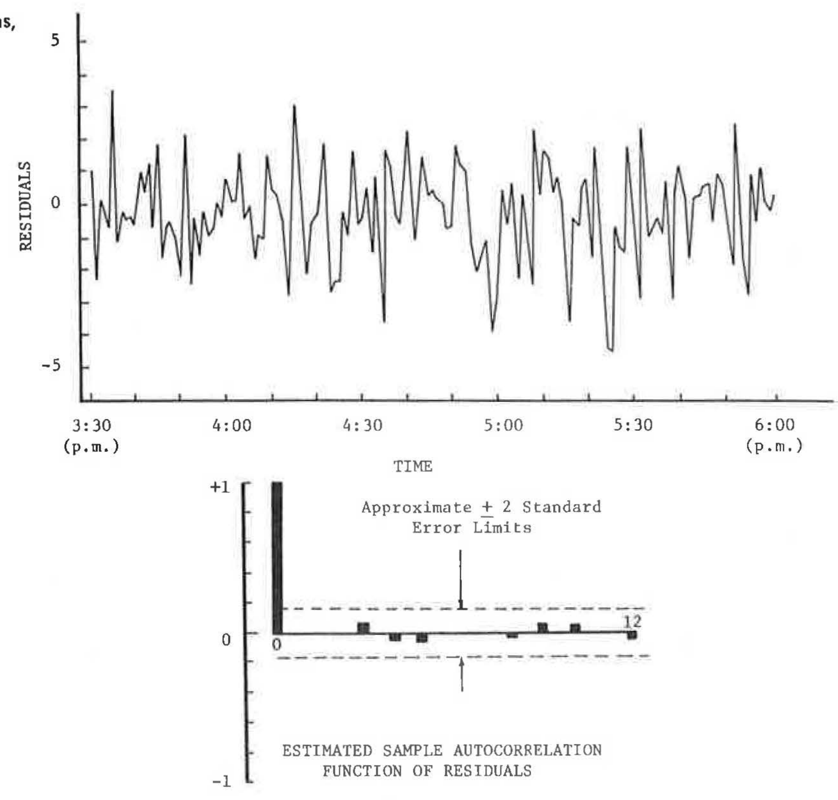


Table 2. Moving-average parameters for volume series aggregated over lanes.

Freeway Location	No. of Data Sets	No. of Observations	Moving-Average Parameters Θ_1	Θ_2	Θ_3
Los Angeles	10	1750	0.7301 ± 0.1885	0.1777 ± 0.5765	0.0391 ± 0.1398
Minneapolis	10	1500	0.7553 ± 0.1375	0.1519 ± 0.5249	-0.1530 ± 0.1438
Detroit	2	381	0.7420 ± 0.0732	0.0403 ± 0.0671	0.0012 ± 0.0143

Table 3. Moving-average parameters for occupancy series aggregated over lanes.

Freeway Location	No. of Data Sets	No. of Observations	Moving-Average Parameters Θ_1	Θ_2	Θ_3
Los Angeles	10	1750	0.5611 ± 0.3541	0.1145 ± 0.2711	0.2596 ± 0.3507
Minneapolis	10	1500	0.4710 ± 0.2160	0.1307 ± 0.2515	-0.0848 ± 0.2249
Detroit	2	762	0.6121 ± 0.1649	0.0859 ± 0.1398	0.0704 ± 0.1678

Table 4. Moving-average parameters for 20- and 60-s lane volumes, Los Angeles.

	Moving-Average Parameters					
	20-s Series (15 750 observations)			60-s Series (5250 observations)		
Lane No.[a]	Θ_1	Θ_2	Θ_3	Θ_1	Θ_2	Θ_3
1	0.8081 ± 0.1263	0.0752 ± 0.1145	0.0426 ± 0.0742	0.8280 ± 0.1720	0.0311 ± 0.1786	0.0085 ± 0.1129
2	0.8701 ± 0.1245	0.0404 ± 0.1133	0.0401 ± 0.0738	0.7860 ± 0.1127	0.0056 ± 0.1788	0.0229 ± 0.1222
3	0.8131 ± 0.1582	0.0296 ± 0.1253	0.0074 ± 0.0933	0.8180 ± 0.1414	-0.0389 ± 0.3364	0.0412 ± 0.1466
4	0.8811 ± 0.1130	0.0569 ± 0.0573	0.0278 ± 0.0622	0.4526 ± 0.3510	0.1250 ± 0.2358	0.0437 ± 0.0904

[a]Numbering begins with the lane closest to the median and increases toward the right shoulder.

Table 5. Moving-average parameters for 20- and 60-s lane occupancies, Los Angeles.

	Moving-Average Parameters					
	20-s Series (15 750 observations)			60-s Series (5250 observations)		
Lane No.[a]	Θ_1	Θ_2	Θ_3	Θ_1	Θ_2	Θ_3
1	0.6196 ± 0.2786	0.1971 ± 0.1988	0.0814 ± 0.1261	0.7057 ± 0.2856	0.1666 ± 0.2337	-0.0581 ± 0.0565
2	0.7096 ± 0.2353	0.1037 ± 0.1234	0.0674 ± 0.0776	0.6284 ± 0.2547	0.1330 ± 0.2681	0.0134 ± 0.1046
3	0.6672 ± 0.2394	0.1658 ± 0.1700	0.0211 ± 0.1116	0.6888 ± 0.3111	-0.0261 ± 0.2991	0.0388 ± 0.1448
4	0.6539 ± 0.1802	0.0400 ± 0.0857	0.1094 ± 0.1203	0.5617 ± 0.3655	0.1855 ± 0.1277	0.0431 ± 0.1687

[a]Numbering begins with the lane closest to the median and increases toward the right shoulder.

where $S_1(t)$ is the smoothed value of X at time t and α is a smoothing constant, $0 < \alpha < 1$. The function $S_1(t)$ is a linear combination of all previous observations weighted by damped exponential weights. The forecast of X_t is

$$\hat{X}_t = S_1(t) \tag{11}$$

Note that single-exponential smoothing is equivalent to an ARIMA (0, 1, 1) process where the smoothing constant α is set equal to θ_1. The double-exponential smoothing model assumes that F_t can be described by a linear trend. The corresponding smoothing function is

$$S_2(t) = \alpha[S_1(t)] + (1 - \alpha) \times S_2(t - 1) \tag{12}$$

Brown demonstrated that the steady-state response of exponential smoothing to a linear trend has a constant lag of $(1 - \alpha)/\alpha$. Therefore, the forecast of the next observation X_{t+1} is

$$\hat{X}_{t+1} = \phi(t) + \psi(t) \tag{13}$$

where $\phi(t)$ is $2[S_1(t)] - S_2(t)$ and $\psi(t)$ is $(\alpha/1 - \alpha)$ $[S_1(t) - S_2(t)]$. Values of α used in the evaluation of the double-exponential smoothing model were 0.1-0.9 in increments of 0.1.

Exponential Smoothing with Adaptive Response

Adaptive approaches for adjusting the smoothing constant α have been suggested by many authors, including Chow (16), Roberts and Reed (17), and Trigg and Leach (18). Most of these approaches use the forecasting performance of the smoothing model to determine the proper adjustment of the smoothing constant. The following is the adaptive approach proposed by Trigg and Leach:

$$TS(t) = SE(t)/SAE(t), -1 < TS < 1 \tag{14}$$

$$SE(t) = \gamma \times e_t + (1 - \gamma) \times SE(t - 1) \tag{15}$$

$$SAE(t) = \gamma \times |e_t| + (1 - \gamma) \times SAE(t - 1) \tag{16}$$

$$e_t = X_t - \hat{X}_t \tag{17}$$

where

$TS(t)$ = tracking signal at time t,
$SE(t)$ = smoothed error at time t,
$SAE(t)$ = smoothed absolute error at time t,
e_t = forecast error at time t, and
γ = smoothing constant, $0 < \gamma < 1$.

Adaptive response of the smoothing constant α is achieved by setting it to equal the absolute value of the tracking signal. The Trigg and Leach model was tested by using nine values of α between 0.1 and 0.9 and three values of γ, 0.1, 0.2, and 0.3.

In evaluating the four forecasting models, the following mean absolute error (MAE) and mean square error

Figure 6. Ratio to Box-Jenkins for mean absolute error.

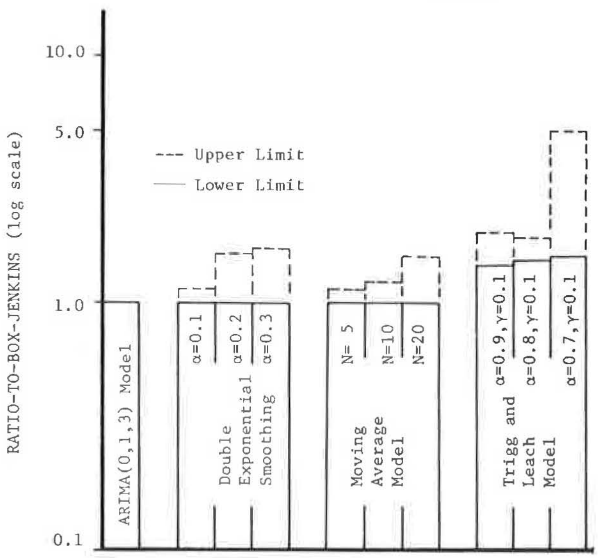
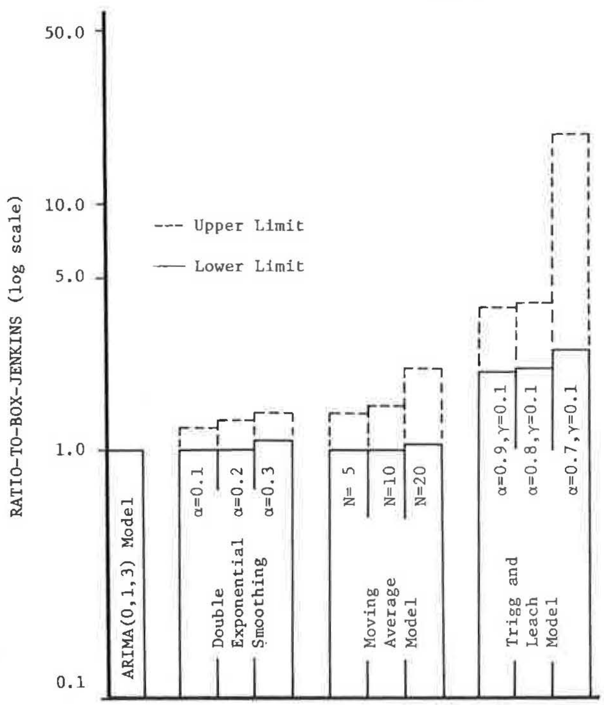


Figure 7. Ratio to Box-Jenkins for mean square error.



(MSE) functions were used as evaluation criteria:

$$MAE = (1/N) \sum_{t=1}^{N} |X_t - \hat{X}_t| \tag{18}$$

$$MSE = (1/N) \sum_{t=1}^{N} (X_t - \hat{X}_t)^2 \tag{19}$$

where

X_t = observed variable value at time t,
$\hat{X}_t$ = predicted value of variable at time t, and
N = the number of observations.

MAE indicates the error expected to be associated with each forecast, while MSE detects the presence of frequent large forecasting errors.

For the purpose of comparing the smoothing performance of the different models, values of MAE and MSE of the fitted ARIMA (0, 1, 3) models were chosen as a basis. These values ranged from 1.30 to 6.50 for MAE, and from 2.80 to 91.41 for MSE. Results of the moving-average model indicated that both MAE and MSE increase with increases in N. When N equaled five, the ratio to Box-Jenkins varied between 1.00 and 1.27 for MAE and between 1.00 and 1.45 for MSE. Larger values of N (10-100) resulted in values of ratio to Box-Jenkins of between 1.00 and 2.85 for MAE and between 1.00 and 6.86 for MSE.

The best results of the double-exponential smoothing model were associated with small values of α. For smoothing constants between 0.1 and 0.3 the ratio to Box-Jenkins ranged from 1.00 to 1.64 for MAE and from 1.00 to 1.43 for MSE.

The Trigg and Leach model did not improve the forecasts. With large initial values of the smoothing constant α between 0.6 and 0.9 and a smoothing constant (γ) of 0.1, which gave the best results for this model, the ratio to Box-Jenkins varied between 1.45 and 8.20 for MAE and between 2.08 and 44.34 for MSE. The reason for the poor performance of the Trigg and Leach model could be the abrupt successive changes in α. Figures 6 and 7 illustrate the ranges of the best values of the ratio to Box-Jenkins for MAE and MSE for the different models. The ARIMA (0, 1, 3) model is seen to be superior: It more accurately represents the stochastic process generating the traffic data.

MODEL APPLICATIONS TO SHORT-TERM FORECASTS

To appreciate the operational value of the ARIMA (0, 1, 3) model, one should examine how it can be used in making short-term forecasts in real time.

Let $\hat{Z}_{t-1}(1)$ be the one-step-ahead forecast made at time (t - 1) for Z_t, which when observed will be represented by Equation 6. If $\hat{Z}_{t-1}(1)$ is the minimum mean-square-error forecast, then its value will be determined by the conditional expectation of Z_t, given the history (H_t) of the series up to time t; that is,

$$\begin{aligned}\hat{Z}_{t-1}(1) &= E(Z_t/H_t) \\ &= -\theta_1 a_{t-1} - \theta_2 a_{t-2} - \theta_3 a_{t-3}\end{aligned} \tag{20}$$

Therefore, the forecast error at time (t - 1) is determined by subtracting Equation 20 from Equation 6:

$$\begin{aligned}e_{t-1}(1) &= Z_t - \hat{Z}_{t-1}(1) \\ &= a_t\end{aligned} \tag{21}$$

Hence, the white noise that generates the process is the one-step-ahead forecast error. In a similar fashion

$$a_{t-1} = e_{t-2}(1) \tag{22}$$

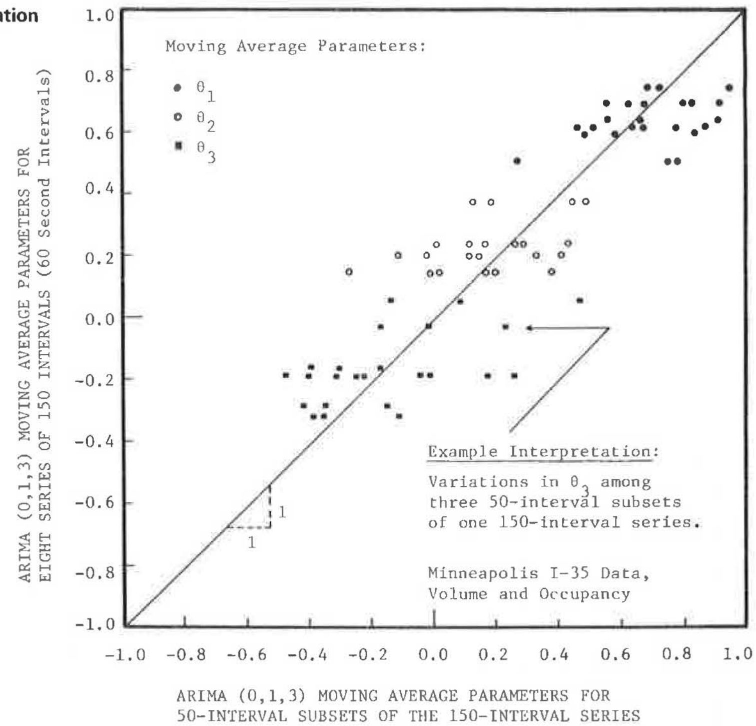
and

$$a_{t-2} = e_{t-3}(1) \tag{23}$$

Consequently, an operational expression for updating the forecasts of the model in Equation 6 is

$$\hat{Z}_t(1) = -\theta_1 e_{t-1}(1) - \theta_2 e_{t-2}(1) - \theta_3 e_{t-3}(1) \tag{24}$$

Figure 8. Variability of ARIMA (0,1,3) parameters by location and over time.



The computational utility of the above expression stems from the fact that its application requires computer storage only of the latest three forecast errors and the current observation.

The sensitivity of the performance of the ARIMA (0, 1, 3) model to variations in the θ parameters over time was tested to a limited extent as follows. A number of volume and occupancy time series, each 150 60-s time intervals from the Minneapolis I-35 data were broken into three 50-interval segments. The ARIMA (0, 1, 3) model was applied separately to each segment.

The variations in the estimated moving-average parameters (θ_1, θ_2, and θ_3) for both sets of series are depicted in Figure 8. The horizontal scatter of points indicates that the parameters do vary over time but no consistent pattern in this variation was noted. However, due to the limited number of observations used in estimating the parameters for each 50-interval segment, the conclusion that these parameters vary with time cannot be accurately drawn. It is also important to note that the same form of the ARIMA (0, 1, 3) model that represented the 150-observation series represented the 50-observation segments just as well.

It may be desirable, although not necessarily warranted, to update the moving-average parameters in real time. It is believed that a rapid adjustment in the parameter estimates—each observation interval, for example—may degrade the overall forecasting performance of the ARIMA model. Past experience with adaptive-exponential-smoothing models, particularly the Trigg and Leach model, has shown that successively changing the smoothing constant value over time yielded potentially larger forecasting errors than those resulting from Brown's original exponential-smoothing models (19). The results depicted in Figures 6 and 7 also tend to confirm this belief.

Another important factor that should be taken into consideration when one is contemplating real-time updating of the model parameters is that of computer computational requirements. One way to lower these requirements would be to update the parameters only occasionally, e.g., at the beginning of peak and off-peak periods. Parameter updating was not explored in this research, in part because the available data sets consisted of afternoon peak-period time series only. Further research along these lines is strongly recommended. Operational expressions for updating the moving-average parameters θ_1, θ_2, and θ_3 can be found in Box and Jenkins (7, pp. 162-164).

SUMMARY AND CONCLUSIONS

In this paper an application of the Box-Jenkins approach for modeling traffic time-series data has been presented. An ARIMA (0, 1, 3) model was found to represent volume and occupancy data from three different freeway systems of varying detector configurations and data-aggregation time intervals. The comparative evaluation of the ARIMA (0, 1, 3) model against some other ad hoc smoothing models has indicated the overall superiority of the ARIMA (0, 1, 3) model in providing short-term forecasts of traffic parameters.

The forecasting model described in this paper should be of use in real-time computerized freeway traffic-control systems and may be applicable to traffic-signal networks. At this writing, the model was being used to develop freeway incident-detection algorithms.

ACKNOWLEDGMENT

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Automobile Diversion: A Strategy for Reducing Traffic in Sensitive Areas

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In recent years awareness of the negative impacts of motor-vehicle travel has increased. One approach to those impacts is automobile diversion, a strategy for reducing vehicle use in congested areas. This paper reports on a recent study directed toward developing and evaluating the potential for automobile diversion in Denver. General traffic problems are identified and a potential yardstick for locating affected areas—the environmental capacity of city streets approach—is discussed. Benefits and problems of notable U.S. background experience in automobile diversion are summarized. A detailed breakdown is given of the various transportation system management strategy-formation elements applicable to automobile diversion, and several implementation techniques are described. Advantages and disadvantages are also presented to demonstrate the use of automobile diversion as a community-improvement tool. Finally, the study determines that the potential for automobile diversion in Denver relies on the degree of citizen interest, the identification and resolution of issues and problems, and sound decision making in the political forum.

In the fall of 1975, the Urban Mass Transportation Administration (UMTA) and the Federal Highway Administration (FHWA) jointly issued urban transportation planning regulations directing appropriate local agencies to develop transportation system management (TSM) plans for their respective urban areas (1). TSM plans are intended to document local strategies for improving air quality, conserving energy, and improving transportation efficiency and mobility through management of existing transportation systems. TSM strategies deal with low-capital, short-range, or policy-oriented urban transportation improvements.

Although many TSM strategies have been implemented in the Denver transportation system, only recently has emphasis been placed on directly identifying and pursuing those strategies in an organized and coordinated manner. For instance, Denver now has computerized traffic control and operations, transit operations, carpooling, and various preference and restraint programs. These management concepts and control strategies, and their respective action elements, were developed and implemented only when the need became obvious.

Because of federal emphasis on TSM and the techniques already in use in Denver, the Denver Planning

Office (DPO) has developed and evaluated the potential of selected TSM strategies to complement midterm and long-term transportation development activities such as downtown pedestrian and bicycle facilities and automobile-diversion practices.

PURPOSE

The automobile-diversion strategy has been identified as one possible means of reducing vehicle use in congested areas (2) or in areas particularly sensitive to traffic impacts on land use and social conditions. This strategy limits the movement of traffic in sensitive land-use areas by diverting traffic around rather than through them. The purpose of this report is to evaluate the potential for automobile diversion as a TSM strategy in Denver.

APPROACH AND STUDY ORGANIZATION

First, land-use and traffic-related changes in Denver and the region are summarized and the general problems and impacts of traffic in sensitive areas are identified. The environmental-capacity philosophy of city streets is examined for appropriateness of application in Denver. Automobile-diversion objectives are formulated to provide a basis for further application and evaluation of the strategy.

A brief summary of U.S. experience with automobile diversion is presented and strategy elements, including a general description of implementation techniques, are discussed. The potential for automobile diversion in Denver is then evaluated by identifying its effectiveness and related advantages and disadvantages as a TSM strategy. Finally, conclusions and recommendations are presented.

PROBLEMS AND ISSUES

Growth

Since World War II, population growth in the Denver region, coupled with increased mobility provided by the automobile and an extensive road network, has resulted in an urban pattern characterized by relatively low-density development extending outward in all directions from the city center. By 1975, the region had a population of 1 500 000 and an employment base of 650 000. During the past 35 years population and employment have more than tripled, and the amount of urbanized land has increased more than sixfold. These figures indicate predominantly low-density peripheral sprawl.

Denver's rate of growth, however, has been much slower than that of the rest of the region. From 1940 to 1975, Denver's population less than doubled and its employment less than tripled. Thus, despite its absolute growth, Denver's share of the region's total population and employment has been decreasing since 1960, which suggests that the city's role in the region is changing from that of a well-balanced residential community to that of a maturing service core for the entire region (3).

The number of motor vehicles in Denver has also increased. From 1965 to 1975 the total increase in motor vehicles registered in Denver was 94 866, while the population increased by only 23 900. This fourfold increase in vehicles over population may be accounted for by greater economic affluence, increased numbers of driving-age individuals, and the transition of the city from a predominantly residential area to a core service area.

The growth of population, employment, and motor vehicles has resulted in increased travel activity in the region and therefore more demand on the highways and transit. For example, in the same 10 years from 1965 to 1975, average motor-vehicle traffic into and out of Denver every 24 hours increased more than 80 percent, from 586 320 to 1 062 540 each day, while Denver's population increased by only 5 percent. Because Denver is the crossroads of the state's two Interstate highways, I-25 and I-70, a substantial amount of the increase can be attributed to the ballooning volume of statewide commercial and recreational travel. Interstate traffic into and out of Denver has increased by about 220 percent.

Ever-increasing vehicle traffic in urban Denver has heavily loaded most major streets and highways, probably because they usually provide the quickest, most direct routes. But many existing streets are congested because of limited capacities, restricted expansion space in older areas, and limited improvement funds. Transit and carpooling can accommodate a small proportion of all metropolitan person trips, but many drivers avoid the congested streets and highways by seeking alternate routes, for example, by using residential streets as shortcuts. Heavy through traffic and occasional speeding vehicles on otherwise quiet streets have thus become increasingly annoying and disruptive to many good residential neighborhood environments.

In addition to the obvious transportation service provided by motor vehicles, there are direct and indirect problems and impacts associated with heavy through traffic. The universally recognized, direct, negative impacts of traffic are

1. Potential street-crossing hazards for pedestrians, especially children and the elderly;
2. Air, noise, and dirt pollution;
3. Vibration; and
4. Inconvenience in parking operations and in driveway entry and exit movements.

These direct problems and impacts within a neighborhood generate real or perceived indirect problems such as social or neighborhood barriers, declining property values relative to areas with light traffic, declining pride in the neighborhood, decreased home and yard maintenance, increased renter occupancy and resident transiency, and additional resident flight to the suburbs.

Environmental Capacity of City Streets

It is usually not an individual motor vehicle that offends residential sensitivity, but rather the cumulative effect of a quantity of vehicles. Thus, consideration must be given to the number of vehicles that may affect adjacent land uses. Some efforts have been made to determine environmental capacities of streets by analysis of field surveys and questionnaires, but results have varied.

A study conducted in Louisville, Kentucky, determined that the maximum daily number of vehicles that should be permitted along street types with various land uses ranges from 14 000 on four-lane (some two-lane), single-family residential streets to 15 000 on four-lane single- and multi-family residential streets to 35 700 on four-lane, commercial, recreational, and industrial streets. A recent study in London, England, set street capacity limits at about 12 000 vehicles for 24 h (4).

In contrast, a report on San Luis Obispo, California, found average daily residential area traffic volumes as high as 4000/day acceptable, while a comprehensive study in San Francisco recommended that traffic volumes should not exceed 2000 vehicles/day on streets where the adjacent land uses include families with children (5). This wide range of acceptability values is indicative of the individuality of various communities and the relative priorities they assign to land use or transportation when these considerations conflict.

If the environmental capacity of streets is to be considered as a factor in determining traffic management in sensitive Denver residential areas, impact studies and attitude surveys of local tolerance levels of traffic volumes will be necessary.

STRATEGY OBJECTIVES

Objectives central to defining TSM strategies and to developing effective methodologies may conflict with each other. The ultimate decision might then be based on satisfying disparate points of view among users, operators, and the general public.

Planning and developing an automobile-diversion strategy needs four general categories of factors: transportation factors, social factors, economic factors, and functional and physical factors (6). Within each are specific objectives:

1. Transportation factors:
   a. Reduce street congestion,
   b. Maintain accessibility,
   c. Improve transit services,
   d. Maintain service to goods movement,
   e. Maintain emergency service,
   f. Encourage shift to nonautomobile travel modes,
   g. Reduce accidents,
   h. Reduce energy consumption,
   i. Reduce parking requirements,
   j. Prevent excessive through traffic in neighborhoods, and
   k. Achieve the functional designation of the transportation system;
2. Social factors:
   a. Increase opportunities for community interaction,
   b. Improve perception of personal security,
   c. Increase use of public areas,
   d. Create perceptible improvements in the environment, and
   e. Stimulate community cohesion;
3. Economic factors:
   a. Encourage private investment,
   b. Stimulate market potential,
   c. Enhance tax base,
   d. Reduce street construction and maintenance costs,
   e. Minimize adverse economic impacts caused by urban traffic, and
   f. Maximize effectiveness of public transit investments; and
4. Functional and physical factors:
   a. Improve air, noise, and aesthetic qualities,
   b. Enhance pedestrian space,
   c. Improve the physical environment to strengthen and support the desired types and patterns of local land use,
   d. Provide separation of motor-vehicle and nonmotor-vehicle traffic movement,
   e. Restore human scale, and
   f. Complement urban land-use goals and objectives.

Beyond these general objectives, which are applicable to most automobile-diversion strategies, other objectives related to specific proposals must be identified by planning or implementing agencies. In addition, the degree to which automobile-diversion projects can fulfill these objectives is subject to factors such as cost, space, and demand.

EXPERIENCE IN THE UNITED STATES

To date, relatively few cities or towns have implemented automobile diversion to any great extent. The techniques most used have been cul-de-sacs, diagonal intersection barriers, and narrowings that prohibit or discourage through traffic. The majority of the cities involved in significant automobile-diversion programs are on the West Coast—Seattle, Portland, San Francisco, and Berkeley (7, 8). In the Midwest, a program has been developed for the community of Oak Park, a suburb of Chicago (9). Although the experience survey is not exhaustive, notable applications are highlighted.

In most instances, automobile-diversion techniques were applied in response to citizen concern about traffic in residential areas. Public works and planning agencies then developed diversion strategies and implementation programs to address the problems identified. In some of the cities, though, planners recognized the conflict between neighborhoods and traffic, held public meetings to discuss problems and possible solutions, and sought citizen support for implementation. Some of the projects began by providing landscaping and increased resident parking and later evolved into constructing traffic controls to prohibit through traffic.

Many cities installed traffic-diversion devices in older areas, in which the typical street pattern is a grid. As long as traffic volumes were low on residential streets, community concern was small or even nonexistent. But, as areas around the older locations developed and generated more traffic, the philosophy of changing street use from traffic to people gathered support. Thus the approach in areas that had grid-system streets was to change traffic-movement patterns to reflect the manner in which modern subdivisions were developed with curvilinear and nonthrough streets (5).

Some automobile-diversion projects have been provided at spot locations such as in Oak Park (9). The typical approach, however, has been to install traffic restrictions on a citywide or neighborhood basis as part of an overall improvement program. For example, San Francisco and Seattle have constructed diversion projects in those neighborhoods where community support was greatest and, in some instances, where current urban renewal or residential improvement programs were under way.

Experience in San Francisco was focused on neighborhood and district installations. Initial emphasis was on townscaping (landscaping and urban design treatments) that shared equal importance with traffic management. Further interest was demonstrated in discouraging heavy, fast, and through traffic, so more stringent controls at intersection necks, stars, and one-way entrances to two-way streets were installed (5). Subsequently, citizen outcry brought a ballot that resulted in traffic-diversion installation removal (10), although the townscaping efforts have in large part remained.

Experience with traffic diversion in Seattle neighborhoods has shown that, while the targeted streets experienced a reduction in accidents, no discernible changes in traffic volume or accidents have been seen on adjacent arterial streets. Emergency vehicles also did not encounter major inconveniences. Neighborhoods have developed stronger identities, and the environment has been enhanced in the areas of safety (primarily for children) and a general feeling of relative serenity (11).

The city of Berkeley, a university suburb of San Francisco, has a population of 110 000 and has moved toward an overall residential traffic-restraint program after an intensive citizen-participation process. There, traffic-restraint devices have been placed throughout the city. To guide their programs, Berkeley citizens set a rollback goal of 25 percent in total vehicle travel and put great emphasis on transit (12). It is a comprehensive strategy, but the overall consequences are not clearly known to most in the community. Some of the initial findings of Berkeley's program were the following (13):

1. Changes in traffic volumes have occurred generally as expected;
2. Traffic increases on arterial and collector streets have not caused serious increases in congestion;
3. Overall travel times along the city's designated circulation system have not changed significantly from pre-program conditions;
4. Traffic accidents and fatalities decreased over the period the traffic management project was in effect, although injury accidents were up slightly; and
5. There was considerable driver disobedience of all traffic-management device types.

Citizen reaction in Berkeley has been substantial. Groups were formed to protest the barricade installations. Twice the issue of removing or reducing the number of diverters went to the voters and was twice rejected. Concurrently, the protesters took action in Alameda County Superior Court that resulted in a ruling that the diverters must be removed. The Berkeley traffic-management installations are still in place pending an appeal (10).

Overall, citizen reaction has ranged from resident delight over having street traffic decreased to automobile drivers' anger about their street-use privileges being denied. Residents along streets experiencing increased traffic have also complained that traffic problems have not been resolved but only shifted to other locations. At the initiation of traffic-diversion programs, there has usually been an immediate public outcry that tapers off after six months of operation.

These experiences suggest that diversion projects should be installed on a low-cost, temporary basis to gauge community acceptance and interest. After a trial period and modifications, physical devices can then be permanently installed in an attractive manner (5).

These experiences also suggest that a comprehensive approach should be taken to planning and implementing automobile-diversion programs in specific areas. This means considering traffic improvements for those streets to which traffic is to be diverted, as well as developing programs to encourage increased use of transit, carpooling, and nonvehicle modes as part of overall area-improvement programs.

Automobile-diversion experience in Denver has been minimal. Several recent Denver neighborhood plans have recommended traffic diverters, but the background analyses were not substantial and citizen interest in implementation was weak. Those proposals have not been carried out.

The Ellis community organization in the Virginia Village area considered the closing of some streets to through traffic to reduce commercial traffic from Writer's Manor to the west (14, 15). The neighborhood was polled by the organization, but in general the residents seemed unwilling to support the effort. The end result was no change.

STRATEGY-FORMATION ELEMENTS

A traffic-diversion strategy is composed of various elements, from which application features can be identified and guidelines on how to address those features can be formed. These elements (16) are

1. Target population,
2. Travel-behavior effects,
3. Scale of application and zone of influence,
4. Strategy interrelationships,
5. Control degree and mechanism,
6. Institutional and legal factors,
7. Area selection, and
8. Public acceptance.

Target Population

The primary targets of diversion are automobile drivers. Secondary targets are truck drivers who travel on sensitive streets to bypass congested streets or to reduce travel time.

Travel-Behavior Effects

Fundamental traffic-management concepts specify intended effects on targets and the periods of time during which impacts can be expected to be felt after strategy programs have been started. The basic traffic and travel-behavior effects of automobile diversion programs are

1. Changes in traffic-flow operations,
2. Changes in choice of streets,
3. Changes in time of day of trips taken,
4. Changes in modes,
5. Changes in amount of traffic on the various routes, and
6. Changes in number of trips.

The primary travel-behavior impact of automobile diversion is generally on the choice of streets, because the actions imposed make target streets unattractive and alternate paths attractive. Secondary effects normally occur on travel flow, because trips may be made longer and on fewer routes. Concentration of travel demand requires increased use of alternate highways and major streets, which may cause congestion and slower travel times. These effects would be expected to last a short time and to dissipate as driver habits change.

Other travel behavior may be affected only marginally, unless the alternate traffic paths fail to meet demand. On the other hand, traffic redistribution by mode or time of day as part of a comprehensive approach to traffic management may result in secondary impacts on mode choice or even on the times at which people choose to travel.

Scale of Application and Zone of Influence

The spatial areas that can be affected, primarily or secondarily, by automobile-diversion applications include (a) spot (intersection), (b) facility (street, highway), (c) corridor (several parallel facilities), (d) subarea (central business district, activity center, neighborhood, preservation area, historic district, or park location), (e) urban area (city), and (f) region (urban area plus suburbs).

Applied at a spot location, diversion would require a change in path at a specific location. For instance, installation of diverters at a through-street location could change the traffic function to that of a local street. On a smaller scale, automobile-diversion techniques could maintain the function of a designated collector-street function and increase traffic volumes. A sub-area application would be possible for a neighborhood or residential area. Even an entire city may be a site for automobile diversion.

If automobile-diversion programs were applied to a Denver neighborhood, the primary zone of influence would be that area itself. The secondary zone of influence would be the urban area, or even the region if the target area were sensitive enough or the magnitude of vehicle diversion such that regional trips would be affected.

Strategy Interrelationships

Interrelationships between the automobile-diversion strategy and other strategies can be classified as synergistic, independent, overlapped, equivalent, or counterproductive.

If a major effort is made on a diversion project, combinations of several diversion strategies may produce a synergistic effect; i.e., their combined total effect may be greater than the sum of their separate effects. For example, drawing from the strategies cited in the joint FHWA-UMTA regulations (1), a synergistic effect might result from the combined application of (a) through-traffic restrictions in sensitive areas, (b) traffic-operations improvements to facilities designated for concentrated major traffic movement, (c) preferential treatment of high-occupancy vehicles, (d) pedestrian- and bicycle-facility improvements, and (e) transit-service improvements.

Induced TSM strategies that are equivalent to automobile diversion (at least in the intent to reduce vehicle use in congested areas) include encouragement of carpooling and other forms of ride sharing, automobile-restricted zones, and area parking surcharges. In Denver, automobile diversion without the use of induced strategies would in all likelihood be counterproductive to eliminating bottlenecks or reducing major street congestion, because more strain would be placed on those major facilities. Thus, induced disincentives would probably have to be offset by positive improvements.

Control Degree and Mechanism

The degree of control exercised in automobile diversion would be mandatory in the restrictions applied, but voluntary in the choice of alternate streets used by drivers. The mechanism of control for this strategy would be both physical and operational, i.e., through traffic restrictions such as physical barriers or simply sign control.

The following types of management techniques can be used to divert traffic to more appropriate streets or to control vehicle movement: diverters, semi-diverters, street closures, median barriers, traffic circles, "chokers" (street narrowing), traffic signals, stop signs, one-way exit streets, and cul-de-sacs (17). Figure 1 presents several conceptual examples of these techniques, and Figure 2 (18) illustrates a more detailed set with landscaping. Most cities experienced in automobile diversion began with a pilot program and temporary diverters. As problems were resolved and as the program became more acceptable and successful, permanent and attractively landscaped changes were made.

Further, these techniques can be applied to effect the strategy objectives for a given area. Figure 3 shows a street grid before and after that conversion to a protected area with curvilinear flows. Note that traffic is not completely prohibited, but rather redirected to the peripheral routes by diagonal diverters.

Finally, there are many additional measures that can and often must be taken to divert traffic and open up neighborhood space, while allowing emergency vehicles and local access. These include installation of new curbs, realignment of existing curbs, relocation of drainage inlets, adjustment of castings and manhole covers, sidewalk construction and reconstruction, street lighting and signing, accommodation for emergency vehicle crossings, additional fire hydrants, and other nontraffic improvements such as special lighting, landscaping, street furniture, and other urban design treatments.

Institutional and Legal Factors

The primary issue of the automobile-diversion strategy is that the basic decision to implement a specific program is a government one and that the decision makers must consider the concerns of automobile-oriented interests.

Because automobile diversion represents a restraint to through traffic, it results in regulation and restriction of the flow of vehicle traffic. The needs of a sensitive area are thereby elevated to a more prominent position with respect to the dominant automobile. This realignment of planning objectives is certain to result in substantial concern by firmly established automobile-oriented interests in a community, city, and region.

To be successful, the approach to diversion must involve different government agencies in planning and implementation, especially if a synergistic combination of positive TSM strategies is to be achieved. The ability of these agencies to work together is essential to success, and agency cooperation is a function of the extent to which local leadership is willing to pursue innovative and controversial approaches to solving small-area problems. The agencies cover a broad spectrum of municipal affairs and their accepting that automobile diversion will achieve multiple objectives will be determined by how the strategy will affect their own areas of concern.

Legal factors can also be of primary importance in implementing automobile-diversion programs. Legal questions can arise as to the ordinances needed to change the control of streets, e.g., improperly installing stop signs to slow down traffic in an area rather than to stop vehicles purely for an intersection safety problem. If time restraints are installed in a designated area, they also may result in legal action. Finally, changes in traffic control would necessitate enforcement to maintain safety and orderly movement.

Area Selection

Various factors could be considered in selecting an area for a traffic-diversion application. Generally, the choice of area is based on the following criteria: amount of citizen interest, significance of the area's problems, feasibility of the methods to be applied, existence of an on-going neighborhood organization to support the concept, and land uses compatible with access limitations.

Based on other cities' experiences, it would also seem important to select an area, noted for its stability, low-density character, relatively high percentage of children, and transitional nature, that could benefit from a decrease in traffic or better traffic control.

Public Acceptance

The private sector would certainly be a key factor in automobile-diversion success. Like any major urban

Figure 1. Types of suggested automobile-diversion techniques.

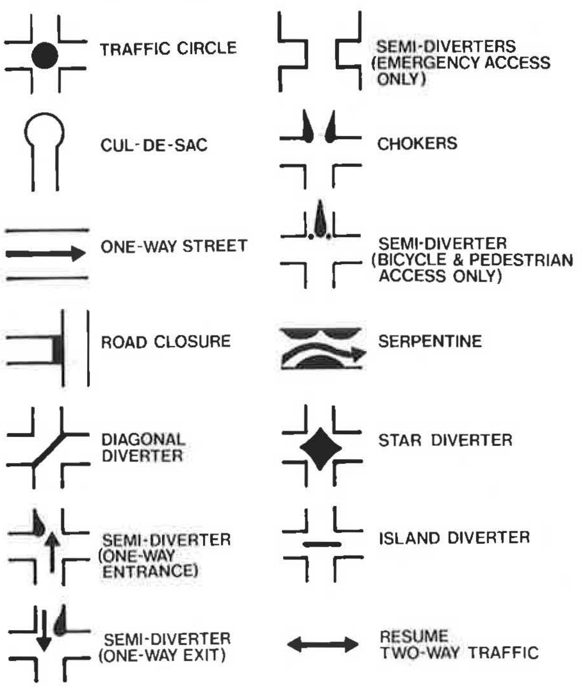
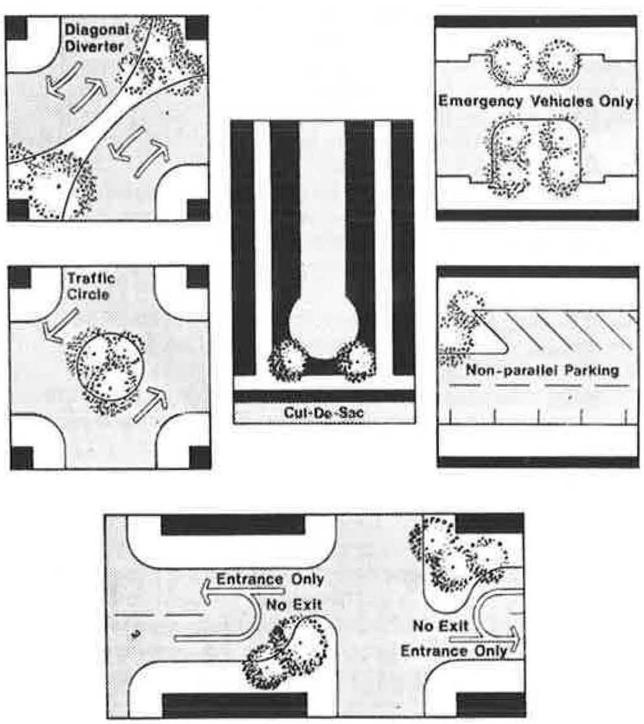


Figure 2. Details of some automobile-diversion techniques.



policy change, planning for this strategy must be conducted with full public participation. Public support and participation will be the most decisive factor in the realization of maximum benefits.

Experience in Berkeley has shown that, because of the very visual nature and potential broad-scale effects of diversions, support of the majority of the public is necessary (5). Large-scale automobile-diversion efforts will not succeed if promoted by a minority or a special-interest group with a single objective.

If government agencies follow a course of action that is negative in nature (restricting traffic), this could alienate most of the interests involved. Positive actions, such as providing incentives to use other paths or modes, must be taken as part of a comprehensive small-area revitalization process founded on strategy goals and objectives.

EVALUATION

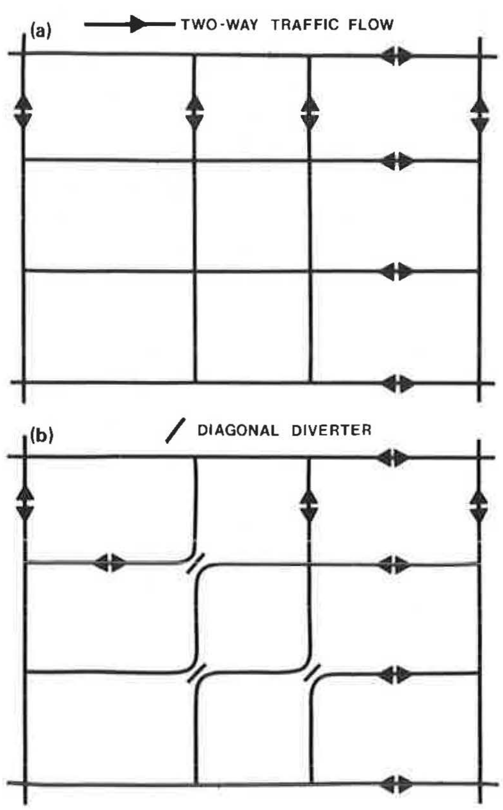
TSM Strategy Effectiveness

It is difficult to judge automobile-diversion strategy effectiveness specifically. Before-and-after studies of a specific case would have to be conducted to determine automobile occupancy, delay, volume, and accident changes and to measure economic, social, and environmental changes. Automobile diversion may require increased efficiency on the major routes to which the traffic might be diverted and would influence more travelers to use transit.

Advantages and Disadvantages

The various major advantages and disadvantages of automobile-diversion programs are described below (5, 11).

Figure 3. Automobile-diversion application.



Advantages

1. Improvement of a small area could be an incentive for middle-class families with children to move back into the area.
2. Diversion can help keep the occasional high-speed vehicle from using primarily residential, local streets as thoroughfares.
3. Selective street closures and the use of cul-de-sacs can provide additional green space, play areas, pedestrian malls, or parking areas for residents or businesses.
4. Closing streets or diverting vehicle traffic may foster a stronger sense of neighborhood or community identity.
5. Crime may be reduced because of increased neighborhood use and surveillance of residential streets and lack of easy vehicle access and escape.
6. Within an affected area, traffic diversion may reduce noise and air pollution, vibration, and perceived or physical crossing of barriers and may make the streets safer for children and other pedestrians.
7. Controlling traffic can act as a catalyst that spurs neighborhood revitalization.
8. Improvements in the public streetscape could provide impetus for rehabilitation of private property.
9. Selective street closures, cul-de-sacs, and traffic-direction controls at local and arterial street intersections can reduce access conflicts and thereby improve traffic flow and safety on arterial streets.

Disadvantages

1. Residents on streets in the vicinity of vehicle diverters may experience higher levels of traffic volume and associated environmental and safety impacts.
2. Diversion may give children or other residents a false sense of safety from motor vehicles.
3. Some cities have found that cul-de-sacs may heighten racial segregation: Closed-off, tightly knit streets may discourage minority families from moving in.
4. Traffic diverted from residential streets may exceed the capacity of adjacent arterial and collector streets and require their upgrading or improvement.
5. Automobile-diversion installations (barriers, signs, islands, and pavement markings) would require additional maintenance by city agencies.
6. Additional right-of-way acquisitions may be necessary for both the target and alternative streets; for instance, diagonal diverters and cul-de-sac construction could be restrained by insufficient existing right-of-way.
7. Diversion may result in access problems and inconvenience for residents and visitors on the affected streets and in the vicinity of the diverters.
8. Access for police, fire, and other emergency vehicles may be hampered and response times may be increased unless adequate provisions are made to ensure access for such vehicles.
9. Application of traffic restrictions without positive and compatible strategies may result in negative reactions from agencies and the public.
10. Traffic may not be eliminated but rather only redistributed.
11. Not all residential streets can have heavy traffic removed by traffic diversion; where traffic impacts on residential streets cannot be reduced through street or transit improvements, those impacts should be offset by public trade-offs such as street landscaping and noise buffers.

CONCLUSIONS AND RECOMMENDATIONS

This study of automobile diversion led to the following conclusions and recommendations pertaining to potential planning and implementation in Denver.

Conclusions

1. Denver's role in the region is changing from that of a well-balanced residential community to that of a maturing service core for the entire region. As the region grows, Denver will experience traffic increases that will affect sensitive areas.
2. Motor-vehicle traffic has direct negative effects on crossing hazards; on air, noise, and dirt pollution; on vibration; and on parking-operation inconvenience.
3. Environmental street-capacity studies have been conducted in various cities; results have varied. To establish the environmental capacity of any one particular area, impact studies and attitude surveys would have to be conducted to determine tolerance to traffic volumes.
4. Several cities, most of them on the West Coast, have applied automobile-diversion techniques with varying degrees of success.
5. Citizens in an affected area may place more value on traffic control and access than on traffic impediments.
6. Automobile diversion can induce more efficient use of major streets around affected areas.
7. The installation of traffic-management devices can modify established neighborhood traffic patterns so that they resemble the curvilinear and non-through-street patterns of modern subdivisions.
8. An automobile-diversion program in a specific area should be part of an overall improvement effort that is approached in a comprehensive and positive manner and should include other compatible TSM strategies such as increased transit, carpooling, and nonvehicle modes.
9. Automobile-diversion techniques can be used to achieve functional designations. For instance, traffic diverters installed at a spot location can change a through-street traffic function to that of a local street or can maintain the designated function of a collector street and prevent arterial-street function and increased traffic volumes.
10. Substantial public support is necessary if an automobile diversion program is to be successful.
11. Improperly installed diversion devices may increase safety hazards, e.g., a diagonal diverter that does not allow the proper sight distance for the posted speed limit may cause accidents.
12. Limited right-of-way in the established portions of the city may prevent some automobile-diversion installations, unless the expected benefits justify property acquisition.
13. Application of automobile diversion in Denver has potential, but implementation at any scale must result from full identification of the problems and issues involved, sound technical and policy analysis of all available alternatives and impacts, and substantial support from all parties interested in the effort.

Recommendations

1. If major public interest is expressed in automobile diversion, the city should prepare and distribute newsletters that explain negative aspects of local traffic, identify the potential benefits of diversion, and suggest a process by which to initiate projects.
2. Further considerations should be given to mea-

suring traffic impacts on residential streets in Denver, possibly by use of environmental capacity studies, involving traffic, noise, safety, and attitude surveys.

3. The automobile-diversion strategy goals, objectives, and techniques contained in this report should be applicable to a specific area in Denver, if potential benefits that outweigh potential detriments can be determined and if support is evidenced by all involved interests and decision-making groups.

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Development and Application of a Freeway Priority-Lane Model

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This report describes the status of freeway priority lanes in the United States, the development of a freeway priority-lane simulation model (FREQ6PL), and the application of the model to a real-life situation. Of the five feasible types of priority lanes, normal-flow exclusive lanes that reserve one or more lanes for priority vehicles are the most prevalent. FREQ6PL can simulate one or more lanes used exclusively by priority vehicles (buses only or vehicles of either three or more or two or more occupants). Three points in time are simulated: the before situation (no exclusive lane), the short-term after situation (the first day of operations with no traveler demand responses), and the longer-term after situation (3-6 months later, after spatial and modal shifts). Performance is measured by an integrated measure of effectiveness that includes costs of travel time, fuel consumption, and vehicle emissions and facility operating and maintenance costs. The model was applied to the Santa Monica Freeway in two parts: (a) to the priority cut-off limit, number of reserved lanes, and length of the exclusive lane and (b) to different parallel arterial speeds, different levels of arterial spare capacity, and different hypothetical mode shifts. It was concluded that reserving an existing or added freeway lane on such a freeway will at best make its performance as good as before and at worst significantly poorer in both the short- and longer-term situations.

In recent years the emphasis in transportation planning has shifted from long-term, capital-intensive, capacity-increasing projects to shorter-term, relatively low-cost projects aimed at using existing transportation facilities more efficiently, by stressing energy conservation and environmental impact analyses.

In September 1975, the Urban Mass Transportation Administration (UMTA) and the Federal Highway Administration (FHWA) issued joint regulations (1) that established planning requirements for such projects in urban areas. These regulations placed heavy emphasis on transportation system management (TSM). The following major categories of TSM actions were identified:

1. Actions to ensure the efficient use of existing road space through
   a. Traffic-operation improvements to manage and control the flow of motor vehicles,
   b. Preferential treatment for transit and other high-occupancy vehicles,
   c. Appropriate provisions for pedestrians and bicycles,
   d. Management and control of parking, and
   e. Changes in work schedules, fare structures, and automobile tolls to reduce peak-period travel and to encourage off-peak use of transportation facilities and transit services;
2. Actions to reduce vehicle use in congested areas;
3. Actions to improve transit service; and
4. Actions to increase internal transit-management efficiency.

Use of exclusive lanes on urban freeways is a TSM technique that provides preferential treatment to high-occupancy vehicles. The terms "exclusive", "priority", and "reserved" lanes are used interchangeably in this report and refer to freeway lanes reserved for the exclusive use of vehicles with two or more occupants, vehicles with three or more occupants, or buses only.

The Institute of Transportation Studies (ITS) at the University of California, Berkeley, has done several types of TSM research over the past decade (2). The Traffic Management Group dealt with freeway emergency detection systems (3, 4), freeway corridor operations studies (5, 6), priority operations (7, 8), traffic management of surface streets (9-11), and traffic management on freeways (11-13). The research on exclusive lanes on urban freeways described here continues this work.

STATUS OF FREEWAY EXCLUSIVE LANES IN THE UNITED STATES

While exclusive lanes on urban arterials are used worldwide, exclusive lanes on freeways are used primarily in the United States. Figure 1 classifies 13 such uses in terms of the following four variables: (a) access to and egress from the exclusive lane, (a) access to and egress from the exclusive lane, i.e., standard right-hand on- and off-ramps, both right- and left-hand on- and off-ramps, or special ramps used only by priority vehicles; (b) the lanes reserved, i.e., the median lane in the peak flow direction, the median lane in the non-peak direction, the outer lane in the peak direction, or a separate roadway for the exclusive use of priority vehicles; (c) the priority cut-off level, i.e., how priority vehicles are defined in terms of the number of occupants; and (d) number of reserved lanes. The 13 identified uses (14-17) in chronological order of implementation are

1. Shirley Highway, Virginia, 1969;
2. I-495 approach to Lincoln Tunnel, New York, 1970;
3. Southeast Expressway, Boston, 1971;
4. Long Island Expressway, New York, 1971;
5. US-101, Marin County, California, 1972;
6. San Bernardino Busway, Los Angeles, 1973;
7. I-93, Boston, 1974;
8. Moanalua Freeway, Honolulu, 1974;
9. I-95, Miami, 1975;
10. CA-280, San Francisco, 1975;
11. Banfield Freeway, Portland, Oregon, 1975;
12. Santa Monica Freeway, Los Angeles, 1976; and
13. CA-580, San Francisco Bay Area, 1977.

The clear trend is for one or more of the existing freeway lanes to be reserved for priority vehicles; this is the most prevalent type.

MODEL DEVELOPMENT

An existing freeway priority entry-control model, FREQ5CP (6), was selected as base model for FREQ6PL, which was developed primarily to evaluate type 1 exclusive lanes but can also evaluate special cases of types 2 and 5.

Model Structure

Figure 2 shows the new model's structure. In the following description step numbers refer to the numbers in Figure 2.

Steps 1-5 represent input to the program. Freeway design features include subsection lengths, subsection capacities, subsection speed-flow curves, position and capacities of on- and off-ramps, grades, curvature, surface texture, and number of lanes. The lane definition refers to which strategy is being investigated in terms of position, time, and the priority cutoff limit.

The freeway demand pattern refers to the origin-destination (O-D) tables and occupancy distribution at each on-ramp. O-Ds may vary from time slice to time slice over the peak period. The alternate route speeds are those specified for different sections of the alternate route and represent the level of service on it. The measure of effectiveness (MOE) refers to the money values placed on the different MOEs by the user. This is discussed below.

Step 6 simulates peak-period traffic operations for the before situation, or no exclusive lane. The results of the simulation, expressed in terms of the performance index (PI), will serve as the basis of comparison for later simulations.

Step 7 is an option in case the user is interested in only the before situation.

In step 8 the structuring of the exclusive lane refers to the splitting of O-D tables (by the program) into different occupancies, changes in the roadway capacities, and other manipulations necessary before the short-term after situation can be simulated. This is also discussed below.

Step 9, the short-term performance with an exclusive lane, is an effort to simulate the first day of operations before drivers have changed their behavior; i.e., all vehicles have the same time, space, and occupancy patterns as before. Performance is expressed in terms of the PI.

Step 10 is an option in case the user wants to compare only the before and short-term after situations.

In step 11, spatial shift refers to certain nonpriority drivers diverted to alternate parallel routes. The spatial shift algorithm is discussed later on.

In steps 12-14, mode shift refers to occupants of non-

Figure 1. Classification of lane and ramp types.

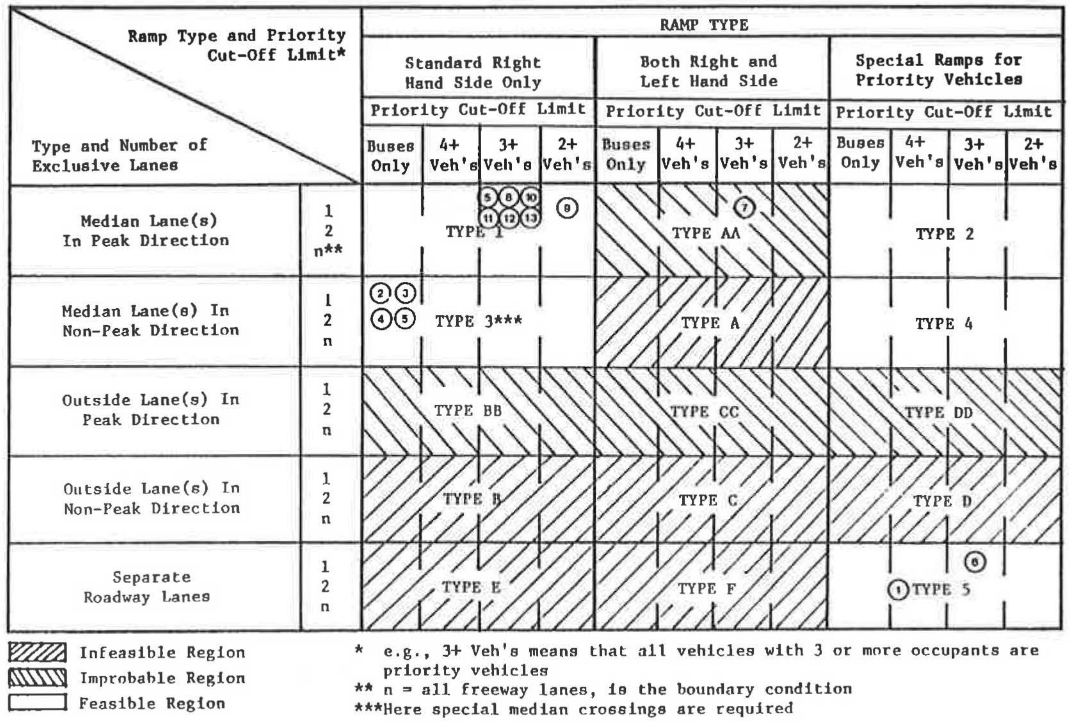
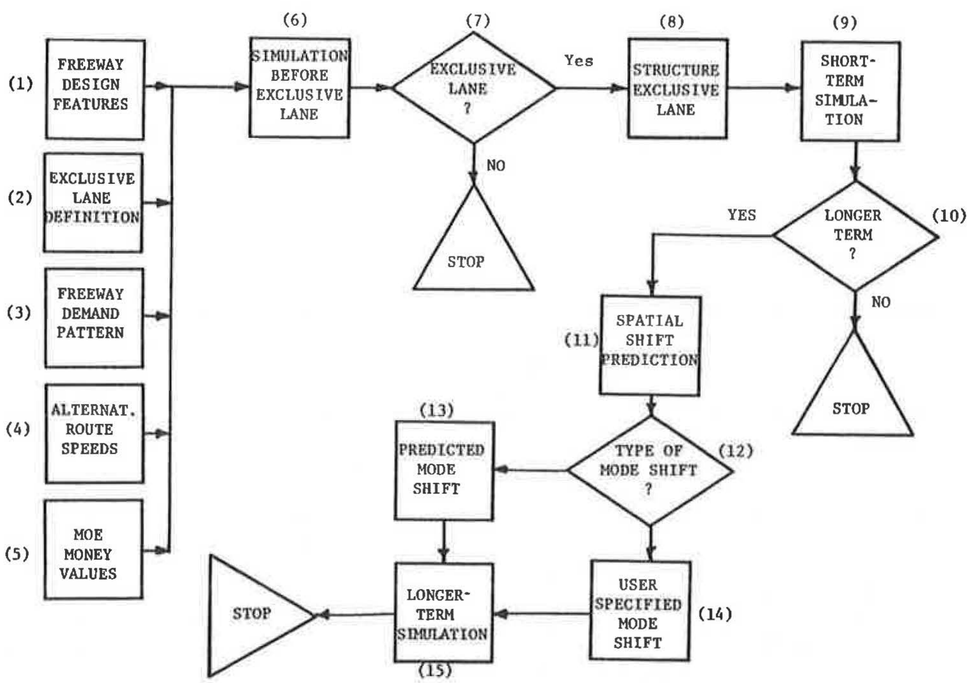


Figure 2. Structure of FREQ6PL model.



priority vehicles who shift to either buses or carpools. Mode shift is either predicted from travel-time differences between priority and nonpriority vehicles or is calculated from user-supplied mode-shift magnitudes.

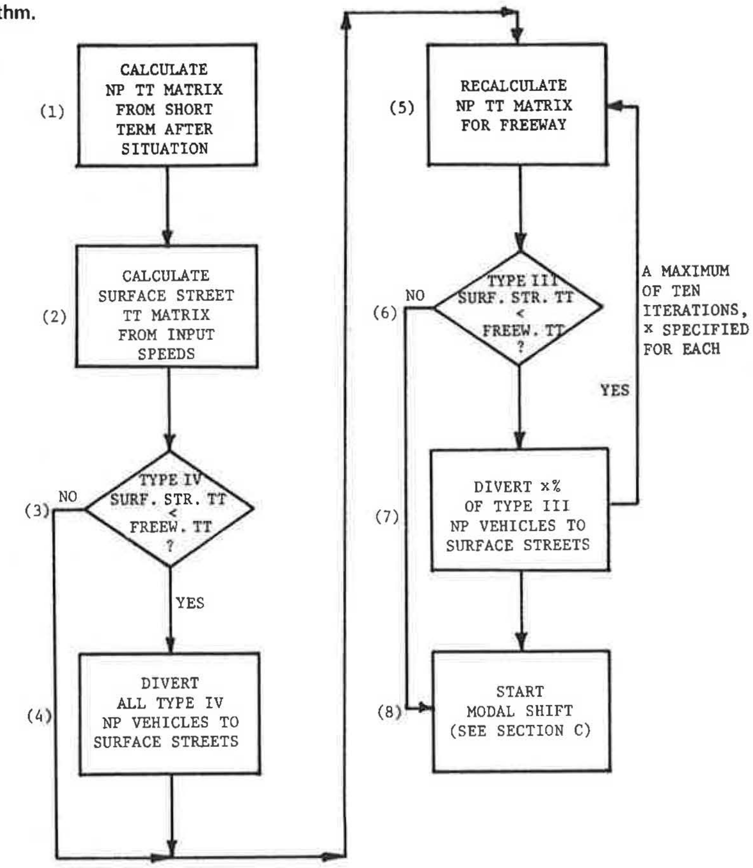
Step 15, the longer-term after simulation, is an effort to simulate operations three to six months after implementation of the exclusive lane, after the demand responses of spatial shift and modal shift have occurred.

Performance Index

PI is defined in this study as costs, in dollars per year, in terms of certain selected MOEs (travel time, fuel consumption, vehicle emissions, construction costs, freeway operating costs, and freeway maintenance costs), of serving a fixed number of people on a freeway (with or without an exclusive lane) for a specific modal split. The situation without the exclusive lane is the base situation, to which the short-term and longer-term after situations for different exclusive-lane designs are compared. Differences in PIs represent either yearly cost reductions (or gains) or yearly cost increases (or losses):

$$PI = TTC + FCC + VEC + CC + FOC + FMC \tag{1}$$

Figure 3. Structure of spatial-shift algorithm.



where

TTC = yearly travel time costs,
FCC = yearly fuel consumption costs,
VEC = yearly vehicle emissions costs,
CC = yearly construction costs,
FOC = yearly freeway operating costs, and
FMC = yearly freeway maintenance costs.

The definition implies that (a) the model will estimate the six cost elements for a given freeway demand, freeway design, and exclusive lane design; (b) the functional variables influencing PI and considered by the model include: exclusive lane type, location of exclusive lane, time duration of exclusive-lane operations, number of exclusive lanes, existing modal split, priority cutoff limit, level of service on the parallel surface streets, and quality of bus service as reflected in mode-shift sensitivity; (c) each of the MOEs must have a known dollar value, supplied by the user, such as a time value of $3.00/person-hour; and (d) PI expresses yearly costs for one peak period per day for the peak directional flow only.

Simulation Submodel

The FREQ6PL simulation submodel performs the following series of simulations:

1. The freeway before implementation of the priority lane,
2. The priority lane in the short-term after situation,
3. The nonpriority lanes in the short-term after situation (including lanes adjacent to the priority lane as well as general purpose lanes before the exclusive lane started and after it terminated),
4. Several iterations of the priority and nonpriority lanes (in order to predict spatial shift and modal shift),
5. The priority lane after spatial and modal shifts have occurred, and
6. The nonpriority lanes after spatial and modal shifts have occurred.

In order to perform these simulations, the original freeway O-D demand is transformed into a priority and a nonpriority O-D. This is done by using the specified priority cutoff limit and four synthetic O-Ds in the following way.

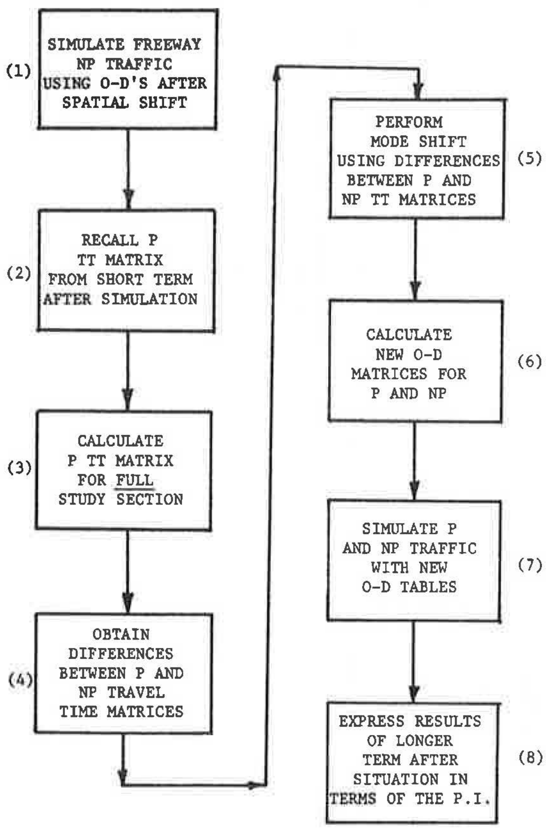
The first synthetic destination "delivers" the priority vehicles from the nonpriority lanes into the priority lane where the priority lane begins. The first synthetic origin then "accepts" these priority vehicles into the priority lane, and the second synthetic destination "delivers" the priority vehicles (with destinations downstream of the priority lane end) from the priority lane into the nonpriority lanes. The second systhetic origin "accepts" these priority vehicles into the nonpriority lanes downstream of the priority lane end.

The model automatically reduces the capacity of the nonpriority lanes along the length of the priority lane and makes further adjustments for weaving into and out of the priority lane. It also allows for different priority cutoff levels (two or more or three or more occupants or buses only), different speed-flow curves on different priority or nonpriority lane subsections, and a different number of reserved lanes.

Spatial Shift

Figure 3 outlines the structure of the spatial-shift algorithm. In the following discussion, step numbers

Figure 4. Structure of modal-split algorithm.



refer to numbers in Figure 3.

In step 1 the nonpriority travel-time matrix is calculated for all O-D pairs and all time slices from the short-term after simulation. Then in step 2 the surface street travel-time matrix is calculated for all O-D pairs from the surface street subsection input speeds. In steps 3 and 4, if type IV trips can save time for any type IV O-D, all such nonpriority vehicles are diverted to the corresponding surface street subsections.

In step 5, after this diversion, the whole peak period is resimulated and new nonpriority travel-time matrices are calculated for each time slice.

Steps 6 and 7 are an incremental assignment procedure where as many as 10 increments of type III traffic are assigned to the surface streets if they can save time. After each assignment the nonpriority freeway traffic is resimulated. The reason for this incremental assignment is that the surface street speeds are assumed to be constant, which would make it very easy to overload the surface streets and cause free flow on the freeway if an all-or-nothing assignment is used. With type IV an all-or-nothing assignment can be used, because type IV traffic normally forms a relatively small portion of freeway demand.

In step 8, after the spatial shift has been completed, the modal shift is predicted. This is described below.

Modal Shift

Predicted Modal Shift

The underlying principle of the modal-shift algorithm is that travel-time differences between priority and nonpriority vehicles are used to predict modal shifts from nonpriority to priority vehicles. Modal-shift sensitivities resulting from the calibration of a multinomial logit model are used (6) to predict the shift.

Figure 4 outlines the structure of the modal shift algorithm. In the following discussion step numbers refer to numbers in Figure 4.

In step 1, after the spatial shift is completed, nonpriority traffic on the freeway is simulated by using the new nonpriority O-D matrices.

In steps 2-4, the short-term after situation, the priority-lane traffic was simulated. However, priority vehicles may also travel certain distances in general purpose lanes before the beginning of the priority lane and after it has ended. Travel-time differences between priority and nonpriority vehicles are therefore calculated over the full distance from an origin to a destination, including distances traveled in general-purpose lanes.

In steps 5 and 6 the FREQ5CP modal-shift sensitivities are used to perform the shift from nonpriority to priority vehicles. Priority vehicles, as discussed before, can be defined as vehicles with either two or more or three or more occupants or buses only. Two new sets of O-Ds are obtained after the modal shift: one for priority vehicles and one for nonpriority vehicles.

In steps 7 and 8 the new O-D tables are used to simulate the final longer-term after situation on the freeway, which again will consist of the priority-lane traffic simulation and the non-priority-lane simulation. The results of the longer-term after simulations are again expressed in terms of the PI and are compared with the before situation.

Specified Modal Shift

The purpose of the specified modal shift is to allow the model user to address such questions as, What happens if the expected modal shift is totally different from that predicted because of travel-time differences only? That is, if a priority lane is implemented when bus fares have decreased and parking costs and fuel costs have increased, the expected shift will be greater than that based on travel-time differences alone.

Too much shift may cause the priority lane's demand to exceed its capacity, which would then defeat one of the purposes of the lane: providing priority vehicles with a travel-time savings. This, in fact, may cause the total costs, as expressed in the PI, to increase. What would be an optimum modal split for a given exclusive lane design?

Depending on some of the external impacts, such as home use of automobiles after a modal shift, the PI may at a given point increase as more modal shifts take place.

Figure 5 outlines the structure of the specified modal shift procedure. In the following discussion step numbers refer to numbers in Figure 5.

Steps 1-3 refer to the simulation of the freeway before implementation of the exclusive lane, the short-term after simulations of both the priority lanes and the nonpriority lanes, and the simulation of the nonpriority lanes after spatial shift has taken place.

In step 4, whereas the predicted modal shift described above made use of shift sensitivity values, the modal shift now is calculated by using specified modal-shift magnitudes. A modal-shift magnitude of 0.2, for example, means that 20 percent of the total existing passenger demand would shift from nonpriority vehicles to priority vehicles. Separate shift magnitudes are specified for carpools and buses.

Step 5 occurs after the priority and nonpriority O-D tables have been changed. The longer-term after situation is simulated and compared to the before situation.

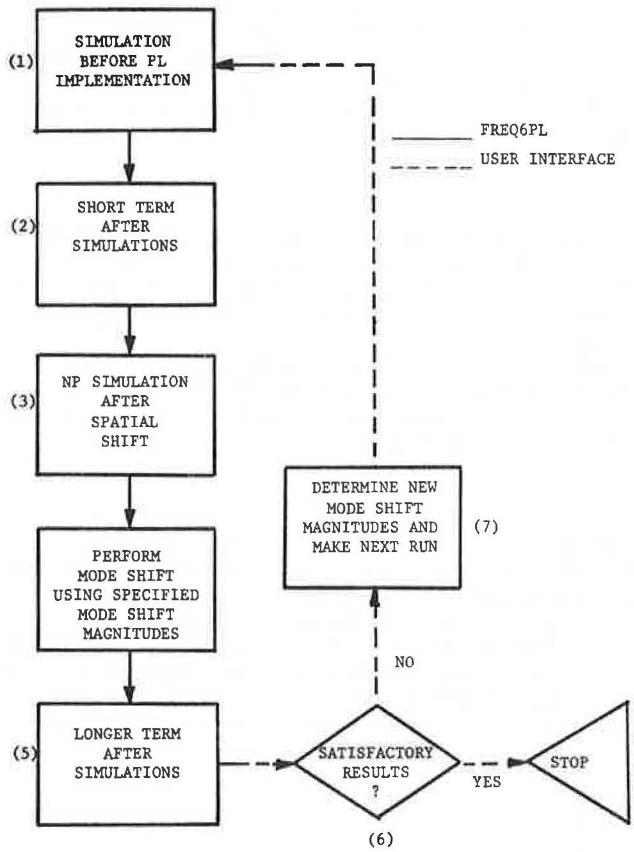
In steps 6 and 7, the user examines the output from the longer-term after with the specified modal-shift magnitudes and, if so desired, decides on a new set of shift magnitudes in order to make another computer run. Different hypothetical modal shifts, compatible with different stimuli (e.g., reduced bus fares or reduced bus fares and decreased parking availability), can then be investigated for a particular exclusive-lane design.

Model Application

The model was applied to the Santa Monica Freeway in the Los Angeles metropolitan area. Data used included actual freeway design features, occupancy distributions for each on-ramp, and O-D data for a 4-h morning peak period. This peak period was divided into sixteen 15-min time slices. The Santa Monica Freeway is essentially an eight-lane facility with a 6.7-m (22-ft) median.

Construction, operating, and maintenance costs, respectively, were taken as \$100 000, \$60 000, and \$10 000/year, and the following money values were assigned to (a) time: \$3.00/h; (b) fuel: \$0.17/L (\$0.65/gal); and (c) vehicle emissions: \$2.55/kg (HC), \$0.02/kg (CO), and \$0.46/kg (NO_x) costs.

Figure 5. Modal-split optimization procedure.



Design of Experiment

The experiment was designed to investigate the following primary variables in the design of a type 1 exclusive lane: (a) length of the exclusive lane, (b) priority cut-off limit, (c) number of reserved lanes, and (d) time duration of exclusive lane. The design of the experiment is shown in Figure 6 and is discussed below.

Part 1 is an analysis of existing conditions. Before any traffic-management strategy can be designed and implemented, it is necessary to understand the existing conditions well. The existing conditions are also needed as a basis of comparison. The analysis of existing conditions is described below.

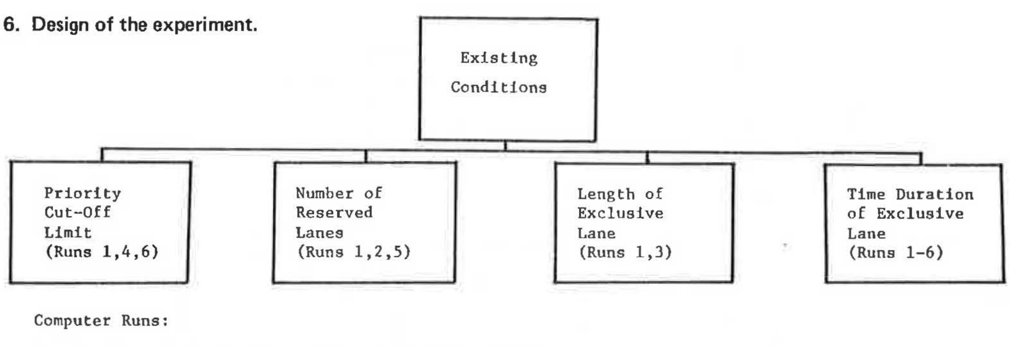
Part 2 is the priority cutoff limit. Three priority cutoff limits are investigated: buses only, all vehicles with three or more occupants, and all vehicles with two or more occupants. The analysis is done for both the short and the longer term.

Part 3 is the number of lanes. Three different lane configurations are investigated: one of the existing lanes reserved for vehicles of three or more occupants, two lanes (one of which is added) reserved for vehicles of two or more occupants, and one added lane for vehicles of three or more occupants. The analysis is done for both the short and the longer term.

Part 4 is the length of the exclusive lane. Two designs are investigated: a long exclusive lane and a short exclusive lane. The analysis is once again done for both the short and the longer term.

Part 5 is the time duration of exclusive lane. The congestion pattern in terms of when congestion starts and when it ends is investigated for all the alternatives.

Figure 6. Design of the experiment.



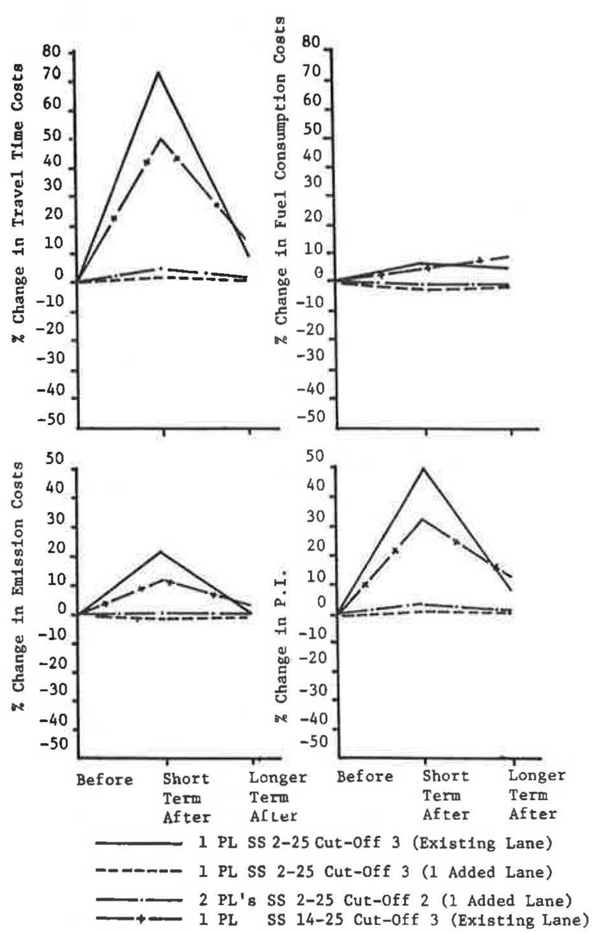
Computer Runs:

1. One Long Exclusive Lane, Priority Cut-Off Limit = 3
2. One Added Long Exclusive Lane, Priority Cut-Off Limit = 3
3. One Short Exclusive Lane, Priority Cut-Off Limit = 3
4. One Long Exclusive Lane, Priority Cut-Off Limit = 2
5. Two Long Exclusive Lanes, Priority Cut-Off Limit = 2, One Long Lane Added
6. One Long Exclusive Lane, Priority Cut-Off Limit = Buses Only

Summary of Results

Figure 7 shows the predicted performance of the different exclusive-lane designs in terms of the relative changes in travel time, fuel consumption, vehicle emissions, and PI. By using Figure 7, the results of the model application can be summarized.

Figure 7. Predicted performance of lane designs.



Travel-Time Costs

Using an existing lane as a priority lane, regardless of length, has severe consequences in the short term and in the long term is still worse than the existing condition.

Adding a lane and then reserving either one or two lanes (with cutoff levels of three and two) does not result in drastic changes in either the short or the longer term.

Fuel Consumption Costs

Using an existing lane as a priority lane, regardless of the length, results in increased fuel consumption in both the short and the longer term.

Adding a lane and then reserving either one or two lanes (with cutoff levels of three and two) has virtually no effect on the fuel consumption in both the short and the longer term.

Vehicle Emissions Costs

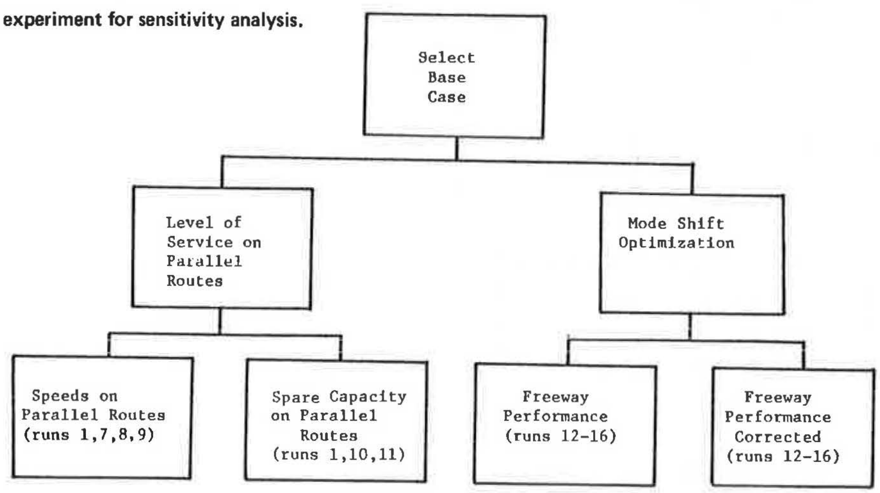
Using an existing lane as a priority lane, regardless of the length, results in increased emissions costs in the short term, whereas in the longer term total emissions costs do not differ from those of the existing situation.

Adding a lane and then reserving either one or two lanes (with cutoff levels of three and two) has virtually no effect on the vehicle emissions costs in both the short and the longer term.

Performance Index

The shape of the PI curve corresponds to the shape of that of travel time costs, which illustrates that travel-time costs are relatively much more important than either fuel or vehicle emission costs in calculating PI. The model application can be summarized by the following two statements. Taking away an existing lane for the exclusive use of priority vehicles results in severe short-term consequences, and even in the longer term

Figure 8. Design of the experiment for sensitivity analysis.



Computer Runs:

1. PL SS 2-25, 25 mph. Arterial Speed, unlimited spare capacity.

7-9. PL SS 2-25, Arterial Speeds of 0, 15 and 35 mph., unlimited spare capacity.

10,11. PL SS 2-25, 25 mph. Arterial Speed, some and little spare capacity.

12-16. Five sets of hypothetical mode shifts.

is still worse than the existing condition. Adding a lane and then reserving either one or two lanes (with cutoff levels of three and two) does not result in any significant changes in either the short or the longer term.

Sensitivity Analysis

The following variables were investigated in the sensitivity analysis: different parallel arterial speeds, different levels of arterial spare capacity, and different hypothetical modal shifts.

Design of Experiment

Figure 8 illustrates the design of the experiment for the sensitivity analysis, which was divided into three parts.

Part 1 was the selection of a base case; part 2 was the investigation of the effect of the level of service on the parallel arterials in terms of the average speed existing on the arterials and the spare capacity available on the arterials. Four arterial speeds were investigated for the base case, 0 km/h in run 7, 24 km/h (15 mph) in run 8, 40 km/h (25 mph) in run 1, and 56 km/h (35 mph) in run 9. Also, three levels of available spare capacity on the arterials were investigated for the base case: unlimited spare capacity on run 1, some spare capacity on run 10, and little spare capacity on run 11. Part 3 was the investigation of the effect of different hypothetical modal shifts on the freeway traffic performance as reflected in the uncorrected and the corrected PI, for the case of no available parallel arterials.

Summary of Results

The results of the sensitivity analysis are illustrated in Figures 9, 10, 11, and 12, about which the following comments can be made.

Figure 9. Longer-term vehicle distances for different speeds.

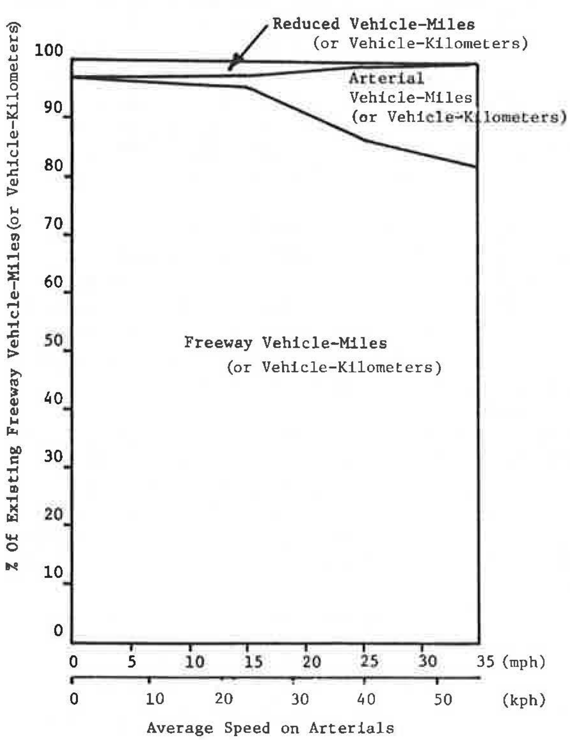


Figure 9

In Figure 9 the maximum reduced freeway vehicle kilometers occur at an arterial speed of 0 when no diversion takes place and maximum mode shift results. The maximum predicted freeway vehicle-kilometer reduction is about 3 percent.

From 0 to 24 km/h (0 to 15 mph) the reduced vehicle-kilometers curve is relatively flat, because very little diversion takes place. Average speeds on the freeway are higher than 24 km/h for nearly all O-D pairs in nearly all time slices.

From 24 to 56 km/h (15 to 35 mph) diversion increases rapidly, and, at an arterial speed of 56 km/h, 17 percent of the vehicle kilometers traveled in the longer-term after situation are on the arterials. This heavy diversion again results in improved nonpriority traffic performance and therefore virtually no mode shift.

Figure 10

In Figure 10 at a 0-km/h arterial speed the longer-term after situation is significantly better than the short-term after situation. This improvement is a result of the modal shift. All elements of the PI improve significantly over the short-term after performance.

At a 24-km/h arterial speed very little diversion occurred, as illustrated in Figure 10. However, the diversion that did occur resulted in improved freeway performance and a 16 percent reduction in total travel time. All elements of the PI show an improvement when compared to the 0-km/h case.

At a 40-km/h (25-mph) arterial speed heavy diversion (13 percent of longer-term vehicle kilometers) takes place and results in reduced travel time and vehicle emissions but increased fuel consumption. The PI is still about 9 percent more than the before situation.

At a 56-km/h arterial speed both total travel time and vehicle emissions are less than the before situation, while fuel consumption shows an 8 percent increase over the before situation. The net effect is that the PI is about equal to what it was in the before situation.

Figure 11

In Figure 11 the case of little spare capacity on the arterials does not represent a realistic longer-term after situation, simply because many vehicles will divert back to the freeway because of the low speeds (caused by the diverting traffic) on the arterials. It does, however, illustrate clearly that total costs may be increased drastically by congestion caused by the diverting vehicles.

The reason why the fuel costs do not change in Figure 11 is that a fuel marginal cost factor of 1 was used for all three levels of congestion.

The significance of the shape of the cost curves in Figure 11 does not lie in the actual magnitudes of the cost increases but in the fact that the upper boundary case (little spare capacity) gives drastically different results than the lower boundary case (unlimited spare capacity). Using marginal cost factors of 1 (or assuming unlimited spare capacity on parallel arterials) would therefore definitely underestimate the total costs.

Figure 12

In Figure 12, the more extensive the modal shift, the better the freeway traffic performance, as illustrated

Figure 10. Performance for different speeds.

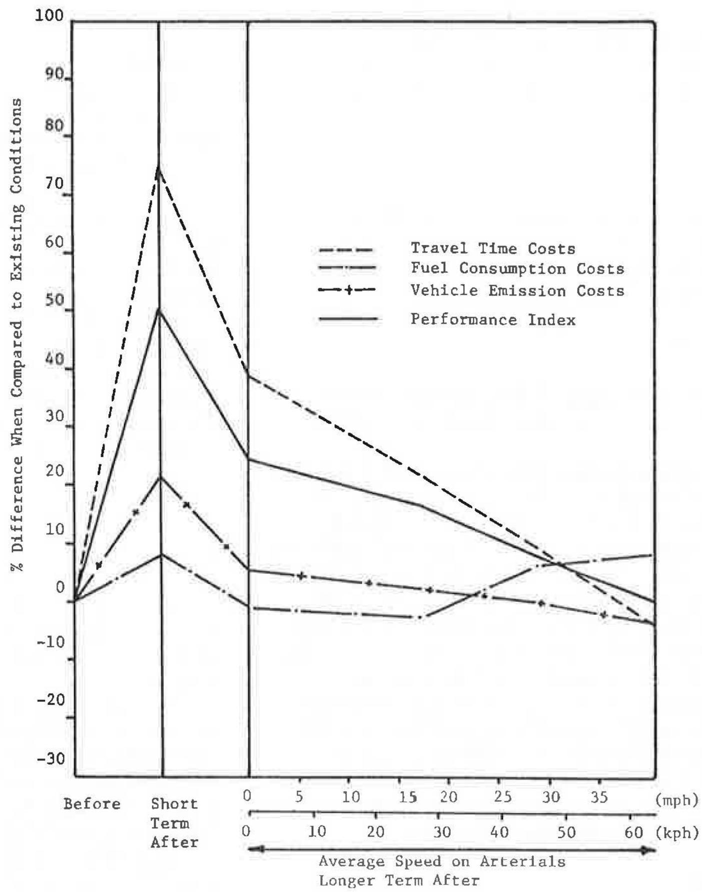
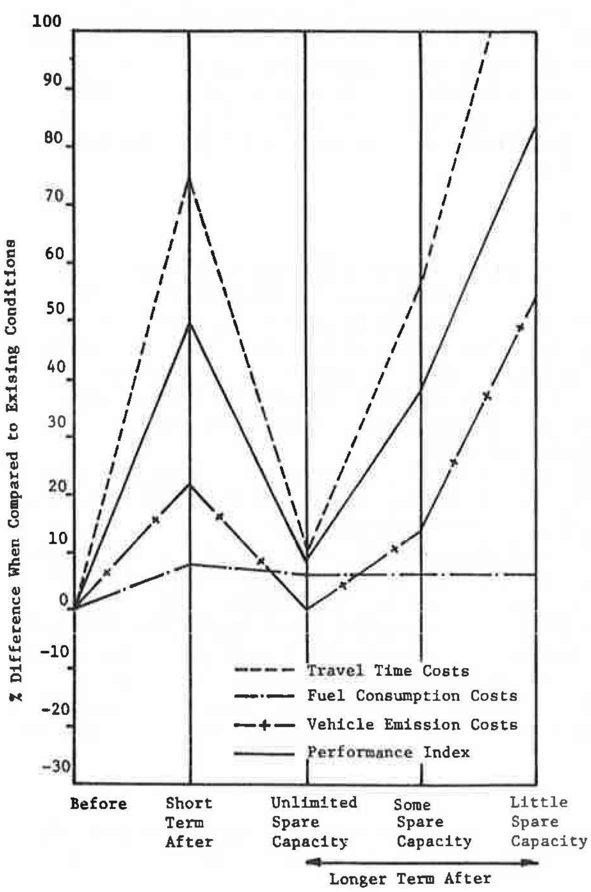


Figure 11. Performance for different capacities.



by the uncorrected PI. This continues until run 16, when the priority lane becomes congested.

Only after substantial specified modal shift (4.5 percent to carpools and 6.4 percent to buses) does the uncorrected PI become less than the before PI. The longer-term after traffic simulation for this case provides the following information: The maximum volume-to-capacity (V/C) ratio in the priority lane is 0.52 and occurs in time slice 4. The nonpriority lanes are congested from time slice 2 to time slice 10 (compared to congestion in the before situation from time slice 3 to time slice 9). The predicted modal shift results in a shift of 2 percent to carpools and 0.6 percent to buses, which is about 25 percent of the shift required to break even.

Shifts of 0.192 to carpools and 0.256 to buses result in a congested priority lane, which is obviously something that will not occur. Priority vehicles will not use the priority lane if they cannot save time by doing so.

The corrected PI does not differ much from the uncorrected PI in the predicted modal-shift range, in spite of the rather unfavorable data used: All vehicles left at home will be used on an 8-km (5-mile) trip.

The corrected PI shows a minimum at run 15, which is explained as follows: As more modal shift occurs, the freeway benefits become relatively smaller and external costs relatively larger. At a shift of 9.6 percent to carpools and 12.8 percent to buses, the freeway gains equal the external costs (primarily the bus and carpool time penalties).

Further modal shift provides greater costs than gains. The reason why the corrected PI never becomes less than the before-situation PI is primarily that the priority lane cannot produce enough time savings to offset the time penalties for bus and carpool specified for these runs.

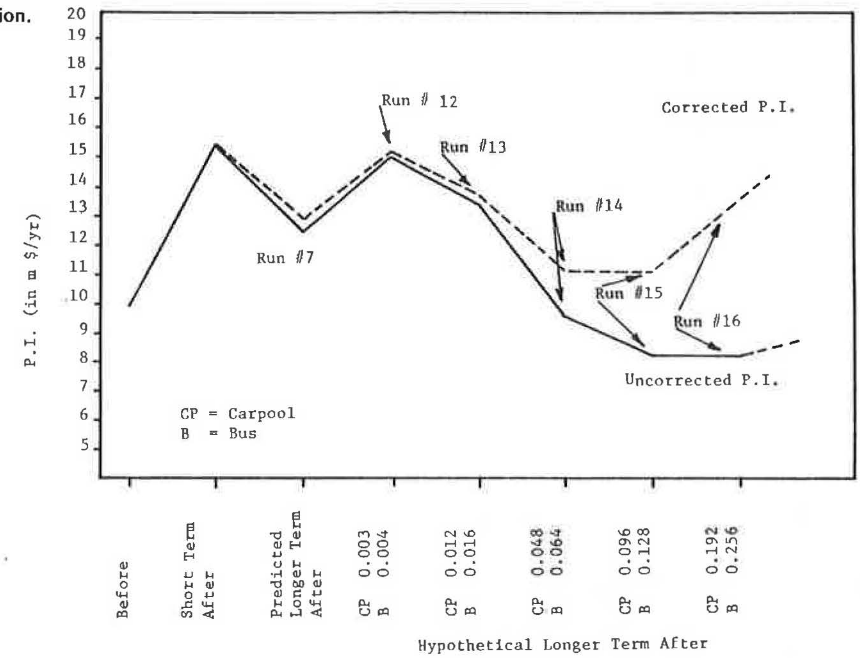


Figure 12. Modal split optimization.

CONCLUSIONS

The results of the research are summarized by the following three general conclusions.

1. A type 1 exclusive lane on a congested freeway is expected to compare unfavorably with the before situation in both the short-term and the longer-term after situations, considering total travel time, fuel consumption, and vehicle emissions.
2. A type 1 exclusive lane on a relatively uncongested freeway is expected to perform as well as or slightly worse than the before situation in both the short-term and longer-term after situations, considering total travel time, fuel consumption, and vehicle emissions.
3. There may be some operating environments significantly different from the Santa Monica environment in terms of occupancy distribution, level of bus service, modal-shift propensity, and parallel arterials. If a type 1 exclusive lane is considered in such an environment, it is recommended that an in-depth analysis be undertaken from the specific type 1 exclusive-lane design before deciding to implement it.

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Safety Considerations in the Use of On-Street Parking

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The research was intended to examine relations among parking configurations (angle, parallel, or no parking), parking density, traffic flow, street width, pedestrian activity, and highway safety. The variables found in this research to be associated with accident rates include (a) functional classification of streets, (b) parking use, and (c) abutting land use. An important and surprising fact is that parking configuration did not emerge as a variable that in itself was related to accident rate. Increased parking use was found to result in significantly higher accident rates, as many as 900 000 space hours per kilometer per year (1 500 000 space hours per mile per year). Streets abutting land uses that generate high parking turnovers and pedestrian activity have higher accident rates than those abutting lower-intensity land uses. Heavily used parallel-parking areas were found to have accident rates comparable to heavily used high-angle-parking areas. Prohibition of parking resulted in the lowest accident rates measured. Parking-related midblock accidents accounted for 49 percent of all accidents along major streets, 68 percent along collector streets, and 72 percent along local streets.

In the early days of urban development, when densities were relatively low, motorists could often park their automobiles on streets near their destinations. As densities have increased, however, curb spaces have become inadequate and parking itself has become a major urban land use. The cost of remaining on-street parking is high in terms of traffic congestion and accidents.

Traffic operations are now commonly evaluated as described by the 1965 Highway Capacity Manual (HCM) (1), which recognizes that curb parking has a significant effect on the capacity and service volumes of highways. The safety aspects of parking practices, however, have not been given equal attention in traffic engineering literature. No widely accepted relations have been identified among parking configurations (diagonal, flat angle, parallel, etc.), parking density, traffic flow, pedestrian activity, and highway safety. The need for such definitions, however, is emphasized by the large number of accidents involving curb parking. One source (2) has estimated that about 20 percent of all urban accidents are related to curb parking. Five primary causes were identified:

1. Vehicles parked in the roadway present obstacles and serve to narrow the usable width of the roadway and to restrict the flow of traffic. Such parking also restricts right-turn movements into and out of side streets, driveways, and alleys. Furthermore, parked vehicles may be struck, or their presence may cause sideswipe or rear-end accidents.
2. Vehicles leaving the parked position disrupt the traffic flow and, by increasing congestion, lead to rear-end and sideswipe collisions.
3. Vehicles entering the parked position frequently require automobiles approaching in the lane adjacent to the parking lane to slow or stop. Parking maneuvers are especially hazardous because they usually involve a backing-and-turning movement. Rear-end and sideswipe collisions can readily result from this maneuver.
4. Drivers or back-seat passengers getting out of parked vehicles on the street side present an added obstacle in the roadway. Not only are the door and the alighting passengers in danger of being struck, but passing traffic may have to swerve or stop suddenly. This causes both rear-end and sideswipe collisions.
5. The sight distance of pedestrians—many of them children—attempting to cross the roadway from between parked vehicles is reduced, and the motorist may not see such pedestrians in time to avoid collision. A danger from impaired view also exists when vehicles are parked close to intersections and driveways. Depending on street grades and speeds, curb parking can create a hazardous sight obstruction if allowed on a major route within even a hundred meters of an egress point.

HCM and other traffic engineering manuals state that parallel parking is the preferred arrangement for any on-street parking adjacent to traveled lanes. The angle-parking alternative has usually been considered undesirable from a safety and capacity standpoint.

The belief that safety and capacity are compromised in the presence of diagonal parking is based on studies from the late 1940s through 1960s and, to a larger degree, on intuitive judgment. However, many early studies of diagonal parking were limited in scope. In particular, Main Street, U.S.A., was almost universally the type of street where diagonal parking was developed and evaluated, but use on local urban residential streets presents a different situation than use on business streets.

An urban pedestrian accident countermeasures study (3) concluded that under certain conditions pedestrian behavior could be favorably modified if parallel parking were replaced by diagonal parking. Crossing pedestrians were reported to have significantly increased their scanning of oncoming traffic at locations where diagonal parking replaced parallel parking. Other favorable behavioral changes were also identified.

Diagonal parking spaces also have the benefit of allowing occupants to enter and exit from the vehicle from either side without entering the traveled way. This and other findings relative to parking arrangements are documented in a Texas study by Zeigler (4), who concluded that flat-angle (22.5°) parking did not affect safety or capacity of the travel lane more adversely than parallel parking.

It should be recognized that, although parallel parking is generally accepted as preferable to the angle layout, there are certain operational disadvantages to this arrangement. The parallel-parking maneuver requires a considerable amount of time and therefore disrupts the flow of traffic. Many drivers are not skillful in the backing maneuver and need to make many tries.

Cities do not have funds to provide adequate off-street parking and to eliminate all curb parking. On the other hand, much curb parking should probably be eliminated because of delay to through traffic and hazards to both pedestrians and vehicles. In addition to roadways in the central business district (CBD), and other major roads, critical areas for parking studies include congested industrial and residential areas.

This study was undertaken because of the lack of widely accepted documented data relating to the safety aspects of on-street vehicle parking, the current involvement of the Federal Highway Administration (FHWA) in reviewing traffic operations on routes other than federally aided primary and secondary highways, and a general need for an evaluation of curb-parking arrangements. Two objects of this study were to determine the safety and operational characteristics of alternatives to curb parking and to develop an unbiased data base on these safety and operational characteristics that would allow comparative analyses of types of parking, operational characteristics, and accident types. Two tasks were included in the study: an on-street parking literature review and accident data collection and analyses.

CURB-PARKING LITERATURE REVIEW

During this study phase, several hundred research reports and technical articles were identified, sixty-five of which included information of specific value. These were abstracted. The following is a summary of some major findings from the literature review.

Overall Parking Accidents

Early data from a sample of 10 large cities revealed that curb-parking accidents represent 5-28 percent of total accidents (5). Later data for one of the largest cities found moving vehicles striking parked vehicles to cause 2 percent of all fatal, 6 percent of all injury, and 26 percent of all property-damage-only accidents in the city (6). A smaller community identified 43 percent of all local- and collector-street accidents to involve curb parking (7). In the same city [see the table below (1 km = 0.62 mile)], frequencies of 8.7 parking accidents/km (14.0/mile) were found on major streets, but only 1.1/km (1.8/mile) on local and collector streets (8).

Street Classification	Average Daily Traffic	Average Trip Length
Major	>5000	1.6 km or more
Collector	1000-4000	0.8 km or less
Local	<1000	Very short

The overall picture of curb-parking accidents, as related in the literature, is grim. This type of collision generally represents about 20-25 percent of urban nonfreeway accidents. A significant proportion of these produce injuries. Furthermore, a distinct probability exists that many accidents related to curb parking are not reported as such, because a parked vehicle was not actually contacted (even though it posed a sight restriction).

Angle-Parking Accidents

Studies in nine Utah cities showed that changes from angle to parallel layout were accompanied by a reduction in parking accidents of 57 percent and a 31 percent overall decrease in injury or fatal accidents for the study section (9). A similar study of two business blocks in Salem, Oregon, revealed a 65 percent reduction in parking accidents.

Analysis of accidents in the Abilene, Texas, CBD in the mid-1970s showed average annual accident frequencies of 3.4/street-km (5.4/street-mile) for angle parking versus 0.9/street-km (1.4/street-mile) for parallel parking. When expressed on a rate basis, the angle-parking streets had 176 accidents/million vehicle-km (MVKM) [284 accidents/million vehicle miles (MVM)], compared to 73/MVKM (116/MVM) for parallel parking.

Ten reports on angle parking that represented many times that number of studies were nearly unanimous in finding extremely high frequencies of accidents compared with parallel parking. However, adequate data were not identified to distinguish any differences in accident frequency or rate as a function of varying angles from the curb.

Flat-Angle Parking

A 1971 report challenged the conclusions of many previous studies of angle parking and the assumption that safety and delay characteristics apply equally to all angle-parking arrangements (4). The arrangement tested differs from most angle parking in that the spaces were laid out at an angle of 22.5° to the curb line, as opposed to the more conventional angle. This layout has been called flat-angle parking. The reported operating experience with this parking layout indicated that it offered advantages over typical angle parking and parallel parking. The following conclusions were reported:

1. Flat-angle parking does not adversely affect the safety or capacity of travel lanes when compared with the generally accepted arrangement of parallel parking. This is true, provided that adequate widths for travel lanes are available.
2. Flat-angle parking results in improved safety for pedestrians entering or leaving parked vehicles.
3. Flat-angle parking results in less disruption of traffic flow than does parallel parking.

Based on the generally favorable results of their limited testing of flat-angle parking in Huntsville, the Texas Highway Department in February 1972 submitted a recommendation that a more extensive evaluation of the advantages and disadvantages of flat-angle versus parallel parking be undertaken.

Curb-Parking Policies

The literature indicates that, while evidence concerning the problems created by curb parking has been accumulating, most cities have been taking steps to encourage or force the development of off-street parking facilities through zoning controls. This will take many decades. In the interim, curb parking will continue to exist in commercial and industrial areas, along parts of major street systems, and on practically all local residential streets.

The curb parking policies of the cities, states, and the federal government, as identified in the literature search, may be summarized as follows:

1. High hazard associated with curb parking in general, and especially with angle parking, is understood;
2. The congestion effect of curb parking is of concern;
3. Positive steps are being taken to reduce future curb-parking demand by enactment and enforcement of zoning controls that require off-street parking supply for new developments,
4. Extensive use of rush-hour or total parking prohibitions is being made;
5. Permission for new angle-parking installations is to be refused; and
6. Limited programs that eliminate existing angle parking or convert to parallel parking have been set up.

STREET- AND ACCIDENT-DATA COLLECTION

A second phase of this project was to determine the magnitude and characteristics of accidents occurring on urban streets and to relate these to varying parking configurations, land uses, street widths, and street classifications. Street and accident data were gathered from more than 270 km (170 miles) of urban streets. A summary of collected data follows.

Field Selection Criteria

Cities were identified for study on the basis of the availability of location-specific accident files, study potential (streets of varying widths, land uses, and parking angles), and range of city size. Regions of the country were selected to represent different climatic conditions.

Study sites were chosen in five states and data were collected from 10 cities: Miami, Coral Gables, West Palm Beach, and Clearwater, Florida; Abilene and Wichita Falls, Texas; Tempe, Arizona; Naperville and Skokie, Illinois; and Jackson, Mississippi.

Within each city, specific streets were identified by driving surveys. General development densities of various land uses were noted, as were parking and curb types. Only paved streets were used, and nearly all of them had curbs. Wherever possible, streets with vertical face curbs 10-18 cm (4-7 in) high were selected.

Study streets had generally consistent land use along each side, but some mixtures of different uses on each side were included. Local residential streets absorbing spillover parking from nearby commercial areas were largely avoided. Streets were not studied if changes in surfacing, pavement, or land use were known to have occurred during the study period.

In the residential areas, a selection of property values was attempted; that is, we investigated both those areas with older homes and those in the higher-value subdivisions. A sensible mixture of straight and curvilinear local streets was selected in each area, and the greatest possible range of local street widths was sampled.

Mixtures of cross streets (short blocks) and long streets that had primary home frontage were selected. Also included were locations where each home had a front driveway rather than an alley garage.

Areas ranged from those having practically no curb parking to those having very dense curb parking.

Major routes were selected in terms of varying widths, parking characteristics, land use, and traffic volumes. Both one- and two-way streets were included as were those with and without barrier medians or two-way left-turn lanes.

A fair distance along major routes with curb-parking prohibitions was selected in order to allow for an assessment of the types of accidents and rates typically observed in the absence of curb parking.

Wherever angle parking was available and proper control conditions existed, studies were made of the various angles to the maximum extent possible.

Coding and Field Measurement

Study sections, or blocks, were composed of segments ranging from a single short city block to as many as a dozen continuous blocks of consistent land use, street width, functional classification, traffic volumes, and other characteristics.

The street width in each section was measured, and, for most, the number of legal parking spaces on each side was counted by a driving survey. Allowance was made for clearance from driveways, stop signs, and fire hydrants, in accordance with local practice in each city and with prevailing car sizes. These data, plus information on land use, parking regulations, one- or two-way traffic flow, and median type, were recorded on field sheets.

The section lengths were taken from city maps or plat books. Traffic-volume data were secured where available and averaged as needed to apply to the midyear of the accident study period. In all cases, land uses were taken as a surrogate value for pedestrian traffic counts; i.e., retail, commercial, apartment, and single-family residential uses represent descending magnitudes of pedestrian volume.

Where curb-parking stalls were painted, the pertinent dimensions were measured. At a few locations where no curbing existed but the shoulder areas were paved for direct pull-off, the typical distance from the bumper line of parked cars to the edge of the traveled way was measured. Such parking exists in many areas, but it usually is so irregular and setback variations occur so often—every 50 m or so (100-200 ft)—that no analysis of accident patterns would be meaningful.

Parking Checks

The number of curb-parked automobiles was counted during three time periods on typical weekdays. The midmorning check was taken between 9:00 and 11:30 a.m., the afternoon check between 1:00 and 4:00 p.m., and the night check between midnight and 6:00 a.m.

By using an analysis from previous but unpublished research by Paul C. Box, it was possible to calculate a multiplier factor for residential areas and to develop and estimate the annual space hours of curb parking for each section. The number of vehicles parked in each of the three study periods was summed for each section and multiplied by 9.4 to arrive at the estimated number of daily space hours. This figure was then multiplied by 360 to provide an estimate of the annual space hours.

rations. This is approximately three times the number of study locations, and many were not found.

Among the additional factors originally considered for use in the analysis were average daily traffic (ADT), width of street, driving width, one-way or two-way flow, and length of study (in kilometer years) for each location. Except for traffic volume, there were no discernible relations between any of these factors and the accident rate, and thus they were excluded from the analysis.

Response Variables

The response variable originally used with major and collector streets was accidents per million vehicle kilometers (acc/MVKM) [per million miles (acc/MVM)]. Lacking traffic counts for local streets, we chose the response variable originally used for these streets, accidents per kilometer per year (acc/KMY) [per mile per year (acc/MY)].

However, it should be noted that all data collection and analysis were done with customary units of measure. The conversion to metric units was made for purposes of this publication. Any effort to perform additional analysis of these data will require that all data presented be reconverted to customary units before analysis is begun. Since the transformation used in the analysis was made prior to the conversion to metric units, all units in this section on response variables and the included transformation equations are shown in customary units for the sake of clarity.

There were two anomalies in these response variables that must be noted. The first, present in both acc/MVKM and acc/KMY and characteristic of accident data in general, is a proportionality between the mean and the variance of accident-rate sets. This means that, if there are two groups (such as A and B) for which the accident rate was measured, and A had a higher average accident rate than B, then A also had a greater variation among the individual location accident rates than B. This anomaly traditionally requires a transformation on the response variable. The description of the transformation and the rationale behind it follow.

The analysis of variance (ANOVA) technique is based on comparing variances computed in different ways. In an ANOVA, the data are grouped into cells and the variance of the data is computed within each cell; then the variance of the data is computed between the cells by using the cell means. The variance based on the cell mean is then compared with the average of the variances within each cell. If there is a significant difference between the cell means, then these two ways of calculating the variance will yield different values. Because of this comparison procedure, a key assumption is that the variability of the responses within each cell is essentially the same for all cells.

This translates into the requirement that the variation

Table 1. Street kilometers by functional classification and city.

Area	Street Kilometers: Major	Collector	Local	Total
Florida				
Miami and Coral Gables	15.67	0.00	17.31	32.98
West Palm Beach	1.08	5.52	10.42	17.02
Clearwater	10.85	2.67	19.13	32.65
Texas				
Abilene	10.19	3.85	37.22	51.26
Wichita Falls	6.31	1.21	13.27	20.79
Arizona				
Tempe	0.00	15.15	9.64	24.79
Illinois				
Naperville	2.46	0.00	11.51	13.97
Skokie	13.35	0.00	37.51	49.06
Mississippi				
Jackson	9.45	2.67	19.80	31.92
Total	69.36	31.07	174.01	274.44

Note: 1 km = 0.62 mile.

Table 2. Street kilometers by land use and functional classification.

Land Use	Street Kilometers: Major	Collector	Local	Total	Proportion (%)
Retail only	18.86	0.00	0.47	19.33	7
Retail mixed with office, motel, or industrial	5.54	0.34	0.27	6.15	2
Office only	6.39	1.19	1.37	8.95	3
Single-family residential only	18.19	20.59	149.94	188.72	69
Apartment	4.28	2.20	12.88	19.36	7
Apartment mixed with single-family residential	2.88	1.05	4.19	8.12	3
All other	13.22	5.70	4.89	23.81	9
Total	69.36	31.07	174.01	274.44	100

Note: 1 km = 0.62 mile.

Table 3. All midblock and parking-involved accidents by severity, street classification, and parking involvement.

Street Class	Severity: Property Damage Only: Other	Parking Involved	Sub-Total	Injury[a]: Other	Parking Involved	Sub-Total	Combined: Other	Parking Involved	Total	Proportion Parking Involved (%)
Local	133	396	529	37	37	74	170	433	603	72
Collector	60	150	210	15	8	23	75	158	233	68
Major	1094	1229	2323	323	112	435	1417	1341	2758	49
Total	1287	1775	3062	375	157	532	1662	1932	3594	54

[a]One recorded fatal accident on a major street in "other" category.

The annual space-hour estimate for retail areas was developed by multiplying each of the three parking check periods by 8 and summing. This figure in turn was multiplied by 310, making allowance for lower parking demand on Sundays and holidays and correcting for the longer night period represented by the night check, to give an estimate of annual space hours. For example, on some weekdays, retail stores in a given area may close at 6:00 p.m. and on other days remain open until 9:00 p.m. Thus, the length of night parking can last for nearly 12 h instead of 8 h. This is, of course, controlled by the degree to which recreation-oriented curb parking for theaters, bowling alleys, taverns, etc., exists.

Accident-Data Criteria and Coding

Two year-long periods were used as a basic minimum for data collection. However, in a few cases where before-and-after conditions were present, only one year was used for each time period.

The tabulation in all cases came directly from police reports, usually in location-specific files in the traffic engineer's office. In some cases, original police files were found useful. In order to examine the frequency of parking-related intersection accidents as well as midblock and nonparking intersection collisions, data were sampled for all accidents occurring in certain areas. This was done both in business and in local residential areas.

Information obtained from each accident report included the month and year, severity (property damage, injury, or fatality), location (intersection, midblock), type of accident (vehicle-vehicle, vehicle-parked car, etc.), and parking involvement (parking, unparking, opening door, etc.).

Summary of Street Data

Table 1 shows the kilometers of data collected, by city and type of street. The breakout of distances for one-way streets versus two-way streets, by functional classification, is given below (1 km = 0.62 mile).

Street Classification	No. of Street Kilometers One-Way	Two-Way	Total
Major	17.23	52.13	69.36
Collector	1.88	29.19	31.07
Local	3.20	170.80	174.01
Total	22.31	252.13	274.44

About one-fourth of the major streets selected were one-way. Only 6 percent of the collectors and 2 percent of the local streets we measured were one-way, in consideration of the lower distances traveled on such streets across the country.

The distances for each width, in 0.6-m (2-ft) increments, by functional classification, were also tabulated. Even though the scheduled collection of only 40 km (25 miles) of major street data was increased to 69 km (43 miles), it was still not possible to collect equal quantities of data for each of the numerous widths found in typical American cities. The problem was compounded by necessities for variable land use and parking regulations.

Table 2 shows street kilometers of data collected by land use as related to functional classification. Most sections selected had common land uses on both sides of the street. The 9 percent of distance related to all other uses refers principally to mixed land uses for which we found too small a sample to be analyzed.

Summary of Accident Data

A total of 4804 accidents were tabulated on the inventoried streets of the 10 cities during their respective study periods. Of these, 3594 were either midblock accidents or intersection accidents in which curb parking was considered to have been a factor. The remaining 1210 were intersection accidents not involving curb parking that occurred on selected streets in Miami, Coral Gables, Clearwater, and Abilene. These were streets in areas where all accident data were tabulated—midblock and intersection—in order to derive the ratio of parking-related accidents to total collisions.

Tabulations of these selected data were analyzed to see whether the accident breakdowns obtained in this study were similar to those reported in other research. The proportions of midblock versus intersection accidents by street classification were found to agree well with those of other studies.

After reviewing these initial data, we determined that additional intersection accident data would not be coded unless they were parking related. Thus, all accident data and analyses subsequently presented will include all midblock accidents plus only those intersection accidents considered to be parking related. As previously stated, a total of 3594 of these accidents were identified in the study.

Table 3 shows the tabulation of accidents for all streets (except intersection, non-parking-related collisions) by accident severity, street classification, and parking involvement.

ACCIDENT ANALYSIS

Description of Classification Variables

In order to make comparisons between different locations, the street condition was defined by each of the following factors: (a) street classification, (b) parking arrangement, (c) land use, and (d) parking use. The functional classifications major, collector, and local were also used. Data from these groups were analyzed separately.

Parking arrangements were grouped into six types: no parking, parallel parking, parallel parking with neutral zones (skips), 22.5°-angle parking, 30°-angle parking, and high-angle parking (combining both 45°- and 60°-angle parking). Different parking conditions could prevail on opposite sides of the street, so 15 combinations of these six conditions can be found in the data.

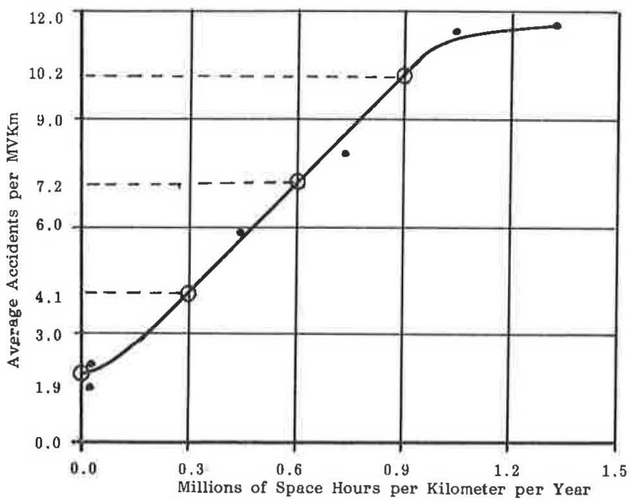
The land uses chosen explicitly for study were retail, office, single-family residential, apartments, motel, industrial, and school or park. Because of limited samples, the last three types were combined into a miscellaneous category. In all, there were 15 combinations of these five types.

Parking use was measured according to the annual space hours per kilometer for each study location. To allow for sampling error and to simplify the analysis, use-level values were assigned one of the four levels below (1 km = 0.62 mile).

Annual Space Hours Occupied per Kilometer (millions)	Parking-Use Level
0-0.6	1
0.6-3.1	2
3.1-6.2	3
>6.2	4

Combining the groups of streets, parking, land use, and parking use results in some 2700 potential configu-

Figure 3. Accident rates versus parking use on major streets for all land uses.



Bonferroni Procedure

The Bonferroni procedure is a modification of variance that allows specific comparisons, planned in advance, to be made. Because these comparisons may not be independent of each other, an adjustment is made in the effective significance level used for each comparison. Specifically, a sum of squares with one degree of freedom is calculated for each comparison. When divided by the mean square error term from the overall ANOVA, the resulting statistic follows an F-distribution under the null hypothesis. This statistic is then compared with the critical point from the appropriate F-distribution by using a significance level of alpha.

As these tests are carried out on the individual comparisons, the probability of a type 1 error increases. Moreover, the dependence of the various comparisons makes it impossible to calculate a true overall alpha level, although an upper bound on this level may be found by simply summing the alpha values of the individual tests.

Scheffé Procedure

The Scheffé procedure is a post hoc procedure that allows questions to be asked after the preliminary analysis of the data has been completed. In effect, any comparisons of cells or any comparison between different collections of cells may be made. For any one comparison a sum of squares is obtained as before, and this is compared with a critical value based on an F-distribution. The Scheffé procedure differs from the Bonferroni procedure in the way in which the critical value is calculated. This value is inflated to give a true overall significance level (equal to the specified value) when all possible comparisons are simultaneously considered. If only a few comparisons are to be made, this procedure is very conservative. However, it does have the advantage of allowing the ranking of cell means and of asking specific questions based on these ranked cell means.

The essential conservativeness of the Scheffé procedure means that the results obtained are good, but its sensitivity suffers as a consequence. Thus, it is used in addition to the more sensitive Bonferroni procedure.

Major Street Analysis

Parking Use

The factor showing the greatest effect on accident rate was the parking-use level. All pair-wise comparisons between the four levels of use showed differences significant at the 0.025 level.

The typical pattern of the relation between accident rate and use is summarized below for parallel parking in retail areas (1 km = 0.62 mile).

Parking Use (millions)	No. of Locations	Average Acc/MVKM	Average ADT
0.0	2	1.0	10 800
0.2	6	3.0	16 000
0.5	20	7.0	13 500
1.9	23	8.3	11 500

As use increases, the accident rate also increases. Figure 3 illustrates flattening of the curve at higher levels of use.

Land Use

In examining the effect of land use on accident rate, 16 comparisons were tested. These were chosen out of the 90 or so possible comparisons because of their ease of interpretation. Most of those not examined involved comparisons between nonhomogeneous or nonsimilar land uses or both. Of the comparisons tested, three were significant at the 0.05 level. These are described below.

1. Retail versus office: Sixty retail locations had an average accident rate of 7.3/MVKM (11.8/MVM), and 21 office locations had an average rate of 5.2/MVKM (8.4/MVM). Except for locations with no parking, or with low use, the accident rates for retail land use were always higher than for office use. This is to be expected, considering the higher parking activity associated with retail operations.
2. Retail versus apartment: Sixty-two retail locations had an average accident rate of 6.3/MVKM (10.1/MVM), and 19 apartment streets had an average rate of 3.3 MVKM (5.4/MVM). The results tend to match expectations; i.e., higher accident rates are associated with higher retail parking activity.
3. Miscellaneous versus apartment: Five locations with industrial, motel, or school or park land uses were grouped for comparison with 10 apartment land-use locations. The average accident rates were respectively 6.6 and 2.3 MVKM (10.7 and 3.7/MVM). The higher-activity uses again show higher accident rates.

Parking Arrangements

In examining the effect of parking type on accident rate, some contrasts were feasible. However, these effectively made only 14 basic comparisons, three of which had differences in accident rates significant at the 0.025 level. These are described below; however, in all cases there is an inconsistency in ADT that will be discussed later.

1. No parking versus parallel parking with skips: Data for 17 locations with no parking were compared with two streets with parallel parking and neutral zones, while land use and use level were held constant. The use level of 25 000 annual space hours per kilometer (40 000/mile) for the parallel parking locations was very low relative to other locations. The streets with no

Figure 1. Mean versus standard deviation for accident rates.

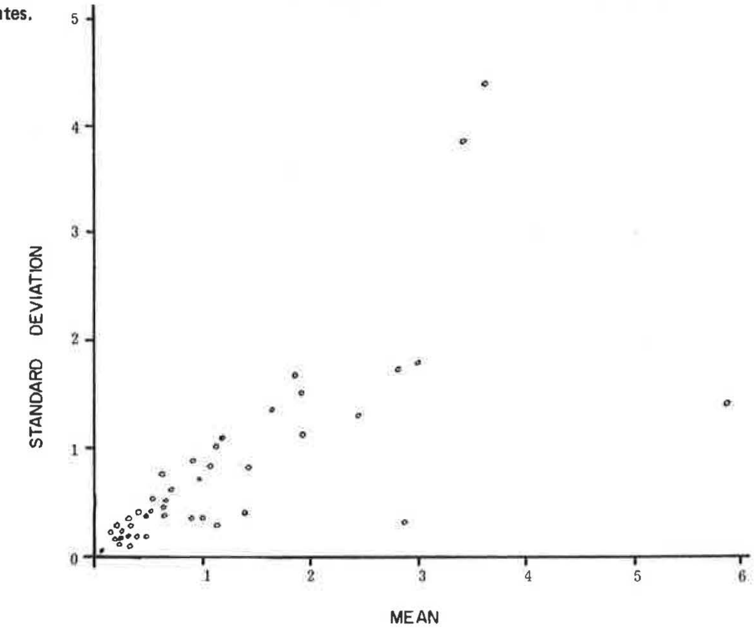
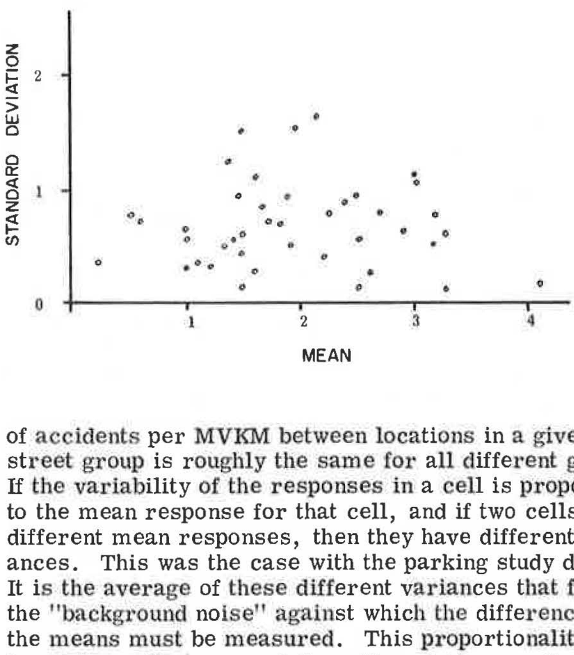


Figure 2. Mean versus standard deviation for transformed accident rate Y.



of accidents per MVKM between locations in a given street group is roughly the same for all different groups. If the variability of the responses in a cell is proportional to the mean response for that cell, and if two cells have different mean responses, then they have different variances. This was the case with the parking study data. It is the average of these different variances that forms the "background noise" against which the difference in the means must be measured. This proportionality between the cell means and the cell variation results in a drastic reduction in the sensitivity of the ANOVA procedure.

For the parking study data (and typically for most accident data) the variation in the accident rate was proportional to the average accident rate when similar locations were considered as a group. For those groupings with more than one location, the group mean is plotted against the group standard deviation, as in Figure 1. The proportionality is clearly seen in the wedge shape of the points in this graph. (If the variation were roughly constant, regardless of the mean, then this graph would show a horizontal band.)

The traditional solution to this problem is to make a nonlinear transformation of the original response variable. By this means, the skewness of the response variable can be minimized, thus stabilizing the variation within each cell while maintaining the intrinsic relation between the individual responses within each cell. For these data, the transformations used were Y = ln (acc/MVKM + 1) and Z = ln (acc/KMY + 1). Figure 2 shows the standard deviation versus the mean for the Y values for each of the groupings used in Figure 1. Here the desired horizontal band is present.

By analyzing the groups with Y or Z as the response, the variability of the response is effectively homogenized for the various groups. This homogeneous variation within each group produced an amount of background noise considerably lower than would have been produced by an analysis that used the untransformed accident rates. Because differences in groups have to be detected in the presence of this background noise, the reduction leads to a more sensitive analysis. For this reason, the responses used in the analysis were the transformed accident rates Y and Z.

The second anomaly, which occurred only with the acc/MVKM values, consisted of a shift in accident rates with low volumes. For locations of less than 5000 ADT, both the minimum and the maximum accident rates increased as ADT dropped. For those locations of more than 5000 ADT, both the minimum and the maximum accident rates were constant as ADT increased. This problem occurs in both the raw and the transformed rates, which suggests that acc/MVKM cannot be used across all ADT levels for comparisons between locations.

Analysis Procedure

After transforming the response variable, the data were separated according to street classification and each portion was analyzed.

First, those specific comparisons between levels of one factor that could be made while the other factors were held constant were identified. Then, by using ANOVA on different street configurations, these specific comparisons were examined for significance by using a Bonferroni procedure (11). After this, a Scheffé post hoc analysis was performed to look for general patterns of differences among the street configurations. A brief description of these procedures follows.

Table 4. Configurations ranked by acc/MVKM on collector streets.

Parking Type	Land Use	Parking Use (annual space-h/km 000 000s)	Number of Locations	Average Accidents/ MVKM	Average ADT
Group 1					
Parallel	School and park	0.0	1	0.0	3000
Parallel	School and single-family residential	0.11	2	0.0	3100
Parallel	Single-family residential	0.13	12	0.3	3025
No parking	Retail and apartment	0.00	1	0.8	4300
Parallel	Single-family residential	0.02	9	0.9	2050
Group 2					
Parallel	Single-family residential and apartment	0.14	1	3.0	1500
Parallel	Retail and office	0.14	1	3.9	2700
Parallel	Apartment	0.55	2	5.0	2800
22.5° angle	Office	0.02	1	5.0	1500
Group 3					
Parallel	Office and motel	0.30	1	7.9	2000
Parallel	Retail and apartment	0.71	1	9.3	1200
Parallel	Motel	0.40	1	11.7	2700
Parallel	Apartment	0.88	3	12.2	1170
Parallel with skips	Motel	0.37	1	15.0	2000
Parallel	Office	0.17	1	15.8	3100
Parallel	Office	0.71	1	18.6	1500
Parallel	Office and industry	0.81	1	23.6	1900
High angle	Apartment	1.38	5	25.5	1840
High angle	Retail and apartment	1.16	1	29.6	1600
Parallel and high angle	Office and school	0.61	1	44.8	1000
Parallel one side	Office	0.55	1	52.5	800
Parallel	Office	0.58	1	55.3	600
High angle	Industry	1.60	1	63.0	700

Note: 1 km = 0.62 mile.

were significant at the 0.05 level or better. Once again, as the number of annual space hours per kilometer increased, the accident rate (in accidents per kilometer per year) increased, as shown below (1 km = 0.62 mile).

Parking Use (millions)	No. of Locations	Average Acc/KMY
<0.06	96	0.4
0.06-0.31	217	0.6
0.31-0.62	81	1.3
>1.62	75	2.5

Land Use

There were 10 of these comparisons, of which 7 were not significant. The three comparisons that were significant, however, show single-family residential land use to be safer than retail, apartment, or single-family and apartment land uses, as described below.

1. Retail versus single family: Three locations with retail land use were matched against 306 locations with single-family land use. The retail locations showed an average of 3.5 acc/KMY (5.7 acc/MY), while the single-family locations showed an average of 0.7 acc/KMY (1.1 acc/MY).
2. Single family versus single family and apartment: The 306 single-family locations had an average of 0.7 acc/KMY (1.1 acc/MY), while the 17 locations with a mixture of single-family and apartment land uses had an average of 2.3 acc/KMY (3.7 acc/MY).
3. Single family versus apartment: The 311 locations with single-family residential land use showed an average of 0.7 acc/KMY (1.1 acc/MY) versus 2.9 acc/KMY (4.7 acc/MY) for 54 apartment land-use locations.

Parking Types

The three local-street comparisons made for parking type were (a) parallel one side versus parallel both sides, (b) parallel versus parallel and high angle combined, and (c) parallel versus high angle. None were found to be significant.

The local-street configurations were then ranked according to their acc/KMY and compared by means of a Scheffé post hoc procedure. They could be divided into two groups, which were found to be significantly different at the 0.05 level. The safer group includes all single-family residential land uses, as well as one mixed single-family residential and apartment and some apartment land uses. These latter categories typically had use levels below 300 000 annual space hours per kilometer (500 000/mile). The more dangerous group included retail, office, and apartment land uses, almost all of which had uses above 300 000 annual space hours per kilometer. The general pattern of variation in accident rate with the changes in use and land use is shown in Table 5.

General Results of Analysis

The results suggest the following.

1. Parking use level is a significant factor for all street categories;
   a. No parking is clearly the safest.
   b. For up to approximately 900 000 space hours/KMY (1.5 million/MY), increases in use result in increases in accident rate.
   c. For use beyond that, the accident rate was not found to increase.
   d. The prohibition of curb parking along major streets, where the existing use is about 300 000 space hours/KMY (500 000/MY), could be expected to reduce midblock accident rates by up to 19 percent.
   e. Prohibitions on major streets with use of about 600 000 space hours/KMY (1 000 000/MY) or more could be expected to reduce midblock accident rates by up to 75 percent.

parking had an accident rate of 2.1/MVKM (3.4/MVM), while the ones with parallel parking and neutral zones had an average rate of 8.9/MVKM (14.3/MVM).

2. Parallel parking versus 22.5°-angle parking: The groupings used in this comparison involved 38 locations with parallel parking and 28 locations with 22.5°-angle parking. The locations with parallel parking had an accident rate of 6.6 versus 10.7/MVKM (10.7 versus 17.2/MVM) for streets with 22.5°-angle parking.

3. Angle parking of 22.5° versus 30°: Holding land use and use levels constant provided 22 locations with 22.5°-angle parking and five locations with 30°-angle parking. The locations with 22.5°-angle parking had an average accident rate of 11.7 versus 2.0/MVKM (18.9 versus 3.3/MVM) for the 30°-angle parking. This finding is very surprising.

One hindrance to a straightforward interpretation of these results is that, in each case, the parking type with the higher accident rates has an ADT of 5000 or less. Thus, these differences in parking types are confounded with differences in ADT levels.

To more fully appreciate the ambiguity caused by the low ADT values, the comparison between parallel and high-angle parking may be considered. This comparison involved 51 and 10 locations of average accident rates of 6.2 and 4.7 acc/MVKM (10.0 and 7.6 acc/MVM), respectively. This difference is not significant, and there were no ADT values below 5000. Thus, those comparisons that might have been expected to be significant were not, while those comparisons that might have reasonably been expected not to be significant (such as 22.5°- versus 30°-angle parking) were found to be significant. Moreover, if a significance level of 0.10 is used, then all of the Bonferroni comparisons for parking type that involved ADT less than 5000 would have been significant, while all of those above 5000 would have been insignificant.

The simplest explanation of these results for parking type is that these data do not support the concept that any differences are due to parking type, but rather that those comparisons found to be significant are all attributable to differences in ADT.

The Scheffé analysis only added one detail to the above results. While the general pattern of increasing accident rates that coincided with increasing use levels was again apparent, this relationship did not continue for the higher use levels; for use above 1.5 million, the accident rate was essentially consistent.

Accident Rates for Combinations of Land Uses

In the examination of traffic safety and operations as related to street improvements, the local public agency has only limited control of land use. In most cases, the uses already exist, as does the curb-parking demand, which is a product of inadequate off-street supply. Therefore, it is appropriate to consider which reductions in accident rates might be achieved by a policy of developing additional off-street parking and removing curb parking as part of a general street-improvement program.

Improvements are usually made on major streets. Therefore, acc/MVKM has been combined by land use, as a function of curb-parking use, which is the dominant factor.

Figure 3 shows the combination of all land uses and represents the potential average accident reduction. Note that these are actual rather than transformed rates. The four intercepts noted correspond to use levels of 0.0, 0.3, 0.6, and 0.9 million space hours per kilometer per year. The average effect of prohibiting parking, where existing demand is at these levels, may be directly calculated from the graph. The reduction in the accident rate would amount to 54, 74, and 81 percent, respectively.

Accident rates are often calculated for a route by including intersection accidents. We and other researchers have found that about 40 percent of accidents occur at midblock, so the overall effect of curb-parking prohibition along a street should be a reduction in the rate of approximately 8 percent for 0.3 million use, 29 percent for 0.6 million, and 32 percent for 0.9 million.

Collector-Street Analysis

The number of Bonferroni comparisons that could be made for the collector streets was very small because of the limited number of groupings. Of the parking-type, land-use, and use comparisons available, the only significant difference was found between office and single-family residential land use. The one office location had an accident rate of 15.9/MVKM (25.6/MVM), while the 12 residential locations had an average rate of 0.3/MVKM (0.5/MVM). All locations involved had parallel parking and uses in the range of 0.06-0.31 million annual space hours per kilometer (0.1-0.5 million space-h/mile).

The Scheffé analysis for collector streets divided the 23 different configurations into three groups; the first and last differed significantly at the 0.05 level. The middle group was indistinguishable from either the first or the last group. Table 4 lists the configurations in order of increasing accident rate and identifies the groups.

1. Group 1 contained 25 study locations in 5 different configurations and had average accident rates from 0.0 to 0.9 acc/MVKM (0.0 to 1.5 acc/MVM).

2. Group 2 contained 5 study locations in 4 different configurations and had accident rates from 3.0 to 5.0/MVKM (4.8 to 8.0/MVM). These configurations were indistinguishable from those in group 1 or 3.

3. Group 3 contained 20 study locations in 14 different configurations and had accident rates from 7.9 to 63.0/MVKM (12.7 to 101.5/MVM). Eighteen of these locations had used levels of more than 300 000 annual space hours per kilometer (500 000/mile).

Thus, based on the Scheffé analysis, it can be seen that collector streets with single-family residential land use and low use levels of on-street parking are significantly safer than those with moderate or high use and non-single-family residential land uses. Furthermore, for non-single-family residential land uses, accident rates were somewhat inversely proportional to ADT. Single-family residential data did not show this same dependence on ADT, however.

Local-Street Analysis

Because of a lack of ADT data, the response variable in the local-street analyses was initially taken to be accidents per kilometer per year. Again, to meet the assumptions of the ANOVA technique, a transformation was required. The transformation (customary) used was $Z = \ln(\text{acc/MY} + 1)$. By using Z, selected comparisons were examined by the Bonferroni technique, and then a Scheffé post hoc analysis was carried out to discern overall patterns.

Parking Use

Of the six possible comparisons between use levels, five

Table 5. Variations in acc/MVKM with changes in parking and land use.

Land Use	Parking Use (annual space-h/km 000 000s)	Number of Locations	Average Accidents/ MVKM
Retail	0.159	1	0.0
Retail	0.358	1	8.18
Retail	0.978	1	12.99
Apartment	0.219	3	0.94
Apartment	0.469	9	1.32
Apartment	0.902	38	3.91
Apartment and single-family residential	0.121	3	1.02
Apartment and single-family residential	0.472	6	4.54
Apartment and single-family residential	0.884	8	1.66
Single-family residential	0.158	211	0.56
Single-family residential	0.456	66	1.01
Single-family residential	0.807	29	1.52

Note: 1 km = 0.62 mile.

   f. Because midblock accidents were found to typically represent 40 percent of total (intersection plus midblock) collisions, the overall accident rate reduction could be up to 8 percent for the 300 000 space hours/KMY (500 000/MY) use level and up to 30 percent for the 600 000 space hours/KMY (1 000 000/MY) level.
2. For all streets, an increasing accident rate was generally associated with changes
   a. From single-family residential to apartment land use,
   b. From apartment to office land use, and
   c. From office to retail land use; and
   d. Since the above changes suggest increases in parking turnover rates and pedestrian activity, it seems appropriate that increases in these variables would be accompanied by increasing accident rates (i.e., the variables may be considered surrogates for increased turnover).
3. Parking configurations were not found to have any effect on accident rate when use, land use, and type of street were taken into account. The data suggest that any kind of on-street parking is unsafe. The level of use rather than the parking configuration appears to be the key to the midblock accident rate.
4. For parking uses beyond 600 000 space hours/KMY (1 000 000/MY) angle parking is no more hazardous than parallel parking, given similar land uses.

CONCLUSIONS

The research was intended to examine relationships among parking configurations (angle, parallel, or no parking), parking density, traffic flow, street width, pedestrian activity, and road safety.

The variables reported in this paper to be associated with accident rates include (a) functional classification of streets, (b) use of parking, and (c) abutting land use. Of major interest, and most surprising, is the fact that parking configuration did not emerge as a variable that in itself was related to the accident rate.

Increased parking use, i.e., space hours per kilometer per year, was found to result in significantly higher accident rates, up to approximately 900 000/KMY (1 500 000/MY). Streets abutting land uses that generate high parking turnovers and pedestrian activity (land use has been used as a surrogate for pedestrian volumes) have higher accident rates compared with lower-intensity uses. Heavily used parallel parking was found to produce accident rates comparable to heavily used high-angle parking, while a prohibition of parking resulted in the lowest accident rates measured. Parking-related midblock accidents accounted for 49 percent of all accidents along major streets, 68 percent along collector streets, and 72 percent along local streets.

The findings on parking use suggest that future studies of accidents related to parking configuration should include measurement of use. Moreover, studies of the effect of a change of parking in one block should include similar studies simultaneously made for nearby blocks. If a parking prohibition or a reduction of spaces caused by change from angle to parallel results in a higher use in adjacent blocks, accidents on such streets might increase. Thus, the overall impact of a change should be assessed and not just limited to the specific study site.

As a final note, future researchers using accident rate data should be aware of the possible need for a transformation of those data. Careful attention should be given to the statistical inferences underlying any analyses and to the proper techniques to be used in those analyses.

ACKNOWLEDGMENT

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Delay, Time Saved, and Travel Time Information for Freeway Traffic Management

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Five studies of freeway motorists' opinions were conducted to determine preferences and reported behavior with respect to hypothetical displayed messages about time delays. Major findings were that the average person stated that he or she would divert from a freeway if the delay duration displayed was 15-20 min and would divert to a bypass route if the time saved displayed was 5-10 min. Incident type, traffic condition, and regional differences in the driver samples were not important factors. The message MAJOR ACCIDENT implied at least a 22-min delay, and MINOR ACCIDENT implied no more than a 12-min delay. The term "delay" was used in reference to unusual conditions at the time of day the message was displayed rather than time held up in traffic. Avoiding delay, saving time, and comparing travel time are all effective messages for describing advantages in taking a bypass route, but comparative travel time takes longer to read.

This study was one of several laboratory and field studies conducted for freeway traffic management to determine which human factors need to be considered for actual motorist information displays. The findings have been incorporated into a human-factors design guide (1).

Although the literature is not consistent on the relative importance of displayed time (temporal) information (2-5), several agencies are currently displaying delay, time saved, or travel time information on changeable message signs (6). A series of studies was undertaken by the Texas Transportation Institute to determine drivers' interpretations of and preferences for specific types of such information signs.

LEVELS OF DELAY TIME

The first study was concerned with particular lengths (levels) of delay time that motorists would consider significant in terms of making a diversion decision. Knowing what percentage of drivers would divert their routes according to various delay increments would be useful in predicting their behavior in traffic.

Because it was suspected that a motorist's previous knowledge of a particular freeway would be an influencing factor, the study was conducted in four widely separated locations to increase the general validity of the findings. It was also suspected that the traffic conditions and type of incident would be relevant variables. Thus the test material was designed to vary the circumstances under which the delay occurred.

Method

The sample consisted of 240 drivers from College Station, Texas; 184 drivers from St. Paul, Minnesota; and 40 drivers from Los Angeles, California.

The drivers were instructed to imagine themselves on a freeway and were given a picture of either light or very heavy traffic as the situation in which they were traveling. Each subject was presented seven cards, in random order, each of which contained two messages: first the type of incident, then the delay period. Subjects were divided into matched groups. Each group received only one type of incident and one traffic picture. The incidents were ACCIDENT, ROADWORK, TRUCK OVERTURNED, RAIN, and ICE. Each card displayed a different delay period: 5, 10, 15, 20, and 30 min and 1 and 2 h.

The experimental task was to check on an answer sheet one of two alternatives, "Yes, Stay on Freeway" or "No, Get off Freeway." The diversion decision was based presumably on a combination of delay period, incident type, and traffic condition factors.

Results

Figures 1 and 2 present the findings for the College Station sample only. The results indicate a similar pattern of yes or no responses to the delay periods regardless of type of incident or the traffic condition pictured. For all types of incidents, 50 percent of the drivers stated they would divert for a delay of between 15 and 20 min. Longer delays naturally resulted in proportionately more drivers expressing an intent to divert. However, RAIN and ICE did not result in complete diversion even up to an hour's delay.

Figure 2 indicates a slight but consistent tendency to divert at a lower level of delay in heavy traffic than in light traffic, but the effect of traffic condition was not statistically significant.

Figure 3 presents the data from St. Paul and Los Angeles along with the College Station data. Drivers in St. Paul were given cards with the same incident types, except that RAIN was deleted. Drivers in Los Angeles received only the ACCIDENT descriptor. The data points almost exactly coincide up to 60 percent diversion. Figure 4 presents a composite, best-estimate function for the effects of delay on a diversion decision. The 2-h delay data are now shown, but they were virtually identical to those for 1-h delay for each incident type.

LEVELS OF TIME SAVED

Time saved can also be used to present temporal information. This descriptor is applicable to a corridor or bypass route rather than to a freeway itself and is one of several ways of describing the benefits of diverting.

Method

This part of the study was conducted in Los Angeles with 127 drivers. The previous study had indicated that type of incident and traffic conditions had little effect on a diversion decision, so only three descriptors—ACCIDENT, ROADWORK, and TRUCK OVERTURNED—were employed. The traffic state depicted was heavy traffic only.

The three incident messages were assigned to independent groups. After the incident, the message displayed was USE TEMPORARY BYPASS TO THE ASTRODOME—SAVE X MINUTES. The time savings were the same periods employed in study 1. Again, messages were presented in random order and instructions were to indicate whether or not one would divert according to the message.

Results

The findings of the time-saved study are depicted in Figure 5. Type of incident again had little effect on the decision to divert, except for five drivers in the TRUCK OVERTURNED sample who refused to divert regardless of the time-saved duration. A savings of longer than 30 min resulted in a virtual asymptote in the numbers of people diverting. There was no difference in effect between a display of 30 min and one of 2 h on reported diversion. In the delay study, only 1- and 2-h delays were equal in effecting diversion decisions.

A major finding was that the average person in this study indicated that he or she would divert at between 5 and 10 min. Figure 6 presents a composite curve for time saved across incident types compared with the composite curve for the delay-time studies.

Before concluding that the time-savings message was the primary contributor to the difference, we should note again that a temporary bypass route was recommended in the time-saved study, whereas, in the delay studies, no alternate route was specified.

MAJOR AND MINOR ACCIDENT MESSAGES

Although studies 1 and 2 indicated that type of incident has no major effect on a diversion decision, it was suggested that the adjectives MAJOR and MINOR modifying word ACCIDENT might well imply different levels of severity and expected delay durations. The research

Figure 1. Effect of incident types and delay on percentage of driver diversion.

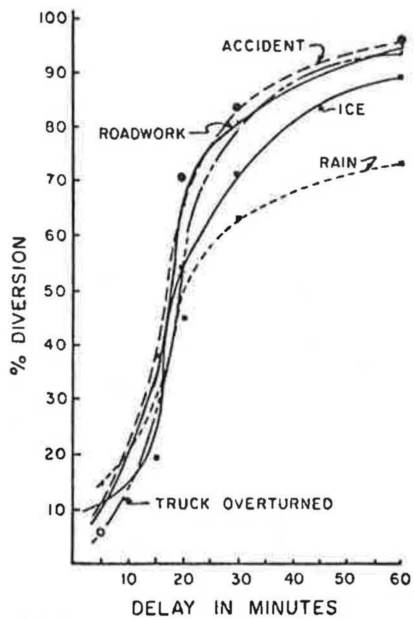


Figure 3. Regional differences in percentages of driver diversion.

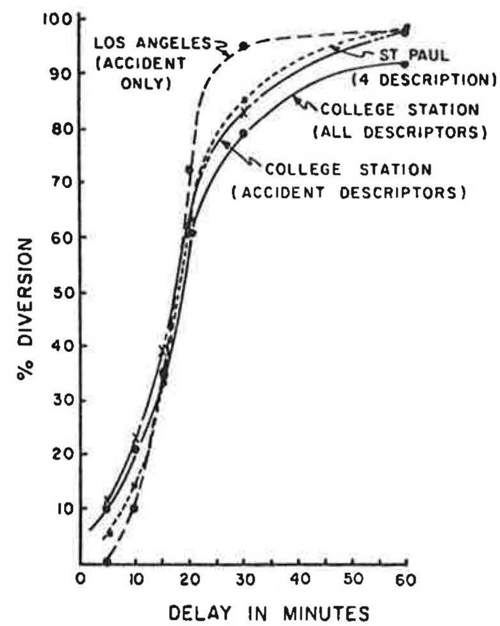


Figure 2. Effect of traffic conditions and delay on percentage of driver diversion.

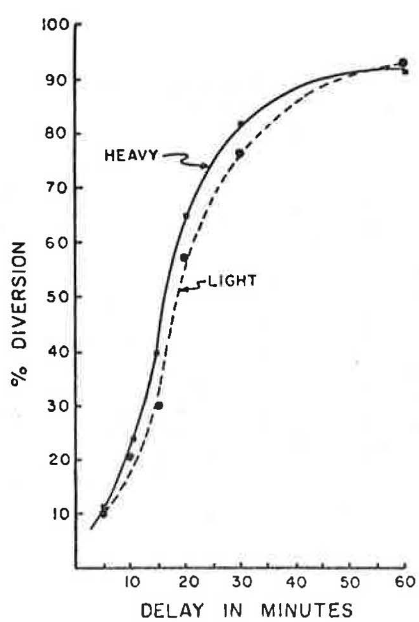
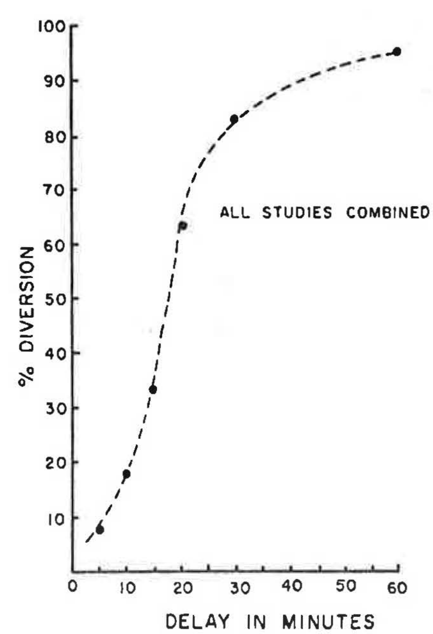


Figure 4. Effect of delay on percentage of driver diversion for all studies.



question related to the durations of delay implied by the messages.

Method

A small study was conducted in Dallas, where 40 drivers received the message MAJOR ACCIDENT and 20 drivers received MINOR ACCIDENT. Their instructions said that they were driving on a Dallas freeway when they saw the sign; they were then to indicate the delay they expected by checking one of the seven periods used in studies 1 and 2. The drivers given the message MAJOR ACCIDENT were to indicate the number of minutes or more they felt the message implied. The MINOR ACCIDENT receivers were instructed to report the number of minutes or less implied by the message. Thus, the values reported indicated slightly different meanings: minimum delay for a major accident and maximum delay for a minor accident.

Results

Figure 7 depicts the cumulative percentage of drivers who reported deciding to divert for various periods of anticipated delay and the respective incident messages. The average driver interpreted MINOR ACCIDENT as implying not more than a 12-min delay, whereas MAJOR ACCIDENT was taken to mean at least a 22-min delay. From study 1, the implications of these delays for a diversion decision may be extrapolated.

Figure 5. Effect of incident types and time saved on percentage of driver diversion.

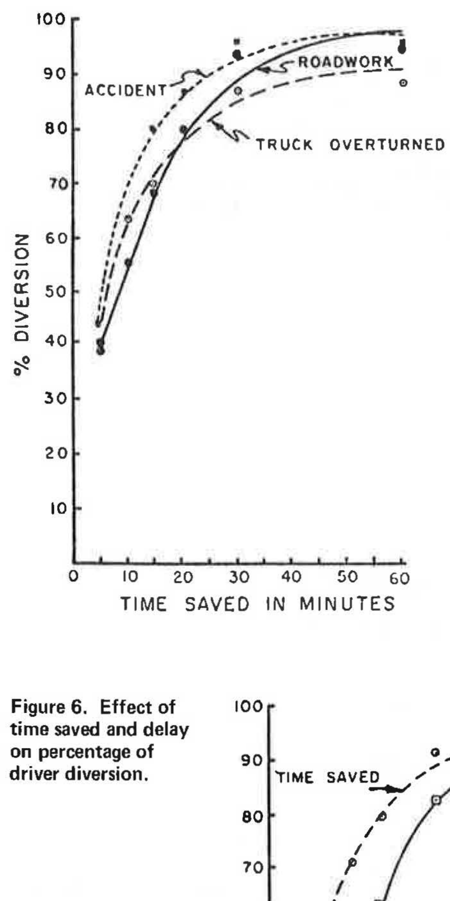
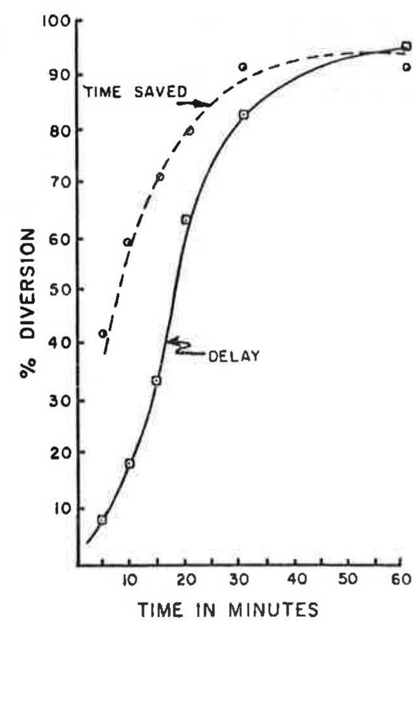


Figure 6. Effect of time saved and delay on percentage of driver diversion.



MEANING OF DELAY

The question has been raised about what specific meaning a given delay duration has for a driver in freeway traffic. For example, does it mean that the driver will be held up in traffic for the specified period or that he or she should expect to arrive at work that many minutes later than usual? What, specifically, was the driver's interpretation?

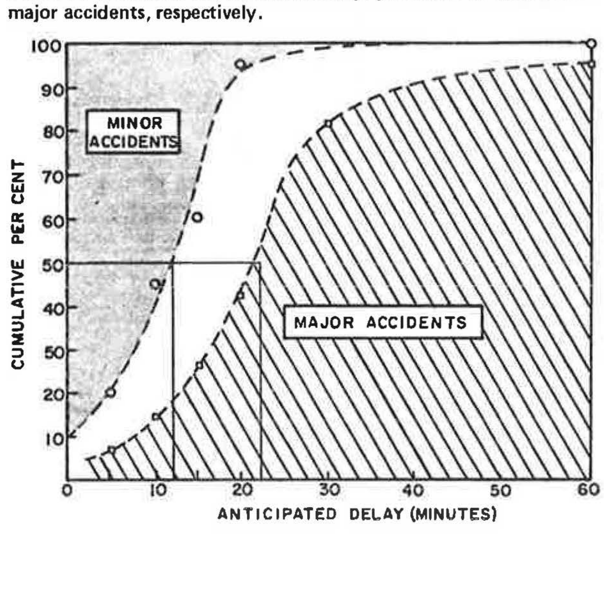
Method

A survey was conducted of 40 drivers in Los Angeles to determine which of five meanings of a 30-min delay message was most strongly conveyed. Drivers were assigned to two different random orders of the five interpretations. This procedure was undertaken to reduce the likelihood of bias from the order of statements in the questionnaire.

Drivers were instructed that they were approaching a freeway on their way to work and were told there had been an accident on the freeway and to expect a 30-min delay. Their task was to check on a five-point Likert scale their agreement with each of the five interpretations (i.e., strongly agree, agree, undecided, disagree, or strongly disagree). The five interpretations were

1. I will arrive at work 30 min later than usual;
2. I will travel for 30 min before the accident is removed;
3. I will travel for 30 min in bumper-to-bumper traffic;

Figure 7. Maximum and minimum delays perceived for minor and major accidents, respectively.



4. Travel time on the freeway will be 30 min longer than usual; and

5. I will be completely stopped in freeway traffic for 30 min.

A score of 1 was assigned to "strongly agree" with the statement, a score of 5 to "strongly disagree". Thus, a lower mean score means closer agreement with the statement. Drivers could choose the same degree of agreement with two or more statements. Identical scores would indicate ambiguity of meaning.

Results

The table below summarizes the ratings in terms of both total rating scores and average rating assigned by the 40 drivers. The mean ratings all ranged from "agreement" to "undecided".

	Rating	
Interpretation	Sum of Scores	Average
1	95	2.375
2	119	2.975
3	114	2.85
4	88	2.2
5	125	3.125
Total	541	2.7

The most popular interpretations were that freeway travel would be 30 min longer than usual (4) and that one would arrive at work 30 min later than usual (1). A test of significance indicated that differences between statements were statistically significant at the 0.05 level ($F_{4,\ 156} = 5.59$). In general, the study findings supported the view of delay as something relative to unusual conditions at that time of day rather than some absolute length of time during which one will be stopped or restrained in traffic.

MODES OF PRESENTING TEMPORAL INFORMATION

In addition to a statement of delay time, there are at least two other modes of expressing temporal information when an alternate route is also under traffic control and surveillance.

Study 5 was a preference study of the three modes of presenting temporal information:

1. Avoiding a 15-min delay by taking a bypass,
2. Saving 15 min (driving time) by taking a bypass, and
3. Saving 15 min or avoiding a 15-min delay as shown by travel times of 25 min on the Interstate and 10 min on the bypass.

Method

A survey of 70 drivers was conducted at a shopping mall in College Station. Drivers were told that they were traveling on I-94 in heavy congestion during rush hour. A lighted sign flashed them a congestion advisory and told them to get off and take a temporary bypass. The bypass rejoined the Interstate at a street beyond the congested area.

The drivers were told that this information would appear on the sign and that, in addition, the sign would show them the "advantage" of taking the bypass. Three different messages on three cards each gave a particular advantage of leaving the freeway. The drivers' task was to read each sign message carefully and to indicate which messages would be most and least likely to convince them to get off the freeway.

The first two parts of the three messages were the same. The message parts were CONGESTION AHEAD—USE TEMPORARY BYPASS TO WHITE BEAR AVENUE. The last part of the message displayed one of the three advantages of taking the bypass route.

Drivers were asked also to provide a reason for being or not being convinced to divert and were asked whether the three messages were communicating different messages or saying the same thing.

Results

The results of the study, in part, are presented below.

	Percentage of Drivers Agreeing	
Message	Message Most Likely to Convince	Message Least Likely to Convince
Avoid 15-min delay	38.6	17.0
Save 15 min	30.0	26.1
Travel time I-94: 25 min, bypass: 10 min	31.4	56.9

The percentage data indicate that the three messages were approximately equally effective in convincing the drivers to divert. However, 56.9 percent of the 65 respondents believed that the message giving comparative travel time would be the least likely to convince them.

The answers given to the open-ended question about reasons for being and not being convinced were extremely diverse. However, 23 of the 37 drivers who rated comparative travel time as least likely to induce diversion mentioned that the message took longer to read than the other messages. Sixty-two of the 68 respondents to the last question (88 percent) indicated that the three messages were saying essentially the same thing in a different way.

SUMMARY AND CONCLUSIONS

1. When delay information is presented along with a message about type of incident and level of congestion, knowing the duration of delay seemed to influence drivers more than other information in making a decision to divert. Three studies in different geographical regions indicate that the average subject will divert in response to a message advising of a 15- to 20-min delay.

2. There is some evidence to support the view that expressing information in terms of 5-10 min of time saved may result in diversion. However, this conclusion applies only when a temporary bypass route is also given in the advisory message.

3. Dallas drivers indicated that MINOR ACCIDENT meant a delay of 12 min or less, whereas MAJOR ACCIDENT meant a delay of 22 min or more.

4. A delay of x minutes was related to the driver's normal travel time (i.e., it normally meant that the travel time on the freeway would be that much longer than usual or that one would arrive at work that much later). Delay information did not necessarily imply stopped or bumper-to-bumper traffic of x minutes, nor did drivers think that the accident itself would necessarily be on the freeway for the indicated period.

5. Three modes of presenting temporal information (i.e., avoid x minutes' delay, save x minutes, and comparative travel time) were viewed as essentially synonymous and evoked no strong preferences. However, comparative travel time was disliked more often because the message took longer to read. Essentially, the driver must subtract one value from the other to obtain the benefits of taking an alternate route.

ACKNOWLEDGMENT

This study was conducted as part of the Federal Highway Administration contract for a project on human factors requirements for real-time motorist information displays. The contents of this paper reflect our views, and we are responsible for the facts and the accuracy of the data reported here. The contents do not necessarily reflect the official views or policy of the U.S. Department of Transportation.

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Empirical Analysis of the Interdependence of Parking Restrictions and Modal Use

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The relation between modal use and parking restrictions was analyzed by examining changes in travel behavior over time during a period of substantial change in parking restrictions, transit service, and transit fares. The situation examined was choice of travel modes to a major trip generator, the campus of the University of Wisconsin-Milwaukee. This area has major parking-congestion problems that have been partially alleviated by special transit services and remote parking lots. These systems have also been developed in conjunction with changes in parking restrictions. From an analysis of modal choices over time, it was found that shifts to transit use have occurred as a result of tighter parking restrictions and that shifts away from transit have occurred as a result of fare changes. Carpoolers seem to be most sensitive to changes, while the drive-along category showed less sensitivity. An analysis of respondents' reactions to probable future situations also indicated similar results.

As cities throughout the United States move toward the development and implementation of transportation system management (TSM) plans, an increasing amount of attention is being given to the relation between parking policy and transit use. Changes in parking policy, such as increasing its price, changing the schedule of rates, removing parking, and increasing parking restrictions, all are seen as potential means of increasing both transit ridership and the efficiency of the existing transportation system. It is felt that by making parking more difficult the relative advantage of the automobile will diminish and the attractiveness of transit as an alternative to it will increase. Given the potential of this strategy, it is surprising to find that the subject has received only limited study.

Mode-shift modeling has been an important part of the transportation planning process for some time, and several recent studies have reported on developing hybrid models to analyze the impact of changes in these variables. One study in particular (1) concludes that subjective preferences are useful for studying travel-mode diversion but that better means of controlling and monitoring changes in modal split through changes in policy-related variables are needed.

Several studies have dealt with short-term changes caused by such things as parking taxes and operator strikes (2, 3), while others have directly addressed the issue of the impact of changes in parking policy on transit use without empirical documentation (4, 5). Some of the literature provides an insight into efforts by our European colleagues to adjust the balance between automobile and transit use, but the applicability of their project conclusions to U.S. urban areas is questionable (6).

Therefore, a review of current literature seems to substantiate the claim that the United States does indeed need to better control and monitor mode-split changes. There has been little work on empirically relating changes in policy variables to mode choice. This current project was intended to help fill the gap and to provide transportation policymakers with a real-world understanding of the interrelationship.

PURPOSE

The purpose of this paper is to provide an analysis of the effects of changes in parking regulations on transit use. This will be done by looking at transit ridership trends and user attitudes in a particular situation over a period of time.

The situation that will be examined is ridership on the UBUS-UPARK, the transit service for the campus of the University of Wisconsin-Milwaukee (UWM). This service has developed over the past few years into a system of nine transit routes serving the campus from throughout the Milwaukee metropolitan area. Paralleling the development of the transit service have been significant changes in the level of parking restrictions in the campus area; the aim has been to make it difficult to commute to the campus by automobile.

This paper will look at (a) the changes in UBUS ridership during the time these restrictions have been implemented and (b) user behavior and attitudes toward these changes as they have occurred. This will be accomplished through the analysis of survey information of the entire market of travelers to the university area. This survey information has been developed to examine how individual travel patterns have changed in relation to changes in parking restrictions, transit service level, and transit prices.

BACKGROUND

Nature of Trip Generator

The UWM campus has many of the characteristics of a major trip generator: high trip-making activity, congested local streets, and severely limited parking supply. The campus is located on the east side of the city of Milwaukee approximately 5 km (3 miles) north of the Milwaukee central business district. The total enrollment of approximately 25 000 students and an additional 4000 faculty and staff yields a total university population of nearly 30 000. As such it is the second largest generator of trips in southeastern Wisconsin. On-campus housing accommodates only 1500 students (6.4 percent of the total), while the remaining students and staff commute to the campus.

The campus, extremely small for a university of this size, covers only 34 hectares (85 acres). Because of this small size, only 1900 parking spaces can be provided on campus for the 10 000 automobiles that are driven to the university each day. This leaves more than 8000 automobiles that must be parked on the surrounding streets.

Transit Service

In response to the severe parking problems in the university area, an extensive system of transit routes has been developed to provide an attractive alternative to the automobile. This service, called UBUS, (a) provides modified urban bus service to the university along bus routes that serve a large portion of the Milwaukee metropolitan area and (b) includes inducements to potential riders, such as direct no-transfer service, convenient schedules, minimum travel times, reduced fares, easily accessible off-street parking, convenient route locations, and a homogeneous rider group.

This service began with one route in September 1973 and carried about 2000 rides/day. It was expanded to a total of nine routes by September 1976 that carried about 6000 rides/day as shown in Table 1. Two types of service are offered: (a) UBUS service, which consists of long, radial transit routes that provide a direct link between many user origins and the campus and (b) shuttle services (UPARK), which directly connect remote parking lots and the campus. When the services came into operation they were free, but later they cost a small fare. Further details on the overall development of the UBUS program are available elsewhere (7-10).

As can be seen in Table 1, transit ridership on the UBUS-UPARK system has fluctuated during its period of operation. Ridership peaked at about 7000 rides/day in the fall of 1975, when UBUS routes operated at a 35-cent fare and shuttle service was provided free. After a fare was imposed in January 1976, shuttle ridership dropped by 1000 rides/day. As will be explained in the following section, these changes in transit service and fares were also accompanied by changes in parking restrictions in the university area.

Parking-Supply Changes

As transit service to the university has been expanded, there have been significant changes in the characteristics of the on-street parking supply surrounding the university. As shown in Figure 1, the number of unrestricted parking spaces has dropped from 2673 or 44 percent of the total available in 1972 to 899 spaces or only 15 percent of the spaces available in 1976. The major drop in unrestricted parking occurred in September 1975, when 1200 unrestricted spaces were changed to 1-h and 2-h parking zones. This was done at the same time that a remote parking lot for 800 cars with a free shuttle service was opened.

The only unrestricted parking that remains in the area is on scattered blocks or far from campus. Enforcement of parking restrictions is quite strict; cars must be moved every hour or two to avoid parking tickets. Furthermore, because of parking congestion, long walks to and from the parking place are necessary.

The net effect of these changes has been to make on-street parking increasingly more difficult both for commuters into the area and for local residents. The basic purpose of this analysis is to determine to what extent these changes in restrictions affected travel behavior in general and transit use in particular. The following sections will describe the analysis procedure and our results.

PROCEDURE

This project is the outcome of a survey distributed to a random sample of 10 percent of UWM's 25 000 students by mail in November 1976. Respondents were offered two options for returning the completed questionnaire: First, to return the completed survey in a postage-paid business reply envelope and, second, to receive a 25-cent cash incentive with the return of the survey to the UBUS ticket window in the student union. The incentive was almost the same as the postage rates for first-class business reply mail, 24 cents.

The survey was designed to obtain necessary personal data and data on value perception, modal preference, and changes in travel patterns from the respondents. Draft forms of the survey were administered to small sample groups for pretesting, and the final survey was printed on buff paper, which will yield a high initial rate of response.

In order to correct for bias in the mail survey, a telephone follow-up of nonrespondents was conducted. With this information, as well as that from previous surveys of the same travel market and ridership counts, it was possible to expand the data to provide a representative sample of the entire student population. Total response to the survey was 671 usable surveys, representing 2.7 percent of the total population.

The survey can be divided into three general categories of information. The first part of the survey provided personal data about the respondents with regard to geographic dispersion, class standing, enrollment history, family life, and employment. The second part dealt with travel behavior, mode choice, and relative effects of changes in transit service and parking availability. The last section posed hypothetical questions about future changes in the level of transit service and parking supply in an attempt to determine the effect of these changes on modal choice.

ANALYSIS

Student Background

The first portion of the analysis of data generated for this study presents an overview of responses to questions about personal background. The following is a summary of background information; a detailed analysis may be found elsewhere (11).

1. About 70 percent of all students enrolled at UWM are employed. More than half of them work more than 20 h/week and about 28 percent are employed full time.
2. The student population is geographically dispersed. The average home is more than 12 km (7 miles) from campus.
3. Most students travel to UWM more than four days a week.
4. About 65 percent of the student respondents attend daytime classes, 20 percent are exclusively night students, and the remainder have class schedules with both day and night hours.
5. Virtually all students (93 percent) possess a valid driver's license.
6. More than 73 percent of the respondents indicate that they have an automobile available on a regular basis for travel to the UWM campus.

A detailed analysis of the above information corresponds extremely well with demographic information obtained in previous studies undertaken by the Center for Urban Transportation Studies and other agencies.

Changes in Modal Choice Over Time

The survey was developed in such a way as to allow analysis of the different modes of travel used by the respondents in the fall of 1974, 1975, and 1976. The results of this analysis are shown in Figure 2 and indicate that the "drive alone" category increased somewhat in 1976 despite an increase in parking restrictions and a decrease in parking supply in the campus area.

Transit has shown a slight decline in overall use, and a substantial decrease (about 50 percent) appears

Table 1. Transit service to UWM, 1973-1976.

Type of Service	Sept. 1973	Sept. 1974	Feb. 1975	Sept. 1975	Jan. 1976	Sept. 1976
UBUS						
Number of routes	1	4	6	7	7	7
Route kilometers	17	76	113.5	138.5	138.5	138.5
Fare, cents	Free	25	35	35	35	35
Average ridership	2000	4763	4453	4744	4827	4399
UPARK Shuttle						
Number of routes	-	-	-	1	1	2
Route kilometers	-	-	-	4	4	10
Fare, cents	-	-	-	Free	15	15
Average ridership	-	-	-	2377	1338	1610

Figure 1. Changes in parking restrictions.

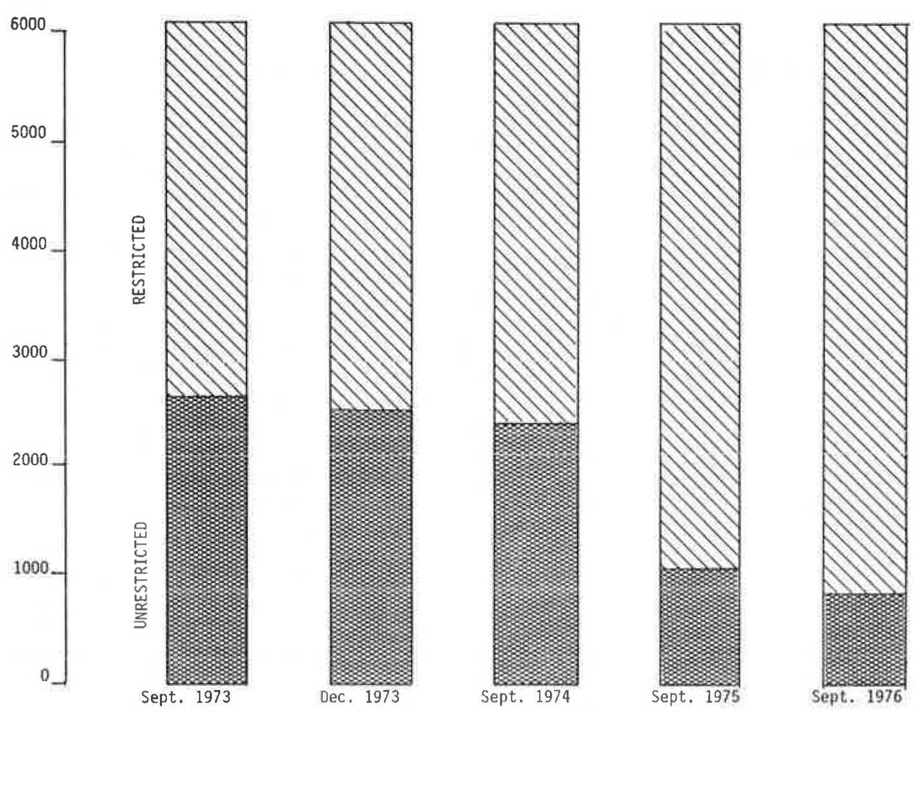


Figure 2. Most frequent travel mode to UWM as a function of time.

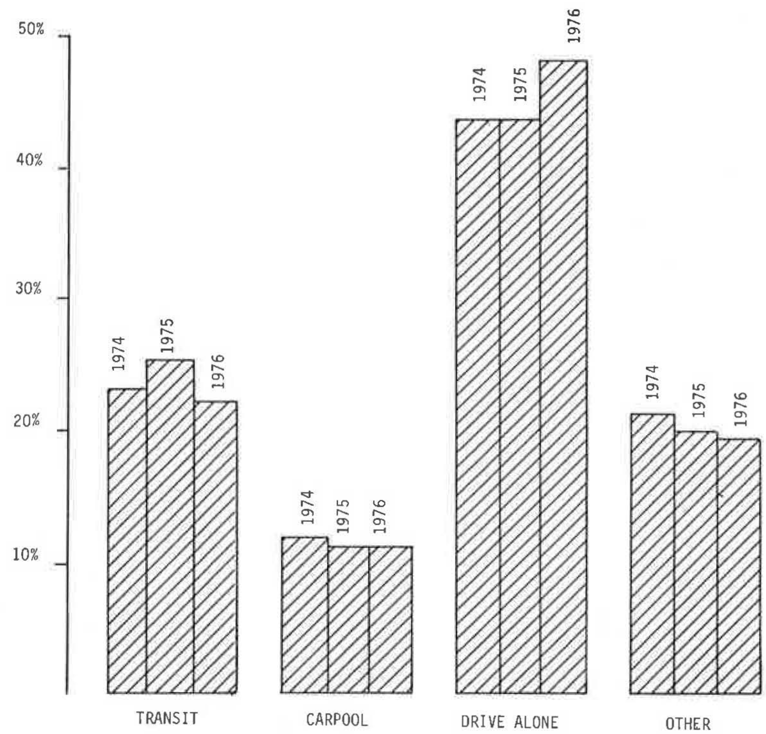


Table 2. Transition matrix for most frequent travel mode for 1974 versus 1975.

Most Frequent Travel Mode 1974	Most Frequent Travel Mode 1975 (%) Transit	Carpool	Drive Alone	Other	Total	Valid Cases (N = 227)
Transit	77	4	17	2	100	53
Carpool	25	57	7	11	100	28
Drive alone	7	3	85	5	100	98
Other	4	6	4	86	100	48

Table 3. Transition matrix for most frequent travel mode for 1975 versus 1976.

Most Frequent Travel Mode 1975	Most Frequent Travel Mode 1976 (%) Transit	Carpool	Drive Alone	Other	Total	Valid Cases (N = 364)
Transit	60	11	22	7	100	103
Carpool	17	46	32	5	100	41
Drive alone	7	6	85	2	100	154
Other	3	2	15	80	100	66

in shuttle-bus use. This decrease in shuttle-bus use is probably attributable to the change in price structure from a free service in 1975 to a 15-cent fare in 1976. Use of carpooling and other categories (chiefly walking, bicycling, and motorcycling) has remained nearly constant over time.

Respondents were asked to explain changes in their most frequent mode of travel, and this information is summarized below. These explanations seem to indicate that most respondents were not adversely affected by changes in parking policy.

Reason for Mode Change	Percentage Responding
Moved	40
Bought a car	15
Change in class times	11
Joined a carpool	7
Additional UBUS service	7
Sold car	5
Quit carpool	5
Other reasons	10
Total	100

A better understanding of the shifts that have occurred over time can be obtained by looking at the intermodal crossover rates in transition matrices as shown in Tables 2 and 3. A transition matrix provides a comparison of the mode of travel used by the respondent in one year with that chosen in the following year. With such a matrix, the portions of travelers that made no modal shifts is shown on the diagonal of the matrix while those that showed modal shifts are shown elsewhere. If there are no modal changes, the matrix would appear as a diagonal with zeros elsewhere. A further explanation of this subject follows.

Understanding the Transition Matrix

This report relies, to a great extent, on the transition matrix as a means of presenting the analysis of survey data. This transition matrix is used to compare attributes of the study group in two time periods and, as its name implies, illustrates changes (transitions) in respondent behavior as a function of time.

Consider the following example. You and 49 other people are employed by a firm that allows each of its employees and their families a one-week all-expense-paid vacation each year. The company allows only four choices: London, Paris, New York, or Disneyworld. Assuming that each year all employees take advantage of this offer, let us further assume that last year the number of employees vacationing at each location is as listed below.

Location	No.	Vacations (%)
London	15	30
Paris	10	20
New York	10	20
Disneyworld	15	30
Total	50	100

All plan to return to the same location this year. In this instance, since there are no changes from the previous year's choices, the table also represents this year's employee vacation choices.

Table 4 presents a transition matrix that shows that there was no change in location choice in the two time periods: There are entries in the diagonal cells (i.e., London-London, Paris-Paris, etc.) only. If, however, you and your fellow employees had been inclined to vacation in a different city, this decision would be reflected by entries in the nondiagonal cells. The table below represents changes that could have occurred, and Table 5 is a transition matrix of all the information in this table.

Last Year		This Year	
Location	No.	Location	No.
London	15	London	3
		Paris	9
		New York	2
		Disneyworld	1
Paris	10	London	1
		Paris	7
		New York	0
		Disneyworld	2
New York	10	London	0
		Paris	6
		New York	1
		Disneyworld	3
Disneyworld	15	London	2
		Paris	8
		New York	3
		Disneyworld	2

The concise format of the transition matrix helps make it a powerful tool in the interpretation of change-related data. By examining the diagonal entries, consistencies become apparent. In our example, we can see that Paris was a more popular spot (70 percent return rate) than New York (10 percent return rate). The matrix also shows us that those who traveled to New York last year are unlikely to vacation in London this year (0 percent) and that those who traveled to Paris are not attracted to New York (0 percent). Many other observations about the travel preferences can be obtained from the transition matrix, and these observations can be related to experience and expectations surrounding each location.

Study Application of the Transition Matrix

This study uses the transition matrix in a manner similar to that of the above example. Also used, however, was information regarding actual and perceived changes in modal attributes, the availability and cost of parking and transit service, and other factors affecting mode choice. Once the reader has become familiar with the development and interpretation of the transition matrix, he or she will more fully understand the content of this report and perhaps be able to use this concept in similar studies.

In the first of the matrices (Table 2), the travel mode used in 1974 is compared to that used in 1975. This was the period during which substantial restrictions were added to street parking and an attractive alternative (a free shuttle bus from a remote parking lot) was provided to the automobile. This was also a period during which policies were strongly directed to encouraging a shift from automobile to transit. However, as can be seen from Table 2, the transit, drive-alone, and other modes remained relatively stable during this period. Transit experienced some shift in use to the drive-alone category (17 percent) but gained some riders who shifted from carpool to transit use (26 percent). Thus, it appears that most of the gain in transit ridership that occurred during the period was from a shift from carpooling (automobile passenger) rather than from driving alone.

Table 3 illustrates the modal shifts that occurred between 1975 and 1976. During this period a fare of 15 cents/trip was added to the shuttle-bus service, and there were some limited additional restrictions placed on parking. This would be thus characterized as a period when a disincentive to transit use (i.e., a higher fare) was imposed. As can be seen from Table 3, this was a period of instability for transit and carpool users.

Table 4. Transition matrix for unchanged vacation choice.

Place Chosen Last Year	Place Chosen This Year (%)					Valid Cases (N = 50)
	London	Paris	New York	Disneyworld	Total	
London	100	-	-	-	100	15
Paris	-	100	-	-	100	10
New York	-	-	100	-	100	10
Disneyworld	-	-	-	100	100	15

Table 5. Transition matrix for changed vacation choice.

Place Chosen Last Year	Place Chosen This Year (%)					Valid Cases (N = 50)
	London	Paris	New York	Disneyworld	Total	
London	20	60	13	7	100	15
Paris	10	70	0	20	100	10
New York	0	60	10	30	100	10
Disneyworld	13	54	20	13	100	15

Figure 3. Parking location as a function of time.

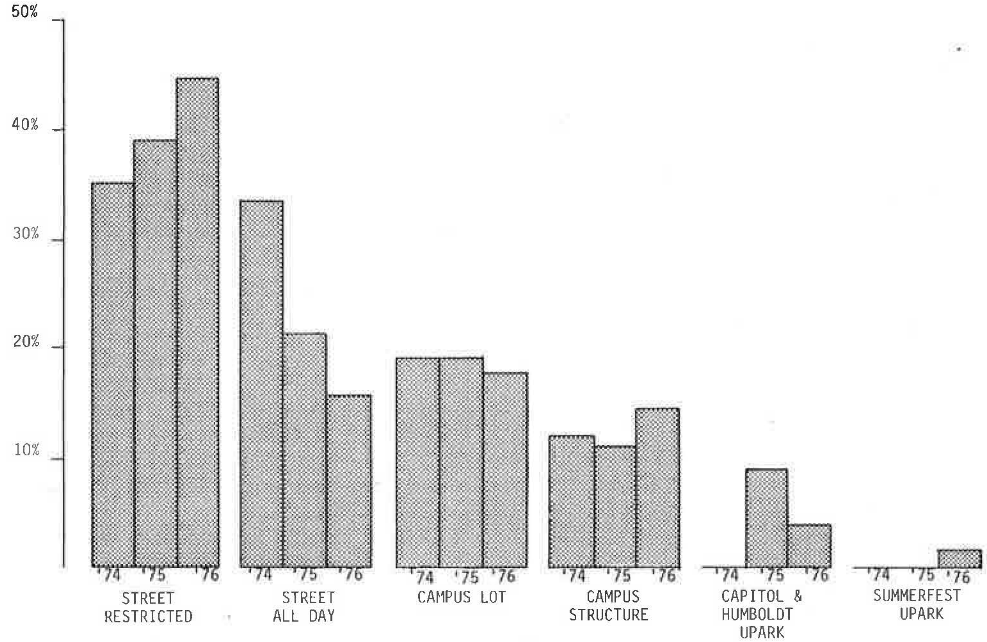


Table 6. Transition matrix for parking location for 1974 versus 1975.

	Parking Location 1975 (%)					
Parking Location 1974	Street Restricted	Street All Day	Campus Facility	UPARK Lot	Total	Valid Cases (N = 157)
Street restricted	84	7	7	2	100	56
Street all day	18	60	6	16	100	50
Campus facility	8	6	84	2	100	51
UPARK lot	-	-	-	-	-	-

Table 7. Transition matrix for parking location for 1975 versus 1976.

	Parking Location 1976 (%)					
Parking Location 1975	Street Restricted	Street All Day	Campus Facility	UPARK Lot	Total	Valid Cases (N = 234)
Street restricted	78	5	14	3	100	94
Street all day	30	51	15	4	100	46
Campus facility	13	6	77	4	100	71
UPARK lot	30	22	22	26	100	23

Former transit users (22 percent) and former carpoolers (32 percent) made fairly substantial shifts to the drive-alone category, but the shift away from transit was partly balanced by a shift from carpool to transit (17 percent).

The policy changes that occurred seem to have had their greatest effect on those who rode with someone else rather than on drivers. There is some limit to the extent to which increases in parking restrictions can lead to a shift to transit use. The respondents seemed to be much more sensitive to price changes than to changes in parking restrictions.

Changes in Parking Location Over Time

Changes in the location of student parking over time are an indication of changes in parking supply and restrictions. Figure 3 graphically summarizes changes in parking location during the 1974-1976 period. This figure illustrates several important points:

1. Use of restricted parking corresponds to increased restrictions on street parking.
2. The Capitol and Humboldt UPARK park-and-ride lot had a much lower rate of use in 1976 than in 1975.
3. The Summerfest UPARK park-and-ride lot is used by only 1 percent of the total market.
4. Campus parking structures are used more now than in the past, despite cost increases.
5. In general, on-street parking has increased by about 10 percent in the 1975-1976 period.

As was the case with mode-choice shifts, the transition matrix is useful for understanding changes in parking locations. Changes in parking location that occurred between 1974 and 1975 are shown in Table 6. Those who parked either in restricted street locations or in a campus facility during 1974 generally did not change locations in 1975. However, there were noteworthy changes in parking location for those who had used all-day on-street parking in 1974: About one-fifth of this group shifted to restricted on-street parking in 1975, and approximately one-sixth began to use the recently completed Capitol and Humboldt UPARK lot.

Changes that occurred between 1975 and 1976 are more dramatic. These changes are shown in Table 7. During this time the fare on the UPARK shuttle bus increased from no charge to 15 cents/ride, and there was a reduction in on-street unrestricted parking. The table shows that about one-third of those who had parked in unrestricted street spaces began to park in restricted locations and about 15 percent began to use a campus parking facility. There were only minor changes in parking location for those who used on-campus parking facilities or parked on the street in restricted areas.

Those who had been using the UPARK shuttle service in 1975 tended not to do so in 1976. About one-quarter of this group began to use each of the other three parking location categories: street restricted, street all day, and campus facility. Only one-fourth of the 1975 UPARK patrons used the service during 1976, and of those who changed parking location in this period, the UPARK lots were the least likely location to be used in 1976.

Survey respondents were also asked to rank the effect of past changes in parking availability and UBUS routes and schedules. This information is summarized in Figure 4, which shows that more than 70 percent of all respondents were not affected at all by changes in UBUS routes and schedules and that only 6 percent were affected very much.

On the other hand, students appeared to be affected much more by changes in the price and supply of street parking. Almost 45 percent were affected somewhat or very much; 18 percent were affected a little; and 37 percent were not affected. Slightly more than 50 percent of the respondents were affected at least a little by changes in campus parking lots, but about 50 percent were not affected at all.

Future Travel Behavior

The third section of the survey was appropriately entitled "The Future" and was used to determine the effect of changes in transit fares and parking supply on mode choice. This was done by describing a series of hypothetical situations and asking the respondents what they would do in such a situation. Although what one says one will do in the future may not be what one actually does, such information is useful in comparisons of past behavior and other hypothetical futures. Analysis of the data obtained in this portion of the survey is provided in Tables 8-14, which are transition matrices similar to those used earlier.

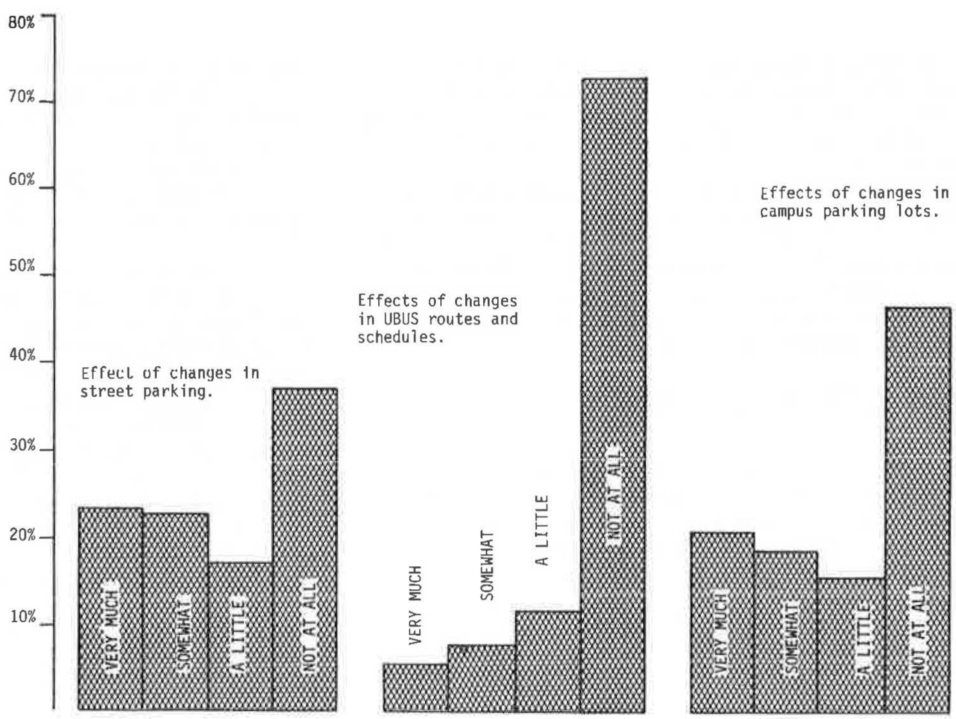
Changes in Parking Restrictions

Tables 8 and 9 provide a comparison of two extreme alternate futures: one in which all street parking is unrestricted, the other in which student street parking is eliminated. Respondents were asked to select the mode they would most likely use if each of these conditions were to occur in the future. As can be seen from the tables, use of the drive-alone mode would increase somewhat with shifts from transit (primarily shuttle-bus users), carpool, and other categories if parking restrictions were removed. On the other hand, if street parking were eliminated, the transit mode would benefit from substantial shifts by the carpoolers and single drivers. However, it is interesting to note that about half of the drivers (48 percent) and the carpools (52 percent) would continue to use the automobile even if there were no street parking available.

Some insight into what would be done by those who now park on the street if parking were severely restricted can be found by looking at Table 10. Respondents were asked where they would park if all street parking were limited to an hour or less. As the table shows, about one-fifth of those who currently park on the street would no longer use their cars. Another fifth would continue to park on the street, while the remainder would seek parking—campus facilities, UPARK lots, or other locations (friends' garages or along UBUS routes). Nearly all of those who now use campus facilities or UPARK lots would continue to use these locations.

From these analyses, it can be seen that travelers to UWM exhibit some degree of sensitivity to potential changes in parking regulations. A loosening of restrictions would lead to some losses in transit ridership, while a tightening of restrictions would lead to larger shifts to transit. However, there is a tendency to stay with the mode being used in most situations.

Figure 4. Effects of past changes in parking availability and UBUS routes and schedules.



Thus, the analysis of future conditions generally agrees with that of past changes.

Changes in Transit Fares

Tables 11-14 deal with future changes in UBUS fares and shuttle-bus fares, respectively. In Tables 11 and 12 the effects of two extremes in UBUS fares are analyzed. Table 11 looks at a free-fare service, while Table 12 deals with increasing the present 35-cent fare to 50 cents. If free transit service were provided, there would be a shift from the carpooling, driving alone, and other modes to transit (39 percent, 29 percent, and 15 percent, respectively). This shift is fairly large but not as substantial as the shift to transit if street parking were eliminated. If, on the

Table 8. Transition matrix for effects of all-day parking for present versus future mode.

Present Mode	Future Mode (%) Transit	Carpool	Drive Alone	Other	Total	Valid Cases (N = 537)
Transit	67	8	19	6	100	145
Carpool	10	68	15	7	100	68
Drive alone	7	7	83	3	100	242
Other	1	6	10	83	100	82

Table 9. Transition matrix for effects of no street parking for present versus future mode.

Present Mode	Future Mode (%) Transit	Carpool	Drive Alone	Other	Total	Valid Cases (N = 511)
Transit	91	1	3	5	100	144
Carpool	32	52	5	11	100	65
Drive alone	37	7	48	8	100	222
Other	3	2	3	92	100	80

Table 10. Transportation matrix for effects of 1-h street parking for present versus future location.

Present Location	Future Location (%) Will Not Use Automobile	Street Restricted	Campus Facility	UPARK Lot	Other	Total	Valid Cases (N = 315)
Street restricted	23	20	34	11	12	100	138
Street all day	21	17	13	26	23	100	53
Campus facility	10	10	70	2	8	100	106
UPARK lot	10	0	0	90	0	100	18

Table 11. Transition matrix for effects of free-fare UBUS for present versus future mode.

Present Mode	Future Mode (%) Transit	Carpool	Drive Alone	Other	Total	Valid Cases (N = 548)
Transit	96	0	1	3	100	150
Carpool	39	47	7	7	100	70
Drive alone	29	3	65	3	100	242
Other	15	2	1	82	100	86

Table 12. Transition matrix for effects of 50-cent UBUS fare for present versus future mode.

Present Mode	Future Mode (%) Transit	Carpool	Drive Alone	Other	Total	Valid Cases (N = 534)
Transit	80	5	10	5	100	148
Carpool	14	75	4	7	100	69
Drive alone	5	5	88	2	100	233
Other	2	5	2	91	100	84

Table 13. Transition matrix for effects of free-fare UPARK for present versus future mode.

Present Mode	Future Mode (%) Transit	Carpool	Drive Alone	Other	Total	Valid Cases (N = 510)
Transit	93	1	2	4	100	134
Carpool	29	54	8	9	100	65
Drive alone	17	5	75	3	100	230
Other	4	5	2	89	100	81

Table 14. Transition matrix for effects of 50-cent UPARK fare for present versus future mode.

Present Mode	Future Mode (%) Transit	Carpool	Drive Alone	Other	Total	Valid Cases (N = 495)
Transit	86	2	6	6	100	125
Carpool	16	65	13	6	100	62
Drive alone	6	5	85	4	100	228
Other	1	5	3	91	100	80

other hand, UBUS fares were raised by 15 cents, only minor changes in mode choice would occur, and most people would stay with their present modes.

Tables 13 and 14 deal with changes in UPARK shuttle-bus fares. Again, two extreme futures are considered: one in which UPARK shuttle is changed to a free-fare structure and one in which UPARK shuttle fares are increased to 50 cents. The transition matrices indicate a shift toward transit (which includes the shuttle-service users) by carpoolers and drivers if the shuttle service were free and relatively little change if the shuttle fare were increased. As has been the case in the previous tables, carpoolers tend to exhibit a greater tendency to change modes than others do. Thus, from these analyses, it would appear that cost reductions in transit service will result in shifts toward transit, while increases will have less effect on ridership. Carpool users again seem to be the most sensitive of travelers to changes in price.

CONCLUSIONS

From the preceding analysis of the relation of transit use to parking restrictions and pricing changes, several conclusions can be drawn about the survey respondents.

From the analysis of past changes, it is apparent that shifts to transit use can occur as a result of tighter parking restrictions. They seem to have their greatest effect on those respondents who ride with someone else rather than on drivers or transit users. The respondents were much more sensitive to pricing changes than to changes in parking restrictions.

From an analysis of future situations, similar patterns occurred. Shifts from automobile to transit are likely as parking restrictions increase, again especially for those respondents who ride with someone. However, even with severe restrictions on street parking, many respondents felt that they would tolerate the inconvenience created by the restrictions rather than shift to transit. Decreasing the price of transit can also lead to increased transit use, while an increase in price had less effect on transit use.

It appears from the situation analyzed that the tie to the automobile is strong for many and that disincentives to automobile use will cause shifts to other modes only to a limited extent. This result should serve as a warning to those who expect major changes in mode use as a result of parking-policy changes.

Such disincentives need to be coupled with strong efforts to provide an attractive transit service as an alternative to the automobile. In that way, the two competing modes can be made to function in a complementary fashion for the overall efficiency of the transportation system. Further analysis of this important issue needs to be made so that a better understanding of the phenomenon can be applied in more effective policymaking.

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Incident-Detection Algorithms

Part 1. Off-Line Evaluation

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Five incident-detection algorithms of the pattern-recognition type were evaluated off-line by using incident and incident-free data collected on Chicago's expressways under various traffic and environmental conditions. Algorithm efficiency was evaluated in terms of detection and false-alarm rates and mean-time-to-detect. Evaluated were a comparative analysis of algorithm efficiency, the effect of lateral detectorization on algorithm performance, a hierarchical analysis of threshold effectiveness, and the effect of incident severity on algorithm performance. Although no specific algorithm was found to be superior for levels of detection lower than 95 percent, for higher levels of detection one algorithm developed by Technology Services Corporation was found to be best. The algorithms did not differ statistically in mean-time-to-detect, which ranged from 2 to 4 min, rendering this parameter ineffective in algorithm selection. The relation between detection rate and false-alarm rate, however, was found to be the critical criterion for algorithm selection. Feature thresholds developed for detector-lane incidents were found to be less sensitive to traffic-flow disturbances than were thresholds developed for non-detector-lane incidents, thus yielding lower false-alarm rates. Analysis of algorithm performance under various traffic and environmental conditions revealed that thresholds developed for a representative sample of incidents were effective when used on the "rush wet", "nonrush dry", and "nonrush wet" traffic data. Therefore, less effort was needed to develop the set of thresholds. Thresholds developed for accidents occurring on the detector lane proved to be effective in detecting accidents and nonaccident incidents on both the detector and non-detector lanes.

Freeway incident-management systems that offer various levels of service to the motoring public have been in operation for quite some time. In essence, each such system provides some or all of the following system elements:

1. Detection of traffic-flow abnormalities,
2. Incident identification,
3. Traffic-management strategies and tactics to be implemented through driver communication and control subsystems, and
4. Early removal of incidents by motorist-aid subsystems.

The comprehensiveness of the incident-management system and the level of sophistication of its elements will determine the operational efficiency of the system.

A key element of such a system is the detection of traffic-flow abnormalities and their identification as capacity-reducing incidents. A positive identification will normally activate the control, driver communication, and incident-handling subsystems. Obviously, a missed incident or a false alarm will affect the efficiency of the management system and its credibility. But the incident-detection process uses algorithms that relate certain measured relations among traffic characteristics to calibrated thresholds to yield a decision with regard to the incident.

The Federal Highway Administration (FHWA) contracted with Technology Services Corporation (TSC) to evaluate existing algorithms (1) and to develop new ones (2). The evaluation included pattern recognitions (3, 4) and time-series algorithms (5, 6). The Illinois Department of Transportation (IDOT) has assumed the task of the off-line and on-line evaluations of the selected algorithms developed by TSC. The facilities of IDOT's Traffic Systems Center will be used for this.

The specific objectives of the research reported here were

1. To determine the efficiency of the selected TSC algorithms in detecting incidents on the Chicago-area expressway system for various traffic and environmental conditions,
2. To develop algorithm thresholds compatible with the traffic characteristics of the expressway system and various environmental conditions,
3. To determine the effect of the existing level of detectorization on algorithm performance,
4. To determine the effect of incident severity on algorithm performance, and
5. To compare the efficiency of TSC algorithms with a pattern-recognition algorithm developed locally.

ALGORITHM DESCRIPTION

This section describes the structure of the incident-detection algorithms evaluated in this research. They include five pattern-recognition algorithms, four of which were developed by TSC (2); the fifth was developed locally in the course of the research.

The research effort of TSC included the development of 10 incident-detection algorithms. Algorithms 1-7 are variations on the classic California algorithm (3), while 8 and 9 use, in addition to those elements of algorithm 7, a feature that suppresses incident detection at any station for 5 min after detection of a compression wave at the downstream station. Algorithm 10 attempts to detect incidents occurring in light-to-moderate traffic that do not lower capacity below the volume of oncoming traffic.

Of these 10 algorithms, 4 were selected for evaluation: algorithms 7, 8, 9, and 10. Preliminary investigation indicated algorithm 7 to be a superior form of the California algorithm. Algorithm 8, which is identical to algorithm 9 except for an added persistence check, was found to have, according to TSC's investigation, a slightly lower false-alarm rate (FAR) but a longer mean-time-to-detect (MTTD) than algorithm 9. Although algorithm 10 did not perform especially well, it was included in the off-line evaluation because it represents a first attempt to solve the problem of detecting incidents that do not produce marked traffic-flow discontinuities.

The TSC algorithms are in binary decision-tree form; at each node of the decision tree a feature value is compared with a user-specified threshold value to determine whether an incident is to be signalled. Obviously the effectiveness of the algorithm depends on the thresholds chosen.

TSC developed a program for optimizing threshold selection. This program, called CALB, uses a random-number generator that produces increments to be added to the current optimal threshold vector to produce a new threshold vector for evaluation. After a predetermined number of iterations, the threshold vector with the lowest false-alarm rate, given a certain level of detection, is termed the optimal threshold vector at that level of detection.

Before CALB was used to calibrate the algorithms for the off-line evaluation, a detailed study was performed to determine how best to set certain user-supplied parameters needed by CALB in the algorithm calibration process. The point was to ensure selection of the best threshold vectors for use in the algorithm evaluation.

Finally, the four TSC algorithms selected were compared with algorithm 16-14, one in a series of pattern-recognition algorithms developed in the course of this research (7).

Following is a detailed description of the above algorithms; the meanings of the features involved in each algorithm are given in the listing below.

Feature Name	Definition
OCC(t)	Minute average occupancy measured at upstream detector at time t
DOCC(t)	Minute average occupancy measured at downstream detector at time t
OCCDF(t)	OCC(t) - DOCC(t)
OCCRDF(t)	OCCDF(t)/OCC(t)
SPEED(t)	Minute average speed calculated at upstream detector at time t
DOCCTD(t)	[DOCC(t-2) - DOCC(t)] /DOCC(t-2)
SPDTDF(t)	[SPEED(t-2) - SPEED(t)] /SPEED(t-2)
OCCRDF(t-1)	[OCC(t-1) - DOCC(t-1)] /OCC(t-1)
UPDF(t)	OCC(t-1) - OCC(t-2)
UPRDF(t)	UPDF(t)/OCC(t-1)
DNDF(t)	DOCC(t-2) - DOCC(t-1)
DNRDF(t)	DNDF(t)/DOCC(t-2)
DPDNDF(t)	UPDF(t) = DNDF(t)
UPDNR1(t)	UPDNDF(t)/OCC(t-1)
UPDNR2(t)	UPDNDF(t)/[OCC(t-1) - DOCC(t-1)]
RDF(t)	OCCDF(t)/[OCC(t-1) - DOCC(t-1)]

Algorithm 7 differs from the classic California algorithm in the following three ways. Whereas the California algorithm produces an incident signal whenever OCCDF, OCCRDF, and DOCCTD are greater than associated thresholds, algorithm 7 replaces DOCCTD with DOCC, suppresses incident signals after the initial detection, and contains a persistence requirement that OCCRDF be greater than the threshold for two consecutive minutes (Figure 1).

Algorithm 9 consists of algorithm 4 (a variant of the California algorithm) coupled with a compression-wave check and uses features DOCC and DOCCTD. It works as follows. First, a compression-wave check is made. If it succeeds, then algorithm 4 is not applied until five consecutive minutes have passed without a compression wave. If it fails then algorithm 4 is immediately applied.

Algorithm 8 is algorithm 9 with an OCCRDF-persistence requirement added. It can also be thought of as algorithm 7 incorporated with the 5-min compression-wave check (Figure 2).

Algorithm 10 separates traffic data into light, moderate, and heavy traffic by using the feature OCC. No incident check is applied to light-traffic data. Algorithm 7 is used under heavy-traffic conditions, and under moderate conditions OCCRDF and SPDTDF, a temporal speed change feature, are applied (Figure 3).

Algorithm 16-14 is a pattern-recognition algorithm developed locally by using occupancy-based features obtained through intensive observations of traffic behavior on different parts of the Chicago-area expressway system (Figure 4).

DEVELOPMENT OF DATA BASE

The data base was divided into two parts: incident data used to compute an algorithm's detection rate (DR) and MTTD and incident-free data used to calculate an algorithm's FAR. The surveillance data that make up each set consist of 20-s occupancies and volumes from each main-line detector on the relevant directional freeway. The data base includes a total of 100 incident and 14 incident-free data sets.

In the collection of incident data sets, "incident" was limited to mean unplanned physical obstructions of the traveled lanes. The incident data were collected by monitors at IDOT's Traffic Systems Center. Indications of a potential incident came in two ways. In the most common case, the data collector would spot a disturbance in the traffic-stream variables by monitoring the expressway system map panel, occupancy maps on the display, or typer output of the surveillance system. In these cases, the monitor would activate a program for saving the surveillance data from the affected directional expressway (the data-collection program kept a 30-min historical file of surveillance data that enabled the requisite 15 min of pre-incident data to be saved, if an incident was detected by the monitor within 15 min of its occurrence). The monitor then requested the IDOT Communication Center to dispatch an emergency patrol vehicle (EPV) to the area for confirmation and identification. In other cases, an incident would be reported by a field unit before signs of it appeared in the surveillance data. When traffic-stream measurements began to manifest signs of the incident's effect on traffic operations, data saving was initiated.

The incident data were collected to represent the following factors:

1. Rush or nonrush traffic conditions (R, NR),
2. Wet or dry pavement conditions (W, D),
3. Accident (10-50) or nonaccident incident (10-46) incident type (AI, NAI) according to Illinois State Police code, and
4. Detector lane or non-detector-lane (DL, NDL) incident lateral location.

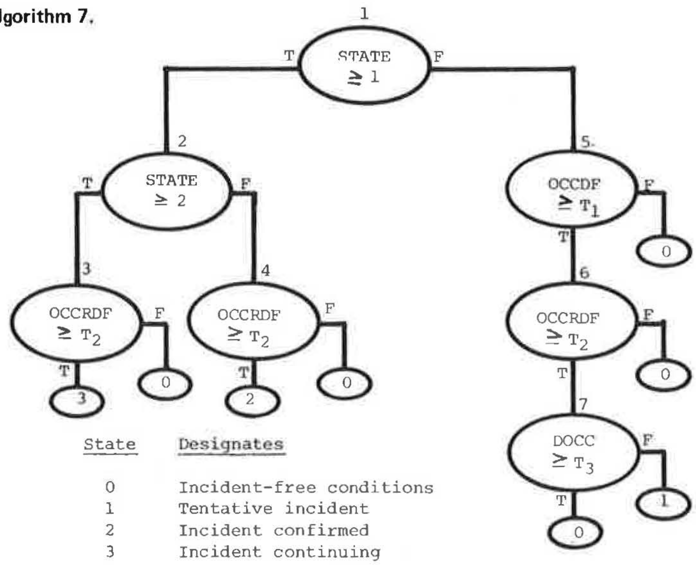
Figure 5 shows the stratification of the incident data and the code of each stratum. The meanings of the codes are explained below.

Code	Interpretation
R	Rush
RW	Rush wet
RD	Rush dry
RD-0	Incident occurring on nondectector lanes during rush dry period
RD-1	Incident occurring on detector lane during rush dry period
RD-50-1	Accident occurring on detector lane during rush dry period
RD-50-0	Accident occurring on nondetector lanes during rush dry period
RD-46-1	Nonaccident incident occurring on detector lane during rush dry period
RD-46-0	Nonaccident incident occurring on nondetector lanes during rush dry period
RD-50	Accident occurring during rush dry period
RD-46	Nonaccident incident occurring during rush dry period

NR, NRW, NRD, NRD-0, NRD-1, NRD-50-0, NRD-46-0, NRD-50-1, and NRD-46-1 have the same interpretation as above except that they refer to the nonrush period.

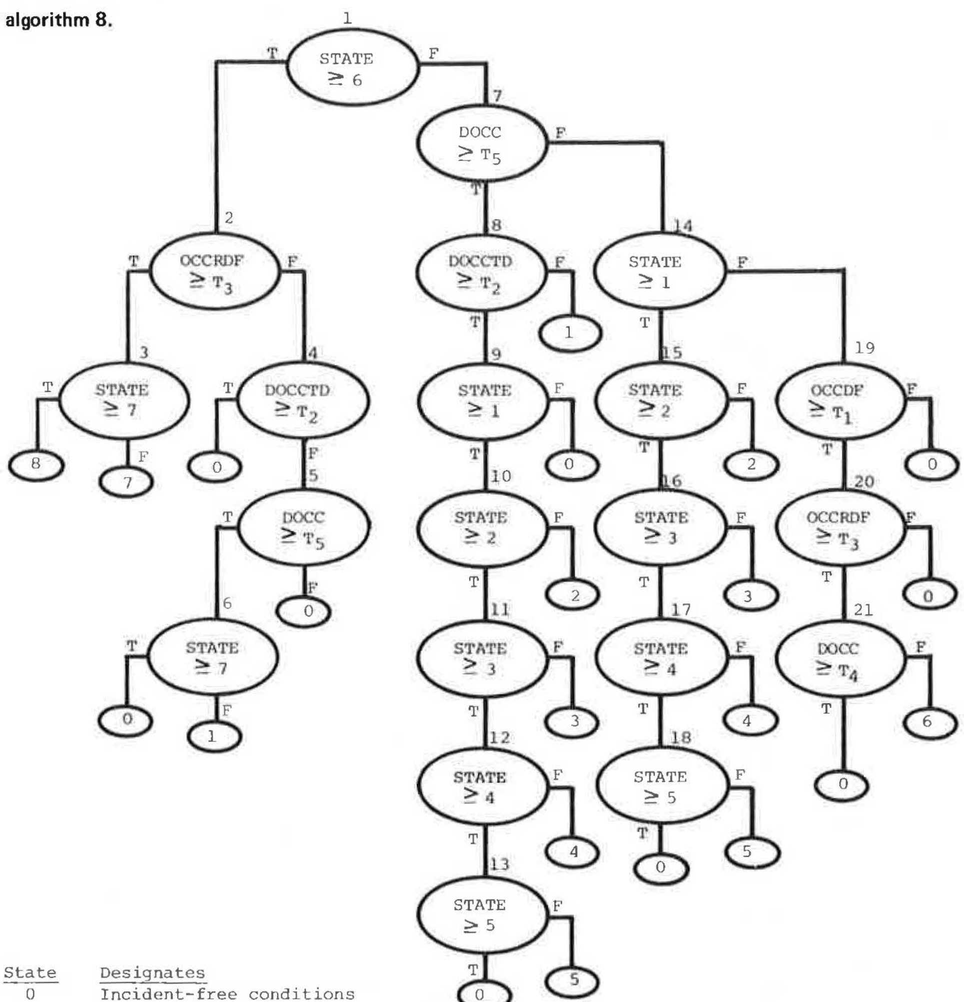
The collection of incident-free data sets involved the use of the same data-saving software as employed in the incident data collection. Verification of these data as incident-free was carried out with the use of a helicopter. Nearly 30 h of incident-free data were collected to appropriately represent rush, nonrush, wet, and dry conditions.

Figure 1. Decision tree for algorithm 7.



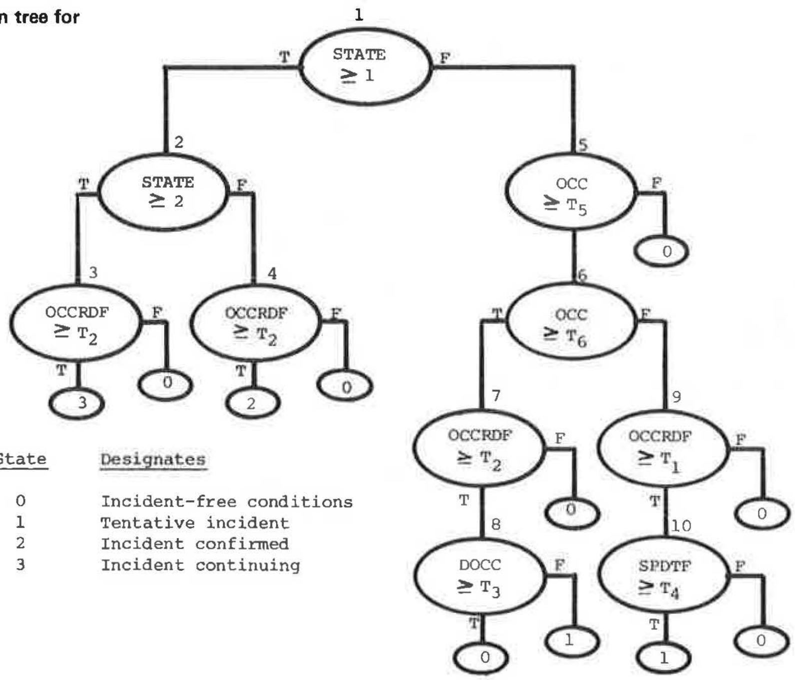
State	Designates
0	Incident-free conditions
1	Tentative incident
2	Incident confirmed
3	Incident continuing

Figure 2. Decision tree for algorithm 8.



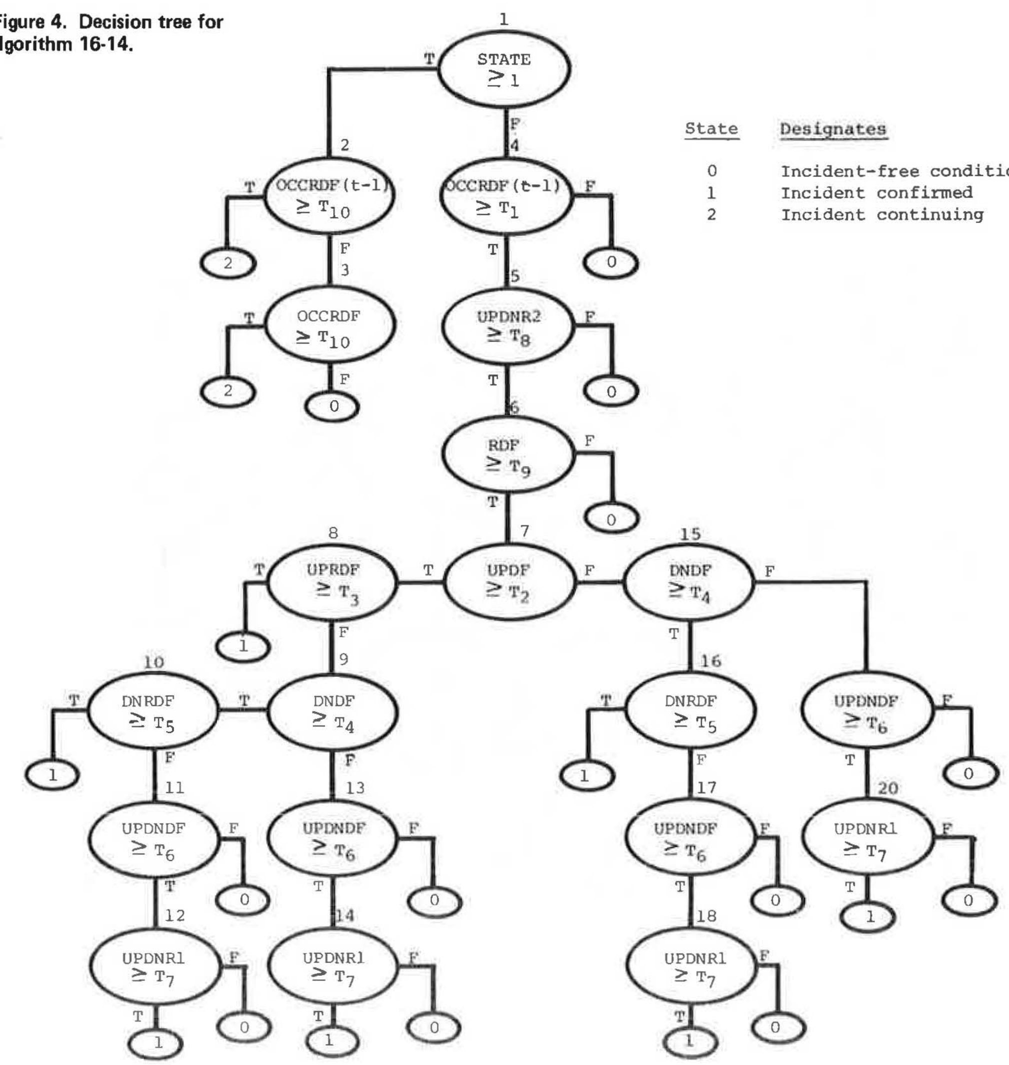
State	Designates
0	Incident-free conditions
1	Compression wave downstream in this minute
2	Compression wave downstream 2 minutes ago
3	Compression wave downstream 3 minutes ago
4	Compression wave downstream 4 minutes ago
5	Compression wave downstream 5 minutes ago
6	Tentative incident
7	Incident confirmed
8	Incident continuing

Figure 3. Decision tree for algorithm 10.



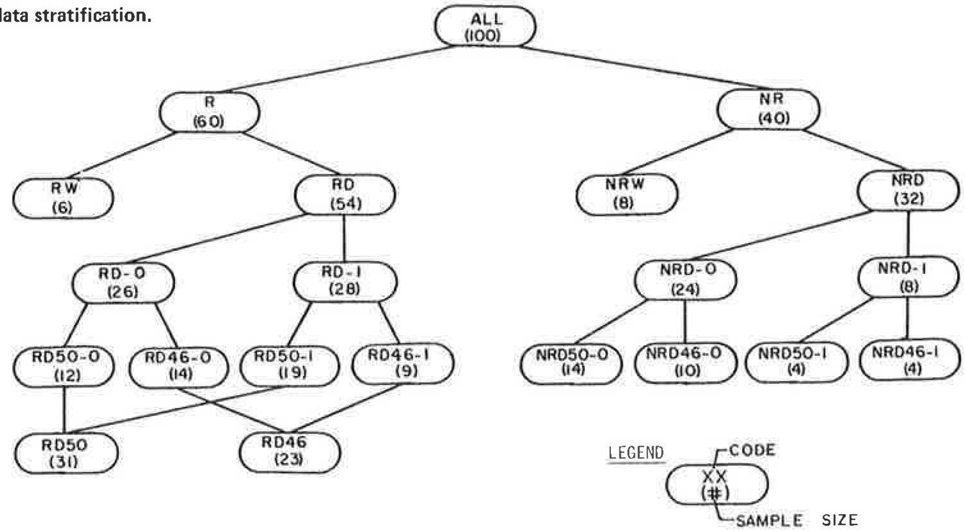
State	Designates
0	Incident-free conditions
1	Tentative incident
2	Incident confirmed
3	Incident continuing

Figure 4. Decision tree for algorithm 16-14.



State	Designates
0	Incident-free conditions
1	Incident confirmed
2	Incident continuing

Figure 5. Incident-data stratification.



OFF-LINE EVALUATION

The ultimate goal of the off-line evaluation was to obtain for the tested algorithms optimal sets of thresholds related to various traffic and environmental conditions. These sets could then be implemented in an operational on-line incident-response system. To achieve that goal off-line evaluation was divided into four major tasks:

1. Comparative analysis of algorithm efficiency,
2. Evaluation of the effect of lateral detectorization on algorithm performance,
3. Hierarchy analysis of the threshold effectiveness, and
4. Evaluation of the effect of incident severity on algorithm performance.

Algorithm efficiency could be determined by three related parameters:

1. DR: percentage of detected incidents out of all incidents that affect traffic and occur during a specified time period;
2. FAR (off-line definition): percentage of incident messages (1s) out of all messages (1s and 0s) where messages are produced at specific intervals (i.e., every 1 min) out of representative incident-free data; and
3. MTTD: the mean delay between the apparent occurrence of incidents, as estimated from changes in upstream and downstream occupancy values, and their detection time by the algorithms during a certain period of time.

Comparative Analysis of Algorithm Efficiency

The comparative analysis of the tested algorithms was performed by running each of the five algorithms—7, 8, 9, 10, and 16-14—through the various incident and incident-free data strata, by using TSC's CALB program, which had been modified for the Traffic Systems Center's computer. The CALB evaluation of these algorithms was performed for six nominal detection rates of 75, 80, 85, 90, 95, and 99 percent and for three incident data categories: ALL, RD, and NRD. The strata of RW and NRW included only six and eight incident cases, respectively, and were excluded from the detailed analysis.

A comparison of the DR-FAR relationships of algorithms 9 and 16-14 with those of algorithms 7, 8, and 10 indicated that algorithms 9 and 16-14 experienced relatively high FAR across the whole DR spectrum. At the same time, however, their DR-MTTD relationships seemed to be more favorable than those of the other algorithms. However, because in many cases the differences in MTTD for the various algorithms were not found to be statistically significant, the relatively poor DR-FAR relationships between algorithms 9 and 16-14 suggested their elimination from further analysis even though favorable results were indicated for algorithm 9 (2). However, for the sake of representative analysis and future on-line evaluation, it was decided to eliminate only algorithm 9.

Overall, the three algorithms (7, 8, 10) produced better DR-FAR relationships for the NRD category than for the RD category. Over the investigated range of the DR, the FAR for the NRD category ranged from 0.00 to 0.01 percent, while the range for the RD category was from 0.02 to 0.11 percent.

Within the RD category no single algorithm displaying invariably better FARs over the DR spectrum could be found. However, for the higher DRs (0.95 and above), algorithm 7 was the most efficient. Also, the same algorithm was found to yield the fewest FARs over the whole DR spectrum for the NRD category.

The time-to-detect analysis used the optimal sets of thresholds developed for the DR-FAR relationships. The MTTD for the RD and NRD categories ranges from 1.9 to 4.4 min and from 3.6 to 6.2 min, respectively. The results for the ALL category (2.2-4.6 min) represent, to a large extent, the combinations of the RD and NRD results.

Within the RD category, algorithm 7 displayed the lowest MTTD for DRs higher than 95 percent. For lower DRs, no single, most efficient algorithm could be found. Within the NRD category no single algorithm displayed invariably lower MTTD over the whole DR spectrum.

Further insight into the differences in MTTD between algorithms for the various incident data categories was gained from the Kolmogorov-Smirnov test and the Mann-Whitney U-test (8) for thresholds representing the 95 percent detection level. This level was selected for its assumed applicability to an operating on-line system. The results of the statistical analyses for algorithms 7, 8, 10, and 16-14 are presented in Table 1. From this table it can be seen that, as far as the MTTD is concerned, no statistically significant difference (0.05 level

Table 1. Comparison of algorithm performance at 95 percent detection rate for ALL, RD, RW, NRD, and NRW conditions.

Traffic Category	Sample Size	Algorithm No. 7	8	10	16-14	Apparent Best Algorithm	Statistically Best Algorithm (for MTTD)
ALL	99						
FAR, %		0.019	0.0297	0.0231	0.11	7	
MTTD, min		3.39	2.85	3.68	2.28	16-14	None[a]
SD, min		3.25	3.01	3.42	3.05		
RD	54						
FAR, %		0.056	0.0786	0.067	0.26	7	
MTTD, min		2.23	2.75	2.88	1.26	16-14	None[a]
SD, min		1.60	2.15	2.65	1.83		
RW	6						
FAR, %		0.0336	0.0	0.045	0.045	8	
MTTD, min		2.83	3.99	2.50	2.33	10	None[a,b]
SD, min		0.69	2.89	5.02	3.03		
NRD	32						
FAR, %		0.002	0.002	0.005	0.018	7	
MTTD, min		3.73	3.56	2.87	3.22	10	None[a]
SD, min		3.75	3.77	2.49	4.72		
NRW	8						
FAR, %		0.005	0.005	0.005	0.009	7, 8, 10	
MTTD, min		2.71	2.63	2.50	1.88	16-14	None[a,b]
SD, min		2.31	1.99	2.24	2.15		

[a]Kolmogorov-Smirnov test at the 0.05 level of significance. [b]Mann-Whitney U-test at the 0.05 level of significance.

Table 2. Comparison of algorithm performance at 95 percent detection rate for RD and NRD conditions.

Traffic Category	Sample Size	Algorithm No. 7	8	10	16-14	Apparent Best Algorithm	Statistically Best Algorithm (for MTTD)
RD-1	28						
FAR, %		0.0449	0.0449	0.0336	0.112	10	
MTTD, min		2.96	2.69	3.18	1.26	16-14	16-14[a,b]
SD, min		1.93	1.93	3.49	1.14		
RD-0	26						
FAR, %		0.0561	0.0673	0.0673	0.112	7	
MTTD, min		2.28	2.32	3.07	2.56	7	None[a,b]
SD, min		1.84	1.91	3.09	2.22		
NRD-1	8						
FAR, %		0.0	0.0	0.0	0.014	None	
MTTD, min		2.12	2.12	2.12	2.75	None	None[a,b]
SD, min		1.89	1.89	1.89	2.5		
NRD-0	24						
FAR, %		0.0	0.0047	0.0094	0.014	7	
MTTD, min		4.08	4.04	4.08	4.13	None	None[a,b]
SD, min		4.06	4.16	2.74	5.69		

[a]Kolmogorov-Smirnov test. [b]Mann-Whitney test.

of significance) was found between the algorithms at the 95 percent detection level for all the incident categories.

It seems, then, that the DR-FAR relationship is more representative of the difference among algorithms than the DR-MTTD relationship and should be the major criterion for selecting algorithms.

Based on the results in Table 1, algorithm 7 was the apparent best for the ALL, RD, NRD, and NRW categories at the 95 percent detection level, while algorithm 8 was the apparent best for the RW category at the same detection level.

Evaluation of the Effect of Lateral Detectorization on Algorithm Performance

In the design process of a freeway surveillance and control system there is always the question of a trade-off between the level of detectorization (longitudinal and lateral) and the gains in terms of control and incident-detection effectiveness.

The Chicago expressway system under surveillance uses full detector stations every 4.8 km (3 miles) and single-detector stations, usually on lane 2 (lane 1 being the inner lane), every 0.8 km (0.5 mile). The analysis presented in this section compares the performance of algorithms 7, 8, 10, and 16-14 as related to incidents occurring on the detector lane (DL) versus those occurring on the nondetector lanes (NDL) under RD and NRD conditions. The results suggest that for both conditions the optimal thresholds obtained for incidents occurring on DL are less sensitive to discontinuities in traffic flow, as expressed in lower FAR, than those obtained for incidents occurring on NDLs.

This is explained by the fact that, generally, incidents occurring on DL have higher feature values that require less sensitive thresholds, which lower FAR. Incidents occurring on NDL have a somewhat attenuated impact when measured off another lane; this requires more sensitive thresholds (lower value) and risks a high FAR.

For the RD category, the relationship between the DR and MTTD is more favorable for incidents occurring on NDLs than for those occurring on DL. This trend could be explained by the fact that FAR increases with DR, while MTTD decreases with DR, which yields a decrease in MTTD with an increase in FAR. Thus, for a certain DR, the FAR on the DL is higher than the one experienced on NDL, which yields a higher MTTD. This, however, is not the case for the NRD category. The reason could be the small sample of incidents (eight) occurring on DL in the NR category.

In order to find out whether there was a statistically significant difference between the MTTD for incidents on DL and for those on the NDL for both RD and NRD categories, the Kolmogorov-Smirnov test was conducted (95 percent detection level). For RD and NRD categories, tests were made for algorithms 7 and 10, respectively, because each was the most efficient algorithm at that detection level. According to the Kolmogorov-Smirnov test, no significant differences between MTTD were

Table 3. Comparison of algorithm performance at 95 percent detection rate for RD conditions.

Traffic Category	Sample Size	Algorithm No.			
		7	8	10	16-14
RD-50-1	18				
FAR, %		0.0225	0.0225	0.0562	0.0337
MTTD, min		4.94	4.83	2.05	2.77
RD-50-0	12				
FAR, %		0.0562	0.0562	0.0786	0.1123
MTTD, min		2.92	3.83	3.08	2.41
RD-46-1	9				
FAR, %		0.0562	0.0562	0.0562	0.1123
MTTD, min		2.11	2.22	2.11	1.89
RD-46-0[a]	14				
FAR, %		0.1123	0.0786	0.0562	0.1235
MTTD, min		0.93	1.35	3.21	2.92

[a]This was the only category that displayed a significant difference.

Table 4. Effect of incident severity on algorithm performance.

Traffic Category	Sample Size	Algorithm No.			
		7	8	10	16-14
RD-46	23				
FAR, %		0.056	0.078	0.078	0.112
MTTD, min		2.31	2.36	2.36	2.27
RD-50	30				
FAR, %		0.056	0.078	0.078	0.112
MTTD, min		2.17	2.53	2.53	1.59
RD-46-1	9				
FAR, %		0.045	0.045	0.045	0.112
MTTD, min		3.34	3.44	2.89	1.89
RD-50-1	18				
FAR, %		0.022	0.056	0.045	0.033
MTTD, min		4.94	2.05	3.22	2.77
RD-46-0	14				
FAR, %		0.112	0.078	0.056	0.123
MTTD, min		0.93	1.35	3.21	2.92
RD-50-0	12				
FAR, %		0.056	0.056	0.078	0.112
MTTD, min		2.92	3.83	3.08	2.41

found for RD and NRD categories at the 0.10 level of significance.

The above analyses suggest that the relation between DR and FAR is more critical than that between DR and MTTD.

As to the relative performance of the individual algorithms within the various incident data categories, Table 2 presents, for the 95 percent level of detection, the MTTD, the standard deviation of the detection time, and the FAR for algorithms 7, 8, 10, and 16-14 and for the incident data categories RD-1, RD-0, and NRD-0. The Kolmogorov-Smirnov and Mann-Whitney tests were conducted for significant differences in MTTD. The results of these tests are also presented in terms of the statistically best algorithm compared with the apparent best. According to these tests, no single algorithm proved to be superior to the others with respect to MTTD for the RD-0, NRD-1, and NRD-0 categories. Algorithm 16-14, however, proved to be the best for the RD-1 category. Considering FAR, algorithm 10 seemed to be the best for the RD-1 category, while algorithm 7 excelled in the RD-0 and NRD-0 categories. No apparent best algorithm was found for the NRD-1 category.

Additional analysis was made of the differences in FAR and MTTD for accident and nonaccident incidents (AI and NAI) occurring on both DL (50-1, 46-1) and NDL (50-0, 46-0). Optimal thresholds were obtained for each particular situation. The analysis included tests for significant differences in MTTD among and within the RD for algorithms 7, 8, 10, and 16-14 at the 95 percent detection level using the Kolmogorov-Smirnov and Mann-Whitney tests at the 0.05 level of significance. The results of this analysis are presented in Table 3.

From this table it can be seen that, as far as MTTD was concerned, there was no significant difference for AI and NAI that occurred on either DL or NDL for each of the tested algorithms. Also, no significant differences in MTTD were found among algorithms within the categories RD-50-1, RD-50-0, and RD-46-1. Algorithm 7, however, was found to be the best within the RD-46-0 category.

As far as FAR was concerned, thresholds that were developed for AI and NAI occurring on DL yielded equal or better results than thresholds developed for AI and NAI occurring on NDL for all the tested algorithms. This is to be expected, because thresholds for detecting incidents on DL could be less sensitive to discontinuities in traffic flow than thresholds for incidents on NDL.

With regard to the individual categories, algorithms 7 and 8 performed the best for RD-50-1, RD-50-0, and RD-46-1, whereas algorithm 10 excelled in the RD-46-1 category. The local algorithm 16-14 yielded relatively high FAR for all categories tested.

The above results indicate that MTTD, unlike the FAR, did not prove to be a major criterion in the selection of algorithms.

It seems that, in order to generate low FAR, thresholds developed for incidents on DL should be used even though the probability of incident occurrence is naturally higher on NDL than on DL. However, these less sensitive thresholds would reduce the rate of detection of incidents occurring on the NDL.

Evaluation of the Effect of Incident Severity on Algorithm Performance

One of the considerations in selecting a particular set of thresholds for the operation of a certain algorithm could be its relative effectiveness in detecting AI and NAI, which usually differ in their impact on traffic flow. As shown previously, thresholds for incidents occurring on DL are less sensitive in terms of FAR than those for incidents occurring on NDL. However, the effectiveness and efficacy of thresholds developed separately for AI and NAI are yet to be evaluated.

Table 4 presents a comparison of MTTD and FAR, at the 95 percent detection level, for algorithms 7, 8, 10, and 16-14, between AI and NAI occurring either on DL or NDL or on both. As can be seen from Table 4, as far as MTTD was concerned, the Kolmogorov-Smirnov and Mann-Whitney tests did not show any significant difference at the 0.05 level. As far as FAR was concerned, thresholds that were developed for the accident data performed better than those developed for the nonaccident data in all cases. This, of course, is predictable, because AI would have a greater disruptive impact on traffic flow than NAI would.

The question that remains to be answered concerns the effectiveness of thresholds developed for AI in detecting NAI. Analysis showed that thresholds developed for accident data on DL at the 95 percent detection level detected only 78 percent of NAI on that lane for algorithms 7 and 8 (FAR = 0.22 percent) and detected all NAI for algorithm 10 (FAR = 0.56 percent). It seems that, if FAR is the major criterion, then thresholds developed for accidents (RD-50-1) could be used to detect other incidents (RD-46-1). This also holds true for RD-46-0 and RD-50-0 for algorithms 7, 8, and 10.

Hierarchy Analysis of Threshold Effectiveness

The effort involved in developing the input necessary for

Table 5. Threshold hierarchy analysis.

	Algorithm No.								
	7			8			10		
Thresholds Compared	DR	FAR	MTTD	DR	FAR	MTTD	DR	FAR	MTTD
ALL on RD v. RD on RD	0.92	0.056	3.40	0.90	0.067	2.52	0.93	0.056	3.25
	0.96	0.056	2.23	0.96	0.078	2.75	0.95	0.067	2.88
RW on RD v. RD on RD	0.85	0.034	3.63	0.64	0.000	5.43	0.81	0.045	4.21
	0.96	0.056	2.23	0.96	0.078	2.75	0.95	0.067	2.88
NRD on RD v. RD on RD	0.92	0.056	3.36	0.83	0.045	2.86	0.87	0.056	2.21
	0.96	0.056	2.23	0.96	0.078	2.75	0.95	0.067	2.88
NRW on RD v. RD on	0.92	0.056	3.40	0.83	0.045	2.86	0.87	0.056	2.21
RD	0.96	0.056	2.23	0.96	0.078	2.75	0.95	0.067	2.88
ALL on RW v. RW on	1.00	0.056	2.33	1.00	0.067	2.33	1.0	0.056	2.50
RW	1.00	0.034	2.83	1.00	0.000	3.99	1.0	0.045	2.50
RD on RW v. RW on	1.00	0.056	2.16	1.00	0.078	2.16	1.0	0.067	2.50
RW	1.00	0.034	2.83	1.00	0.000	3.99	1.0	0.045	2.50
NRD on RW v. RW on	1.00	0.056	2.21	0.84	0.044	2.80	1.0	0.056	1.99
RW	1.00	0.034	2.83	1.00	0.000	3.99	1.0	0.045	2.50
NRW on RW v. RW on	1.00	0.056	2.33	0.84	0.044	2.80	1.0	0.056	1.99
RW	1.00	0.034	2.83	1.00	0.000	3.99	1.0	0.045	2.50
ALL on NRD v. NRD on	1.00	0.005	3.78	0.96	0.014	3.61	1.0	0.009	4.28
NRD	0.96	0.005	3.93	0.96	0.005	3.67	0.96	0.005	2.87
RD on NRD v. NRD on	1.00	0.014	3.46	1.00	0.023	3.46	0.93	0.019	4.23
NRD	0.96	0.005	3.93	0.96	0.005	3.67	0.96	0.005	2.87
RW on NRD v. NRD on	0.90	0.009	4.38	0.68	0.009	4.27	0.97	0.005	4.71
NRD	0.96	0.005	3.93	0.96	0.005	3.67	0.96	0.005	2.87
NRW on NRD v. NRD on	1.00	0.005	3.78	0.68	0.009	4.27	0.96	0.005	2.87
NRD	0.96	0.005	3.93	0.96	0.005	3.67	0.96	0.005	2.87
ALL on NRW v. NRW on	0.87	0.005	2.71	1.00	0.014	2.62	0.87	0.009	5.14
NRW	0.87	0.005	2.71	1.00	0.005	2.63	1.0	0.005	2.87
RD on NRW v. NRW on	1.0	0.014	2.50	1.00	0.023	2.50	1.0	0.019	6.00
NRW	0.87	0.005	2.71	1.00	0.005	2.63	1.0	0.005	2.50
RW on NRW v. NRW on	0.75	0.009	7.14	0.75	0.009	3.34	0.87	0.009	5.14
NRW	0.87	0.005	2.71	1.00	0.005	2.63	1.0	0.005	2.50
NRD on NRW v. NRW on	0.87	0.005	2.85	1.00	0.005	2.63	1.0	0.005	2.50
NRW	0.87	0.005	2.71	1.00	0.005	2.63	1.0	0.005	2.50
RD on RD-0 v. RD-0 on	0.96	0.056	2.47	0.96	0.078	2.04	0.92	0.067	3.04
RD-0	0.96	0.056	2.28	0.96	0.067	2.32	0.96	0.067	3.07
RD-1 on RD-0 v. RD-0	0.77	0.045	2.45	0.77	0.045	3.15	0.69	0.033	3.72
on RD-0	0.96	0.056	2.28	0.96	0.067	2.32	0.96	0.067	3.07
RD on RD-1 v. RD-1 on	0.96	0.056	1.99	0.96	0.078	1.81	0.96	0.067	2.74
RD-1	0.96	0.045	2.96	0.96	0.045	3.77	0.96	0.033	3.18
RD-0 on RD-1 v. RD-1	0.96	0.056	1.84	0.96	0.067	1.88	0.96	0.067	2.77
on RD-1	0.96	0.045	2.96	0.96	0.045	3.77	0.96	0.033	3.18
RD on RD-46 v. RD-46	0.96	0.056	2.31	0.96	0.078	2.05	0.96	0.067	2.86
on RD-46	0.96	0.056	2.31	0.96	0.078	2.18	0.96	0.078	2.36
RD-50 on RD-46 v.	0.96	0.056	2.31	0.87	0.067	2.44	0.91	0.078	2.56
RD-46 on RD-46	0.96	0.056	2.31	0.96	0.078	2.18	0.96	0.078	2.36
RD on RD-50 v. RD-50	0.97	0.056	2.17	0.97	0.078	1.83	0.93	0.067	2.89
on RD-50	0.97	0.056	2.17	0.97	0.067	2.56	0.97	0.078	2.53
RD-46 on RD-50 v.	0.97	0.056	2.17	0.97	0.078	2.03	0.94	0.078	2.03
RD-50 on RD-50	0.97	0.056	2.17	0.97	0.067	2.56	0.97	0.078	2.53
RD-0 on RD-46-0 v.	0.93	0.056	1.84	0.93	0.067	2.00	1.0	0.067	2.43
RD-46-0 on RD-46-0	0.93	0.056	1.84	1.0	0.078	1.35	1.0	0.056	3.21
RD-46 on RD-46-0 v.	1.0	0.078	1.35	0.93	0.078	1.77	0.93	0.078	2.00
RD-46-0 on RD-46-0	0.93	0.056	1.84	1.0	0.078	1.35	1.0	0.056	3.21
RD-1 on RD-46-1 v.	1.0	0.045	3.34	1.0	0.045	3.44	1.0	0.045	2.89
RD-46-1 on RD-46-1	1.0	0.045	3.34	1.0	0.045	3.44	1.0	0.045	2.89
RD-46 on RD-46-1 v.	1.0	0.056	2.11	1.0	0.078	2.78	1.0	0.078	2.89
RD-46-1 on RD-46-1	1.0	0.045	3.34	1.0	0.045	3.44	1.0	0.045	2.89
RD-0 on RD-50-0 v.	1.0	0.056	2.65	1.0	0.067	2.67	0.92	0.067	3.30
RD-50-0 on RD-50-0	1.0	0.056	2.92	1.0	0.056	3.83	1.0	0.078	3.08
RD-50 on RD-50-0 v.	1.0	0.078	2.58	1.0	0.067	3.42	1.0	0.078	3.08
RD-50-0 on RD-50-0	1.0	0.056	2.92	1.0	0.056	3.83	1.0	0.078	3.08
RD-1 on RD-50-1 v.	0.95	0.045	2.76	0.95	0.045	2.85	0.95	0.045	3.22
RD-50-1 on RD-50-1	0.95	0.022	4.94	0.95	0.022	4.83	0.95	0.045	3.22
RD-50 on RD-50-1 v.	0.95	0.078	1.49	0.95	0.067	1.77	0.95	0.078	2.11
RD-50-1 on RD-50-1	0.95	0.022	4.94	0.95	0.022	4.83	0.95	0.045	3.22

an optimal on-line incident-detection system could be enormous. Part of this effort lies in developing thresholds appropriate for various environmental, geometric, and traffic conditions. In addition, for freeway systems that have low levels of detectorization, the question exists as to whether thresholds representing AI or NAI on either DL or NDL should be used.

This section evaluates the efficiency, in terms of DR, FAR, and MTTD, of applying lower-level thresholds to higher-level incident data categories (i.e., thresholds developed for the ALL category are tested on the RD category). The object of such an analysis is to investigate the possibilities of reducing the amount of effort required to develop the optimal sets of thresholds.

The thresholds for each lower-level incident category were obtained for the 95 percent nominal DR and were applied to a higher-level incident category to yield appropriate values for the other measures of effectiveness. The Mann-Whitney V-test was applied to establish the significance of the difference between MTTD of each two compared incident categories. Table 5 presents the results of this analysis.

As it can be seen from this table, thresholds developed for ALL could be used during the RW period by all algorithms. On the other hand, when used during the RD period, the ALL thresholds yielded reduced DR (algorithms 7, 8, and 10) and also equal or reduced FAR. As far as the MTTD was concerned, the ALL thresholds yielded larger values, which were significantly different, however, for algorithm 10 only. It was also indicated that during the NRD period, as well as during the NRW period, the ALL thresholds could be used quite effectively in algorithms 7 and 10.

The RD thresholds were found to be generally inferior in terms of FAR to those developed for ALL when they were used during the RW, NRD, and NRW periods.

Thresholds developed for the RD category were applied to both the RD-1 and RD-0 categories. In both cases these thresholds were found to be inferior to the thresholds representing the two categories. When RD-1 thresholds were applied to the RD-0 category, FAR improved but DR decreased for all algorithms. When RD-1 thresholds were applied to the RD-50-1 category, there was no change in DR and no significant difference in MTTD. Other threshold hierarchy relations could be easily obtained from Table 5. The few significant differences that appeared in the threshold comparisons are shown below.

Thresholds Compared	Significant Difference (algorithm no.)
RW on RD v. RD on RD	7, 8
RD on NRW v. NRW on NRW	10
RD-50 on RD-50-1 v. RD-50-1 on RD-50-1	7, 10

FINDINGS, OBSERVATIONS, AND RECOMMENDATIONS

From the data collected and the various analyses, the following are the major findings.

1. Algorithm 9, which was found to yield favorable results in previous studies (1), displayed a poor DR-FAR relationship compared with algorithms 7, 8, and 10.
2. For the RD period and for detection levels lower than 95 percent, no best algorithm with respect to FAR could be found.
3. For detection levels of 95 percent and above, algorithm 7 was found to have the lowest FAR for the RD period.
4. No significant differences in MTTD among algorithms were found at the 95 percent detection level at the 0.05 level of significance.
5. In order to be detected, incidents that occurred on DL required less sensitive thresholds than those on NDL.
6. For the most efficient algorithms, 7 and 10, for RD and NRD, respectively, no significant difference in MTTD was found for the 95 percent detection level at the 0.10 level of significance.
7. For incidents occurring on DL and NDL during the RD period, algorithms 10 and 7, respectively, were found to be the most efficient as far as FAR was concerned at the 95 percent detection level.
8. No significant differences in MTTD among algorithms were found to exist for AI and NAI on either DL or NDL, at the 95 percent detection level for the RD category.
9. At the 95 percent detection level, thresholds developed for AI and NAI on DL are less sensitive to false alarms than those developed for the above incident data on NDL for all algorithms during the RD period.
10. Thresholds developed for AI yielded lower FAR than thresholds developed for NAI for both DL and NDL at the 95 percent detection level and for the RD period.
11. Thresholds developed at the 95 percent detection level for AI occurring on DL detected only 78 percent of the NAI on that lane, for algorithms 7 and 8, and all such incidents for algorithm 10.
12. Thresholds developed at the 95 percent detection level for a representative sample of incidents (ALL) could effectively be used during RW, NRD, and NRW periods.

Based on the major findings of this study, the following observations could be made.

1. MTTD should not be a critical criterion for selecting an operational algorithm because no significant differences in this parameter were found among the tested algorithms for desired detection levels.
2. The DR-FAR relationship should be a critical criterion in the process of selecting incident-detection algorithms.
3. On the whole, algorithm 7 seemed to yield the most favorable results of all the algorithms tested in this study.
4. Thresholds developed for accidents on DL could be used to guarantee the lowest FAR.
5. The level of lateral detectorization is not a critical issue as far as detection time for incidents on various lanes is concerned.
6. If a high level of lateral detectorization (fully detectorized lanes) exists, algorithms should be applied to each lane in the detection process to yield low FAR and high DR.
7. The effort in developing thresholds for the RW, NRD, and NRW periods could be avoided by using thresholds developed for a representative sample of incidents (ALL).
8. Complicated algorithms are not necessarily the best ones.

The following recommendations are made.

1. Conduct an on-line evaluation of the above algorithms.
2. Conduct a discriminant analysis of traffic features to find the best combination of features to be used in an algorithm.
3. Develop algorithms based on speed-related features.
4. Investigate traffic-feature characteristics in bottlenecks during incidents to improve detection and false-alarm rates.
5. Because there are some differences between the results of this study and those of TSC, evaluating other non-pattern-recognition algorithms with the above data ought to be considered.

ACKNOWLEDGMENT

The research reported in this paper was conducted within the framework of IDOT and with the cooperation of FHWA. We wish to acknowledge the support of the staffs of the Bureau of Air Operations and the Traffic Systems Center in the data-collection and programming phases of this study. The contents of this report reflect our views, and we alone are responsible for the facts and the accuracy of the data presented here. The contents do not necessarily reflect the official views or policies of IDOT or FHWA. This report does not constitute a standard, specification, or regulation.

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Part 2. On-Line Evaluation

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Five algorithms were evaluated on-line by using the facilities of the Traffic Systems Center of the Illinois Department of Transportation. Three of the algorithms developed by Technology Services Corporation (TSC), were of a pattern-recognition nature. The other two—a pattern-recognition and a probabilistic or Bayesian algorithm—were developed locally. Thresholds for the features used in each of the pattern-recognition algorithms were developed by TSC. The thresholds for the probabilistic algorithm were developed by using accident data on the Eisenhower Expressway. The measures of effectiveness in the evaluation were detection rate, false-alarm rate, and mean-time-to-detect. The three TSC algorithms were evaluated twice on the Eisenhower Expressway at the 80 and 90 percent levels of detection thresholds, and then problem areas showing high false-alarm rates were represented by the 50 percent level. The three TSC algorithms were then evaluated on a section of the Dan Ryan Expressway that was free of geometric problems, for comparison purposes. Statistical analysis showed no difference in detection rate, false-alarm rate, and mean-time-to-detect among the three TSC algorithms at any of the evaluated detection levels. Introduction of the 50 percent level improved certain measures of effectiveness. Algorithm 7, the best of the TSC algorithms, showed overall superiority to the two local algorithms. The false-alarm rate was shown to be related to geometric and other features of the problem areas and yielded algorithm 8, which uses a shockwave-suppressor mechanism and requires the least effort in developing appropriate thresholds.

This paper discusses the on-line evaluation of five incident-detection algorithms that were all evaluated off-line in the preceding paper to obtain the optimal threshold sets used in the on-line evaluation.

The specific goals of this research were

1. To determine the on-line efficiency of algorithms proved effective in the off-line evaluation,
2. To correlate algorithm efficiency parameters derived from the on-line evaluation with those derived from the off-line evaluation, and
3. To evaluate combinations of thresholds with respect to geometric conditions on the freeway.

ALGORITHM DESCRIPTION

Consider an n-lane freeway section of length L between two fully detectorized stations. At each station a set of flow characteristics for occupancy, volume, and speed is measured at specific time intervals.

Suppose that at time t_0 an incident occurs at a certain point on one of the lanes in section L. A shock wave will develop and travel upstream of the incident with an intensity that is dictated by the severity and lateral location of the incident and by environmental and geometric conditions. At time $t_0 + dt$ an incident-detection algorithm, by continuously measuring and comparing the flow characteristics upstream and downstream of the incident with predetermined thresholds, will detect the incident.

This section describes the structure of the incident-detection algorithms evaluated in this research. Of the five algorithms evaluated, three of the pattern-recognition type were developed by TSC (2) and the other two, one pattern-recognition and one probabilistic (7), were developed locally in the course of this research.

The research effort of TSC included the development of 10 incident-detection algorithms that could be grouped into three categories. The first, comprising algorithms 1-7, is composed of variations on the classic California algorithm (2). The second consists of algorithms 8 and 9, which are characterized by suppression of incident detection after a compression wave is detected. Finally, algorithm 10 represents an attempt to detect those incidents that occur in light-to-moderate traffic but do not lower capacity below the volume of oncoming traffic.

Of these 10 algorithms 3 were selected for evaluation, 1 from each category. The algorithms selected (7, 8, and 10) were chosen for a number of reasons. Preliminary investigation by TSC had indicated algorithm 7 to be a superior form of the California algorithm. Algorithm 8 is identical to algorithm 9 except for an added persistence check. According to TSC's preliminary investigation, algorithm 8 has a slightly lower FAR but a longer MTTD than algorithm 9. Although algorithm 10 did not perform especially well in TSC's view, it was included in the on-line evaluation because it represents a first attempt to solve the problem of detecting incidents that do not produce marked traffic-flow discontinuities.

The TSC algorithms are in binary decision-tree form; at each node of the decision tree a feature value is compared with a user-specified threshold value to determine whether an incident is to be signaled. Clearly, the effectiveness of the algorithm depends on the thresholds chosen. The program TSC developed for optimizing threshold selection has been described in the first part of this paper. It uses a random-number generator that produces increments to be added to the current optimal threshold vector to produce a new threshold vector for evaluation. After a predetermined number of iterations, and given a certain level of detection, the threshold vector that has the lowest FAR is termed the optimal threshold vector at that level.

The thresholds obtained by using the CALB program for the evaluated detection levels were used in this analysis.

Also evaluated, in addition to the above three TSC algorithms, were 16-14, the pattern-recognition algorithm, and the Bayesian or probabilistic algorithm, both developed locally. Threshold selection for algorithm 16-14 was accomplished by using the CALB program; calibration of the Bayesian algorithm used accident data collected on the Eisenhower Expressway.

The meaning of the features involved in each algorithm and the tree structures of these algorithms are given in Part 1 of this paper.

ON-LINE INCIDENT-DETECTION SYSTEM

TSC controls 360 directional km (224 miles) of expressways through its Freeway Traffic Management System (Figure 6). The backbone of the system is the detector subsystem that uses full detector stations [5 km (3 miles)] and single-detector stations [800 m (0.5 mile)].

The major function of the on-line incident-detection system is to detect a capacity-reducing incident through its incident-detection logic, which uses three algorithms simultaneously and then delivers a message to the monitor. Another function is to provide continuous evaluation of algorithm performance, refinement of thresholds, and evaluation of response to incidents. Figure 7 presents the basic on-line incident-detection system.

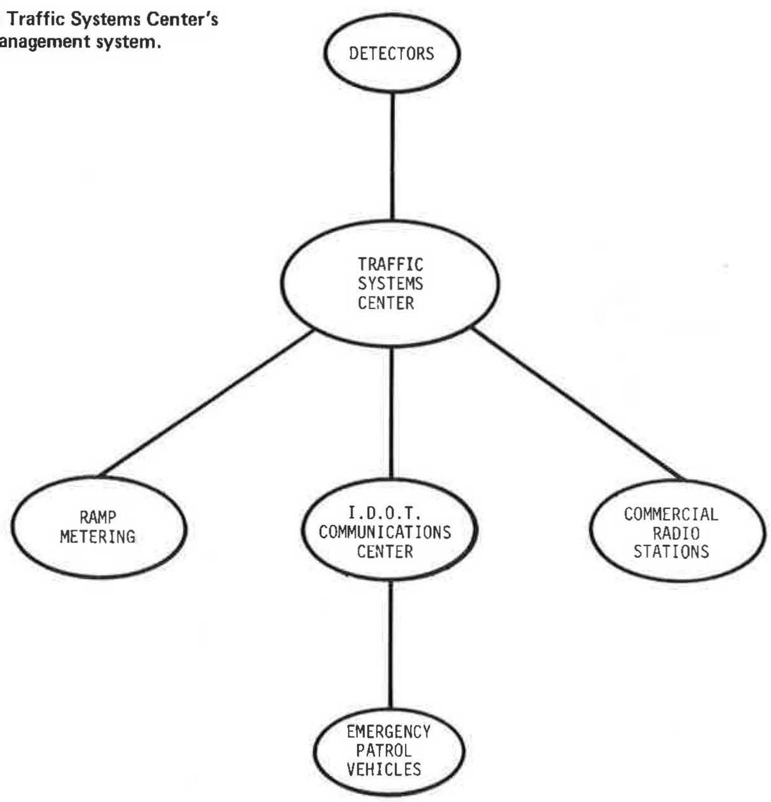
The basic programs for both functions of the on-line system are the incident-determination logic program (ST) and the incident message program (S). The former uses appropriate thresholds obtained from previous analyses to determine the incident status of each of the main-line detectors. A status matrix is used for recording the status and is updated every minute. At the end of the update, S scans the matrix for detected incidents and generates an appropriate message. The generated messages include information on detector subsection, upstream occupancy, downstream occupancy, time of incident, day, and date and are maintained in a disk-based file.

Once the incident message is produced it becomes possible to monitor the incident file through the display as part of the incident message management phase, which the display program (E) directs. Appropriate parameters are passed into programs Q and O for operation. Program Q controls the queuing and the displaying of the incident messages. Queue manipulation enables the operator to inspect the incident file and delete old messages, because new messages are ignored when the queue is full. These messages consist of six elements. Three describe location: expressway name, direction [inbound (IB) or outbound (OB)], and detector station; the others are vector number, incident file number, and earliest detection time.

Program O can handle various options initiated by the operator. In the future, these options could include communications between the Traffic Systems Center in Oak Park and the IDOT Communication Center in Schaumburg.

Other related programs in the on-line software are

Figure 6. IDOT's Traffic Systems Center's freeway traffic management system.



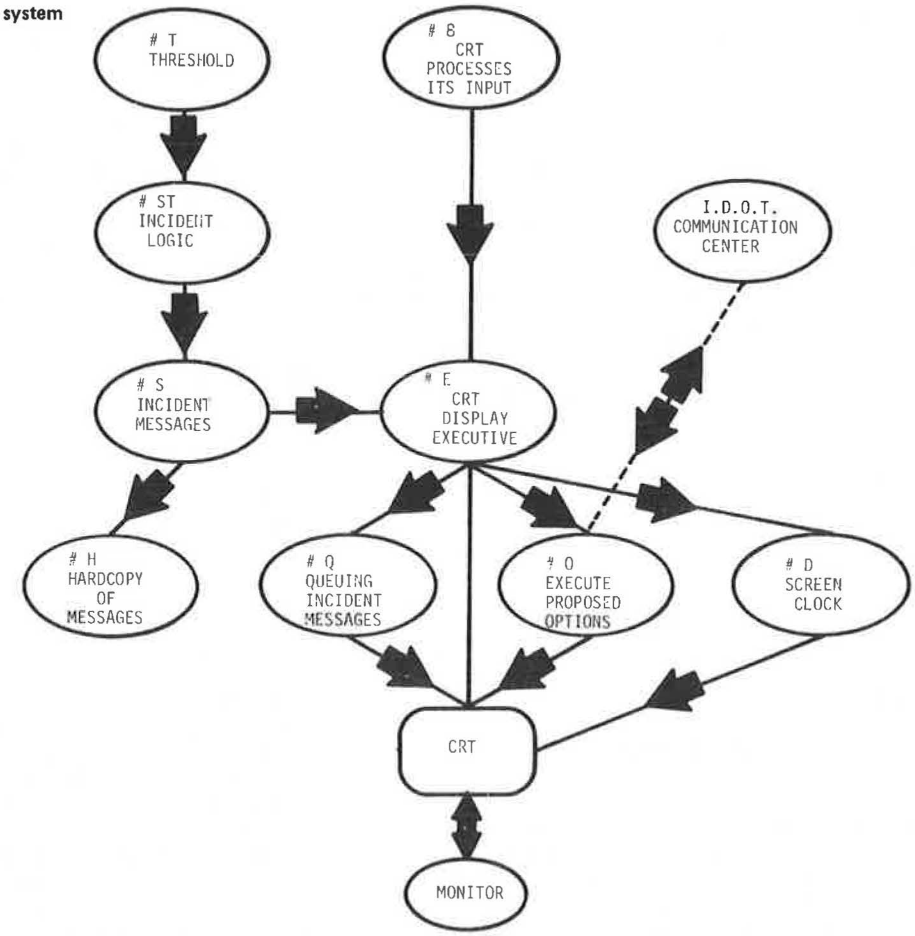


Figure 7. On-line incident-detection system software.

program H, which produces a hard copy of the incident file; program D, which records the time it takes the operator to respond to the message and displays the clock time on the screen; and program 8, which is an existing program extended to include the input required by the display program (E).

DEVELOPMENT OF DATA BASE

The major site chosen for the study was the Eisenhower Expressway (I-90) between I-94 and Wolf Road (Figure 8). This expressway contains various characteristics along its 8-km (13-mile) length. Geometrically, the expressway is four lanes wide between I-94 and Austin Boulevard and then drops to three lanes from Austin to Wolf Road. This lane drop is the major bottleneck area for westbound traffic. For eastbound traffic, First Avenue is the major problem area. Here the degree of curvature, change in grade, and volumes of traffic are the main causes of congestion. Both sections are quite a challenge for the on-line incident-detection algorithms, especially during peak hours. For comparison purposes, another expressway (the Dan Ryan between 65th and 95th Streets) was chosen for study. This section of expressway is a straight section, four lanes wide, with no major bottlenecks between its terminal points.

The time period picked for the survey was 3:00-5:00 p.m. Monday-Friday. During this period, four capacity-reducing incidents are expected on the Eisenhower Expressway.

A helicopter aerial survey of the study section was made to collect the incident data. The information obtained for each stopped vehicle included time of spotting, longitudinal location (IB or OB), lateral location (a cross street), lane, vehicle description, reason for stopping (if ascertainable), type of aid present (if any), and comments to describe or explain traffic operations.

The helicopter was able to maintain an average speed of 180 km/h (110 mph), which allowed one trip along the entire length of expressway, i.e., terminal points of the study, to be made in about 7.5 min. In reality, however, each point was viewed nearly every 5 min because of the visibility from the helicopter flying at about 200-250 m (700-800 ft) above the expressway.

At the completion of each day of data collection, the aerial survey data were correlated with the incident information produced by the on-line operating algorithms. This recorded information included longitudinal location, lateral location, lane, detection time of each individual algorithm being tested, termination time, computer and actual duration times, type of incident or congestion-causing situation, comments, and actual time of occurrence, detection, and termination.

After completing this correlation of computer-recorded incident messages and actual recorded incidents, various statistics were determined. These were DR, FAR, missed incidents, and so forth, calculated for each day for each individual algorithm.

A total of 29 days of data on the Eisenhower Expressway and 4 days on the Dan Ryan Expressway were collected.

ALGORITHM EVALUATION

Based on the off-line evaluation of the algorithms it was decided to conduct the on-line evaluation by using optimal

Figure 8. On-line study site on the Eisenhower Expressway.

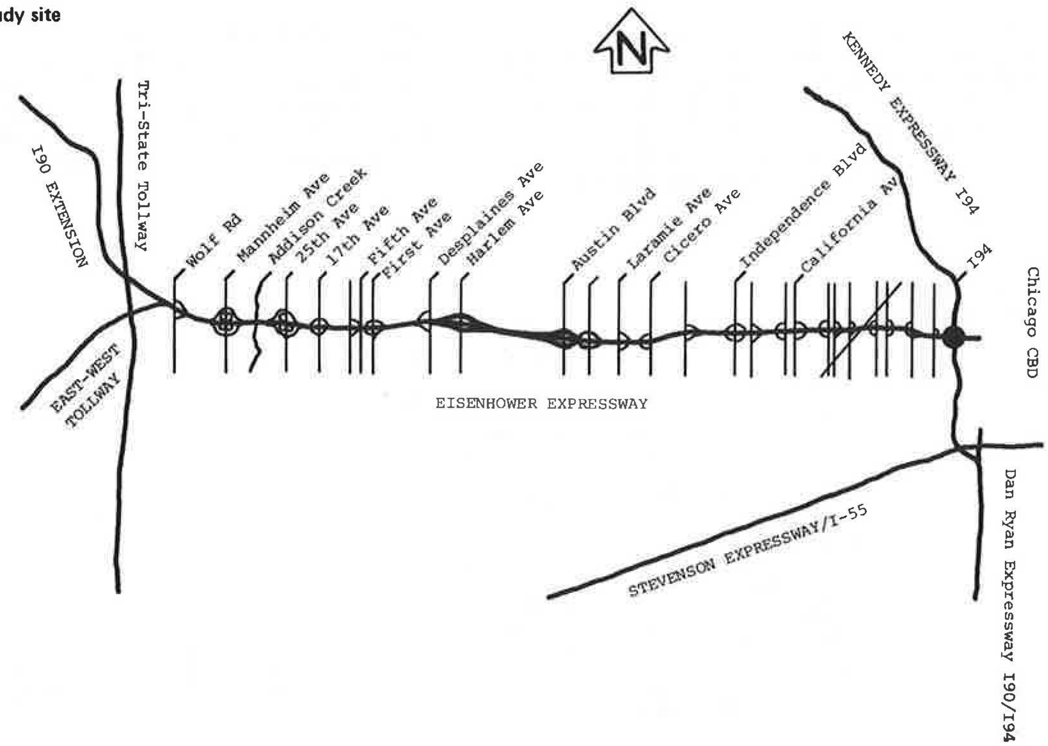


Table 6. Summary of on-line evaluation process.

Study Case	Facility	Off-Line DR (%)	Algorithms Evaluated	No. of Data Days
1	Eisenhower	80	7, 8, 10	11
2	Eisenhower	90	7, 8, 10	10
3	Eisenhower	90-50	7, 8, 10	4
4	Ryan	90	7, 8, 10	4
5	Eisenhower	90-50	7, 16-14, Bayesian	9

thresholds developed for the 80 and 90 percent DRs as obtained in that evaluation.

In the first phase, algorithms 7, 8, and 10 were evaluated on the Eisenhower Expressway during the afternoon rush. Preliminary analysis of the data suggested that problem areas (bottlenecks and curves) were producing a considerable number of FARs, and it was decided to run an evaluation after having introduced less-sensitive thresholds—the off-line 50 percent DR—into the problem areas. Then an evaluation of the algorithms on the Dan Ryan study section was conducted with thresholds representing the off-line 90 percent DR.

In the second phase, the apparent best algorithm among the three above was selected to operate simultaneously with algorithm 16-14 and the Bayesian algorithm on the Eisenhower Expressway. Each of the study cases referred to in Table 6 was analyzed for differences in DR, FAR, and MTTD among the algorithms. Algorithm efficiency at the 80 percent detection level was compared with that at the 90 percent level, and the efficiency at that level was compared with algorithm efficiency at the 90-50 percent detection level, which was represented by a set of thresholds derived for the 90 percent and 50 percent detection levels at nonproblem and problem sections, respectively.

The most promising algorithms at the detection levels of 90 percent and 90-50 percent were selected for further analysis. In this analysis the cumulative distributions of the message duration of false alarms and real incidents were compared in order to give an indication as to the change with time of the probability that an incident message is true. Also, the distribution of false alarms with respect to time during the rush period was investigated to yield an indication of the need for threshold refinement.

To clarify the relationship between numbers of false alarms and geometric features of the problem section, an analysis was conducted at the 90 and 90-50 percent levels of detection. In this analysis the number of false alarms for each problem section for one detection level was compared with that for the other detection level. This was done for algorithms 7, 8, and 10.

Tables 7 and 8 present the types of problems on the various sections of inbound and outbound Eisenhower. These problems had a tendency to produce a high number of false alarms. The sections that were operating with thresholds related to the 50 percent detection level during the 90-50 percent detection level evaluation period are also indicated. No attempt was made to find the relation between DR and the geometric features of each section because of the relatively low number of incidents (16) during the 90-50 percent detection-level evaluation period.

Comparative Analysis of Algorithm Efficiency

Tables 9, 10, and 11 present DR, FAR, and MTTD for algorithms 7, 8, and 10 for the off-line detection levels of 80, 90, and 90-50 percent, respectively.

As can be seen from these tables, the on-line DRs are lower than the off-line rates. However, the positive correlation between DR and FAR, which was found in the off-line analysis, seems to exist in the on-line analysis, as shown for the off-line 80 and 90 percent detection levels in Tables 9 and 10, respectively.

The statistical t-tests conducted for each off-line detection level for differences in the measures of effectiveness among the algorithms did not indicate any significant (0.05 level) differences for any of the measures of effectiveness for any of the detection levels. Differences in MTTD values between the off-line and on-line evaluations were also noted. The on-line evaluation yielded MTTD values ranging between 2.7 and 8.9 min for thresholds representing the 90-50 percent detection level. The off-line evaluation yielded MTTD values ranging from 2 to 4 min. The large MTTD values obtained in the on-line evaluation could be attributed to some inherent inaccuracies in determining the exact time of occurrence of an incident because of the obvious limitations of the aerial survey. Taking this into consideration, as far as the MTTD was concerned, both the

Table 7. Relation between FAR and geometric features on IB Eisenhower Expressway.

		90 Percent Threshold			90-50 Percent Threshold		
		Algorithm No.			Algorithm No.		
IB Eisenhower Section	Problem Description	7	8	10	7	8	10
Wolf to Mannheim	Horizontal curve (downgrade)	1	-	-	-	-	-
Mannheim to Addison Creek	-	-	-	2	-	-	-
Addison Creek to 25th Street	Bridge effect[a] (upgrade)	4	-	3	-	-	-
25th Street to 17th Street	Horizontal curve	-	-	-	-	-	-
17th Street to 5th Avenue	-	1	-	1	1	1	1
5th Avenue to 1st Avenue	Double merge	1	-	1	-	-	2
1st Avenue to Desplaines	Horizontal curve	-	-	-	-	-	-
Desplaines to Harlem	-	-	-	-	1	2	2
Harlem to Austin	Upgrade[a]	2	1	2	1	1	1
Austin to Laramie	Vertical curve	-	-	-	-	-	-
Laramie to Cicero	-	-	-	-	-	-	-
Cicero to Independence	Horizontal curve	-	-	-	-	-	-
Independence to California	Close bridges effect	2	2	2	1	1	3
California to I-94	-	-	-	-	-	-	-
Total		11	3	11	4	5	9

[a] Threshold for 50 percent DR used.

Table 8. Relation between FAR and geometric features on OB Eisenhower Expressway.

		90 Percent Threshold			90-50 Percent Threshold		
		Algorithm No.			Algorithm No.		
OB Eisenhower Section	Problem Description	7	8	10	7	8	10
I-94 to California	-	1	1	-	-	-	-
California to Independence	Close bridges effect[a] (downgrade)	-	-	-	-	-	1
Independence to Cicero	Horizontal curve sun effect	-	-	-	1	-	-
Cicero to Laramie	-	-	-	-	-	-	1
Laramie to Austin	Vertical curve	1	1	1	1	1	1
Austin to Harlem	Lane drop[a]	2	1	3	4	1	4
Harlem to Desplaines	-	1	1	1	-	-	-
Desplaines to 1st Avenue	Horizontal curve sun effect[a]	-	-	1	1	1	1
1st Avenue to 5th Avenue	Sun effect[a]	4	2	2	-	-	-
5th Avenue to 17th Street	-	-	-	-	-	-	-
17th Street to 25th Street	Horizontal curve[a]	-	-	-	-	-	-
25th Street to Addison Creek	Bridge effect[a] (downgrade)	2	1	1	1	3	2
Addison Creek to Mannheim	-	-	-	-	-	-	-
Mannheim to Wolf	Horizontal curve (upgrade)	-	-	-	-	-	-
Total	Total	11	7	9	8	6	10

[a] Threshold for 50 percent DR used.

Table 9. On-line algorithm efficiency for off-line 80 percent detection level for RD conditions on Eisenhower Expressway.

Measure of Effectiveness	Algorithm No. 7	8	10	Apparent Best Algorithm	Statistically[a] Best Algorithm
DR	0.28	0.25	0.26	8	None
FAR	0.87	0.70	0.82	8	None
MTTD, min	8.8	9.3	9.0	7	None

[a] At the 0.05 level of significance.

Table 10. On-line algorithm efficiency for off-line 90 percent detection level for RD conditions on Eisenhower Expressway.

Measure of Effectiveness	Algorithm No. 7	8	10	Apparent Best Algorithm	Statistically[a] Best Algorithm
DR	0.37	0.36	0.34	7	None
FAR	0.86	0.73	0.80	8	None
MTTD, min	8.9	6.3	2.7	10	None

[a] At the 0.05 level of significance.

Table 11. On-line algorithm efficiency for off-line 90-50 percent detection level for RD conditions on Eisenhower Expressway.

Measure of Effectiveness	Algorithm No. 7	8	10	Apparent Best Algorithm	Statistically[a] Best Algorithm
DR	0.56	0.41	0.56	7, 10	None
FAR	0.63	0.74	0.73	7	None
MTTD, min	7.5	5.3	6.2	8	None

[a] At the 0.05 level of significance.

Table 12. On-line algorithm efficiency for off-line 90 percent detection level for RD conditions on Dan Ryan Expressway.

Measure of Effectiveness	Algorithm No. 7	8	10	Apparent Best Algorithm	Statistically Best Algorithm
DR	0.75	0.75	0.75	All	All
FAR	0.58	0.25	0.50	8	None
MTTD, min	10.0	11.0	13.5	7	None

Table 13. On-line algorithm efficiency for off-line 90-50 percent detection level for RD conditions, for algorithms 7, 16-14, and Bayesian, on Eisenhower Expressway.

Measure of Effectiveness	Algorithm No. 7	16-14	Bayesian	Apparent Best Algorithm	Statistically Best Algorithm
DR	0.60	0.71	0.53	16-14	None
FAR	0.71	0.88	0.77	7	7, Bayesian
MTTD, min	8.02	6.08	12.14	16-14	7, 16-14
No. of false alarms	3.4	15.7	4.4	7	7, Bayesian

Figure 9. Cumulative distributions of the duration of incidents and FAR for algorithm 7 at the 90 percent detection level.

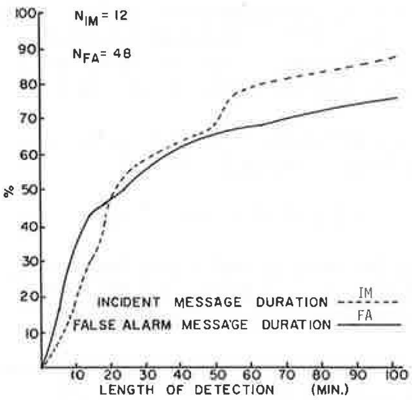
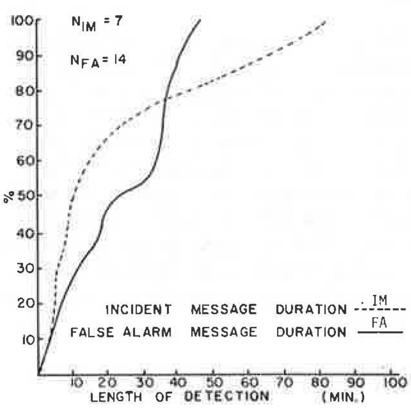


Figure 10. Cumulative distributions of the duration of incidents and FAR for algorithm 7 at the 90-50 percent detection levels.



on-line and the off-line evaluations presumably gave the same results.

A statistical comparison of algorithm efficiency with the 90 percent detection-level thresholds with that with the 90-50 percent detection-level thresholds was carried out at the 0.05 level of significance. It was found that introduction of 50 percent detection-level thresholds into problem areas improved algorithm 7's performance in terms of DR and FAR, but not MTTD. For algorithm 8, the introduction of the problem-section-related thresholds did not statistically improve any of the measures of effectiveness. In the case of algorithm 10, such analysis indicated significant differences for DR and MTTD but not for FAR.

Comparing the efficiency of each of the above three algorithms at the 80 percent detection level with that at the 90 percent detection level showed no significant differences for any of the measures of effectiveness for algorithms 7 and 8. For algorithm 10, however, there were no significant differences in DR and MTTD but there was one in FAR.

The results of the limited algorithm evaluation on the Dan Ryan Expressway at the 90 percent detection level are presented in Table 12. Statistical analysis at the 0.05 level of significance for differences among algorithms 7, 8, and 10 indicated no significant differences for any of the measures of effectiveness.

During the second phase of the study algorithm 7, which was found to be the apparent best for the 90-50 percent detection level, was compared with algorithm 16-14 and the Bayesian algorithm. Table 13 presents the results of this evaluation. Statistical analysis at the 0.05 level of significance indicated that, as far as the detection rate was concerned, no best algorithm could be found. Algorithm 7 and the Bayesian algorithm were superior to algorithm 16-14 with respect to the FAR, while algorithms 7 and 16-14 were superior with respect to the MTTD.

Duration of Incident Messages

To increase decision credibility regarding an incident message, one could require the message to have a certain duration, the assumption being that a false message will terminate after a short while. Thus, if the distributions of durations of true and false messages are determined, it should be feasible to relate message duration to the probability of a message's being true.

Cumulative distributions of duration of false alarms and incident messages for algorithm 7 are shown in Figures 9 and 10 at the 90 and 90-50 percent detection levels. From these figures it can be seen that the distribution of duration of false-alarm messages is such that for both levels of detection, nearly 50 percent of the messages endure 30 min or more. This, of course, indicates a weakness in the algorithm that experienced between 0.60 and 0.70 FAR.

The distribution of false alarms with time (by 30-min intervals) during the daily study period (3:00-5:00 p.m.) was found to be uniform. This suggests that no change in thresholds with time was necessary for any particular location.

Relationship Between FAR and Geometric Features

The introduction of problem-section-related thresholds representing the 50 percent detection level led to some improvement in the efficiency of the algorithms. The relationship between the number of false alarms and geometric features that resulted from the operation of algorithms 7, 8, and 10 is presented in Tables 8 and 9 for the 90 and 90-50 percent detection levels for both directions of the Eisenhower Expressway.

Algorithm 7 showed the most improvement in terms of reduction of false alarms because individualized thresholds were incorporated. The other algorithms did not show consistent improvement. For example, the introduction of thresholds representing the 50 percent DR at the lane drop at Austin (Figure 8) did not change the FAR of algorithm 8 but rather increased it (not necessarily significantly) for algorithms 7 and 10. This lane drop causes the most severe shock waves on the facility for most of the afternoon rush period.

The long duration of false alarms in this section is a major cause of the high percentage of messages of long duration in the cumulative distribution of incident-message duration (Figures 9 and 10).

When shockwaves are less severe, as in the case of the sun effect on traffic on the outbound freeway near Des Plaines Avenue, the individualized thresholds (related to the 50 percent detection level) seemed to improve the false-alarm situation considerably for all algorithms. Another problem section inducing false alarms and rendering the individualized set of thresholds there ineffective was the bridge near Addison Creek between 25th Avenue and Mannheim Road, where only algorithm 8 showed improved operation. The effect of other problem sections inducing nonincident shock waves resulting in false alarms can be determined from the above figure.

FINDINGS, OBSERVATIONS, AND RECOMMENDATIONS

Based on the analyses conducted in the course of this research the following are the major findings and observations.

1. No statistically significant differences at the 5 percent level of significance in DR, FAR, and MTTD were found among algorithms 7, 8, and 10 for the 80, 90, and 90-50 percent detection levels, when they were operated on the Eisenhower Expressway.
2. The introduction of individualized thresholds at problem sections did not affect algorithm 8 but improved DR and FAR of algorithm 7 and improved DR and MTTD for algorithm 10.
3. As far as the MTTD was concerned, no apparent differences between the on-line and off-line evaluations were observed.
4. The efficiency of algorithms 7 and 8 remained statistically the same for the 90 and 90-50 percent detection levels.
5. When compared with the locally developed algorithms (16-14 and Bayesian) at the 90-50 percent detection level, algorithm 7 showed overall superiority.
6. Nearly half of all incident and false-alarm messages lasted longer than 30 min.
7. The introduction of individualized thresholds at problem sections could reduce the number of false alarms generated in these sections.
8. DR obtained by algorithms in the off-line evaluation are considerably higher than those obtained in the on-line evaluation.
9. The shockwave-suppressor mechanism of algorithm 8 seemed to be quite effective; required less effort to prepare thresholds for this than for any other algorithm.
10. FARs are quite high, and reducing them poses the biggest challenge in refining present algorithms or developing new ones.
11. The distribution of false alarms over time seemed to be uniform for the 90 and 90-50 percent detection levels, which indicates that no changes in thresholds at any particular section with time during rush hour were necessary.
12. Algorithms 7 and 8 seem to operate quite similarly, but algorithm 7 was apparently better.

The recommendations for further action are

1. To investigate the behavior of traffic features at bottlenecks during incidents in order to be able to distinguish between incident- and non-incident-related shockwaves,
2. To develop an effective and inexpensive supportive incident-verification system to minimize FAR, and
3. To develop an improved nonincident shockwave-suppressor mechanism and to incorporate it into the efficient pattern-recognition algorithms.

ACKNOWLEDGMENT

The research reported in this paper, like that in the preceding paper, was conducted within the framework of IDOT and with the cooperation of FHWA. We wish to acknowledge the support of the staffs of the Bureau of Air Operations and the Traffic Systems Center in the data-collection and programming phases of this study. The contents of this report reflect our views, and we alone are responsible for the facts and the accuracy of the data presented here. The contents do not necessarily reflect the official views or policies of IDOT or FHWA. This report does not constitute a standard, specification, or regulation.

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Development of a Transport System Management Planning Process in the Delaware Valley Region

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The joint Federal Highway Administration and Urban Mass Transportation Administration (FHWA-UMTA) guidelines require cities to develop a transportation system management (TSM) element, a short-range element of the transportation plan. The metropolitan planning organizations (MPOs) initially responded to these requirements by preparing a plan report that includes a composite list of projects from the highway and transit capital programs (reverse process). Then, the MPOs began to improve on their initial submissions and to create a process for developing the TSM elements of the plans. This paper presents the Delaware Valley's experience, the outcome of the first

two stages of TSM element development, and the process currently being followed in developing future TSM plans.

Growing government emphasis on short-range transportation system management (TSM) planning has prompted individual urban areas to reformulate the transportation planning process. Experiences around the country have varied, and much can be learned from examining them.

This paper presents the response of the Delaware Valley Region (DVR) to the requirement of TSM planning by providing a regional perspective on the transportation system. The experiences and outcome of the first two stages of TSM development and the process currently being followed in developing future TSM plans are also presented.

REGIONAL PERSPECTIVE

A full appreciation of DVR's response to the TSM planning requirement can only be gained through an understanding of the region's transportation network.

Public Transportation System

Unlike most urban areas in the United States, the DVR Planning Commission (DVRPC) region possesses an extensive system of various types of fixed-guideway rail transit [1333 track km (828 miles) in 1976, of which 269 km (167 miles) was streetcar, 894 km (555 miles) was 13 commuter railroads, and 171 km (106 miles) was rapid transit]. Most of these rail systems have been in place for 50 or more years, and the development of the region closely followed these lines for many years.

A total of 10 895 parking spaces are available at 153 suburban and exurban railroad stations. Bike racks are also provided at 45 stations, 37 of which are in the suburbs.

The two highest-density corridors served by line-haul transit in the region are the Broad Street corridor and the 69th Street-Center City-Frankford corridor. The density along these Southeastern Pennsylvania Transportation Authority (SEPTA) rapid-transit lines results in a high percentage of passengers boarding rapid transit by foot or from surface transit.

The rapid transit system is supplemented by 73 bus routes, 5 trackless lines, and 12 light rail routes, all operated by SEPTA's City Transit Division (CTD). Five of the light-rail routes avoid congestion by operating underground for 4 km (2.5 miles) on the way to the center of the central business district (CBD). Two light rail routes, the Media and Sharon Hill lines, feed the 69th Street terminal from Delaware County; one suburban rapid transit line, the Norristown Line, also feeds into this terminal.

The Delaware River Port Authority's (DRPA) high-speed Philadelphia-Lindenwold line from New Jersey also serves the CBD with four stations and brings people from the New Jersey suburbs into the Philadelphia CBD.

Highway System

The road network within the nine-county DVRPC region is composed of more than 11 000 km (6900 miles) of streets and highways. Of this, approximately 6 percent is limited-access facilities (turnpikes, freeways, and parkways), 5 percent is divided highways, and the vast majority (89 percent) is undivided arterial streets and roads.

More than 85 000 000 km (53 000 000 miles) were traveled on this highway system on an average day in 1972, of which 64 percent was carried by the network in the five Pennsylvania counties and 36 percent by the network in the four New Jersey counties.

While the great majority (78 percent) of the regional system operated at acceptable levels of service with free or stable flow, traffic exceeded capacity on 15 percent of the route kilometers. This is the equivalent of level of service F, or very poor.

An additional 7 percent of the system distance operated between levels of service D and E, which indicates unstable traffic flow with extensive to critical delays, particularly during peak periods of travel.

INITIAL RESPONSE TO TSM PLANNING REQUIREMENT

The initial TSM document (1) for the DVR was produced under a very strict time limitation: only six months from the September guideline to the March submission date. The metropolitan planning organization (MPO), in this case DVRPC, used funds previously allocated to the transit development program to create the TSM plan. As a result of the foregoing, several decisions made by the MPO largely influenced the content and style of the original TSM plan.

1. More emphasis on TSM came from the Urban Mass Transportation Administration than from the Federal Highway Administration. That emphasis was reflected in more transit staff involvement at the MPO level and in the transit emphasis in the issued document.
2. Staff of the various agencies involved in the preparation of the TSM viewed it lightly as just another federal requirement. The tight schedule that caused the railroading of the plan was resented because primary emphasis was on meeting the deadline.
3. The plan was both mode and project oriented. Multimodal proposals were few. TSM was largely a reflection of the transportation improvement program, while regulatory strategies for the most part were ignored.

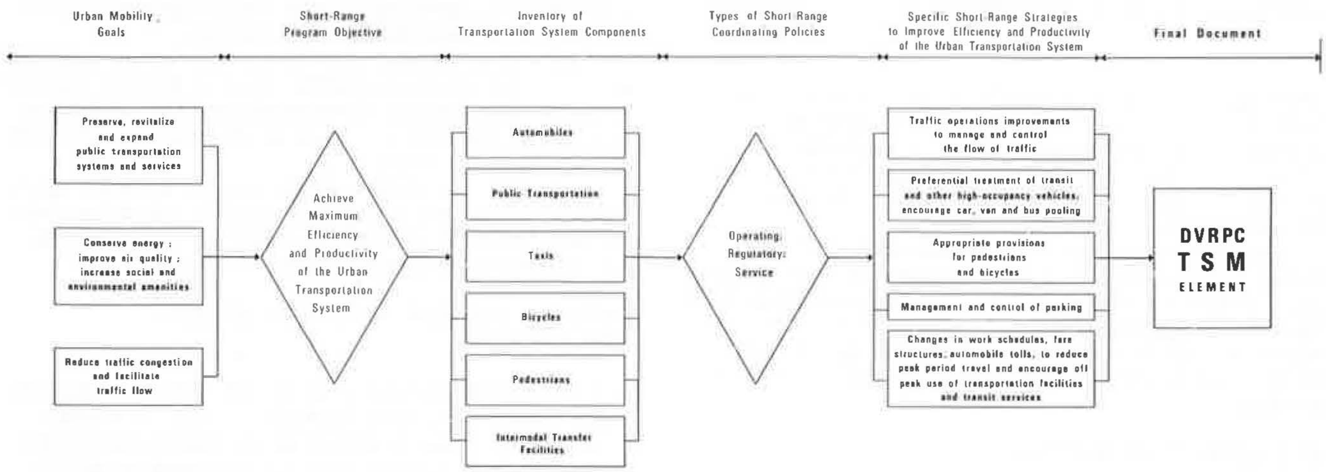
The first TSM effort for the DVRPC region could best be described as a catalog approach. Although this approach was successful in achieving what the MPO staff felt were the primary concerns (be completed on time and address fully each element of the federal guideline), experience has shown the TSM process to differ entirely from the production of the TSM document. Figure 1 shows the logic used in the development of the original TSM plan.

SECOND PHASE OF TSM PLANNING

A second phase of TSM planning began at DVRPC after March 1976. This phase was research oriented and focused on discrete elements of the transportation system. Unlike the previous planning, adequate time was available to collect appropriate data, to propose various possible strategies or actions, to solicit local input and participation, to analyze the impacts of various strategies or actions, and to make recommendations. Several studies of this nature were under way concurrently at the MPO; the results of one even received national attention. These studies included

1. Demand modification strategies program (2),
2. Evaluation of Trenton Commons and Chestnut Street Transitway study (3),
3. Parking analyses for the short range (4),
4. Short-range program development, and
5. Impact on mobility, energy, and emissions.

Figure 1. Planning process for developing original DVRPC TSM plan.



These studies were undertaken by the MPO staff, assisted occasionally by other agencies, particularly in the area of data collection. Four of these studies closely followed a case-study approach to detailed analyses of discrete elements of the regional transportation system. The fifth study was an attempt to measure the total regional impact if the entire original TSM plan were implemented. An unencouraging note was the conclusion of the fifth study, which showed the TSM plan as having only a small impact on total regional mobility, energy consumption, and fuel emissions. This finding will undoubtedly affect the next TSM plan.

During this second phase of TSM planning, transportation professionals' appreciation of their TSM concept heightened greatly. Criticism of the concept ended completely, and efforts to integrate local, county, city, and transit-operator improvements into the TSM framework became evident.

One member government, the city of Philadelphia, and its major transit operator, SEPTA, began TSM planning projects of their own. It should be noted, however, that the city of Philadelphia, Port Authority Transit Corporation, SEPTA, Mercer Metro, and the Pennsylvania and New Jersey Departments of Transportation have had ongoing project-oriented technical studies that supply numerous TSM improvement projects.

Twenty-two months after the publication of the original TSM plan in March 1976, DVRPC published a TSM summary of activities that reported on all TSM developments and research efforts in the region. This document represented a second benchmark in the TSM planning process for two reasons. First, it reported the results of numerous independently conducted and implemented efforts to fulfill the spirit of the TSM planning requirement. Second, it marked the demise of the view of TSM as a fragmented effort in which each agency advanced efforts in its own best interests but with scant joint planning or coordination.

During this period, a comprehensive roles and responsibilities statement had been prepared by the MPO staff. However, the board of the MPO failed to endorse the document because they felt it to be doubtful that the MPO board could impose such an agreement on other constituted boards such as transit authorities, toll roads, and bridge commissions or authorities; these other agencies are not represented on the MPO board.

THIRD PHASE OF TSM PLANNING

The need for improved interagency cooperation was recognized in January 1978, when the region formed a special TSM task force to assist DVRPC staff in preparing the short-range transportation plan for the region. Performing an advisory role, the task force was to monitor, comment on, and solicit input as DVRPC staff prepared a new short-range transportation plan for the region. The eight-step process is to be followed in this order:

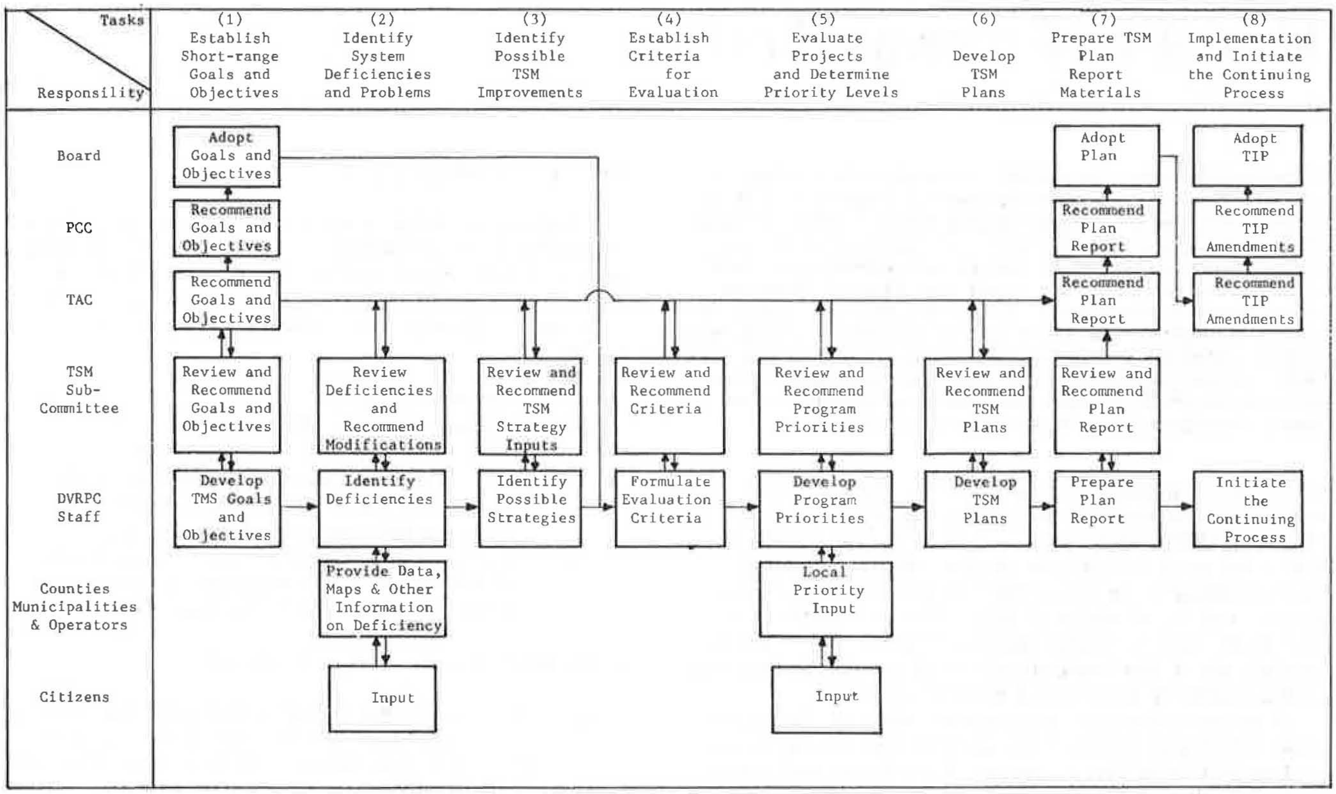
1. Establish short-range goals and objectives;
2. Identify transportation system deficiencies and problems;
3. Identify possible TSM improvements, including current plans and programs;
4. Establish criteria for project and plan evaluation;
5. Determine project and plan study priorities;
6. Develop a plan from the above activities;
7. Prepare TSM report materials; and
8. Initiate needed follow-up activities and planning studies.

In cooperation with the task force, DVRPC staff have prepared TSM planning guidelines for the region. Input from all agencies involved in surface transportation will be used to develop the short-range transportation plan. An important step was taken when the DVRPC board, the MPO governing body, adopted short-range goals and objectives for transportation planning. These goals and objectives were developed by DVRPC staff with the assistance of the TSM task force:

1. Goals
   a. Improve efficiency, mobility, safety and productivity of the transportation system;
   b. Conserve resources such as energy and money;
   c. Improve environmental quality;
2. Objectives
   a. Reduce congestion;
   b. Reduce energy consumption;
   c. Improve transit use;
   d. Improve air quality;
   e. Reduce noise level;
   f. Reduce accidents;
   g. Increase automobile occupancy;
   h. Improve accessibility of transportation services to all potential users; and
   i. Reduce cost.

Figure 2 illustrates the process for developing the new TSM plan for the region. Important innovations

Figure 2. Planning process for developing revised DVRPC TSM plan.



over the previous process are

1. Full participation of counties, transit operators, cities, MPO, and state departments of transportation on the TSM task force (thus all participants will be involved in the process);
2. Clear linkage with the technical advisory committee on highways and transit plans and the planning coordinating committee and board of DVRPC;
3. Systematic study of transportation deficiencies and problems and possible remedies;
4. Priorities assigned to projects recommended in the TSM plan;
5. Goals and objectives developed specifically for TSM planning in the DVRPC region (these goals and objectives will be used when the plan is evaluated); and
6. Provision for timely citizen input during development of the TSM plan.

The process outlined in Figure 2 has not advanced far enough to state definitely the strengths and weaknesses of the process. One apparent strength is broad-based interest in TSM planning. One apparent weakness is the pace at which the task force can assimilate, review, and comment on what is prepared by the MPO staff. The process calls for task force recommendations at each step in the process, so a slow pace will ensure a two- or three-year effort to produce the new plan.

It should be kept in mind that DVR covers portions of two states, four cities, nine counties, three transit-operating authorities, four toll-road authorities, and three interstate bridge agencies. Obtaining agreement from all these jurisdictions and coordinating it is time-consuming and requires substantial diplomacy. Nevertheless, the conditions of DVRPC are not totally unique, and other large regions may benefit from its experience.

ACKNOWLEDGMENT

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FREFLO: A Macroscopic Simulation Model of Freeway Traffic

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Three categories of simulation models for freeway traffic have been developed in the past: microscopic, mesoscopic, and macroscopic. Microscopic models represent individual vehicle movements; mesoscopic models represent platoon movements; and macroscopic models represent traffic flow in terms of aggregate measures such as density, space-mean-speed, and flow rate. This paper discusses a model in the macroscopic category that is particularly useful for evaluating freeway operations. The model is described in mathematical detail for basic flow simulation, ramp metering and diversion, surveillance, and representation of freeway incidents. Computation of performance measures is also detailed. The simulation model, FREFLO, which is based on the model equations presented, is then described and illustrated with a sample run.

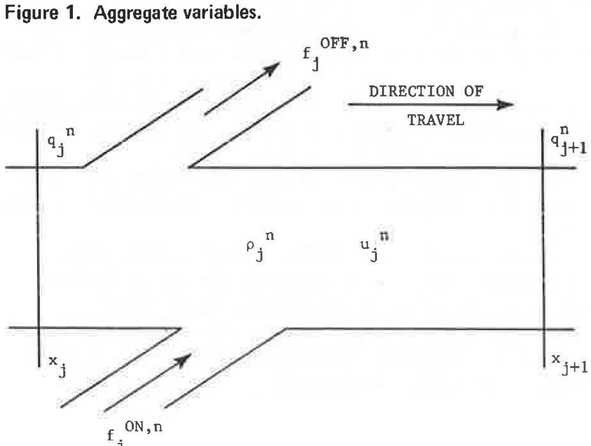
A variety of models of traffic flow on highways and freeways has been developed during the past two decades. These models range from analytically tractable car-following models that have limited ability to predict vehicle behavior in real traffic to highly detailed simulation models, of which INTRAS (1) is the most recent and most comprehensive representative. All of these models are at the level of individual vehicle movements and are usually referred to as microscopic.

A second category, macroscopic models, has also been developed (2,3) and is characterized by representations of traffic flow in terms of aggregate measures such as volume (or flow rate), space-mean-speed, and traffic density. This category of model sacrifices a great deal of detail but gains by way of efficiency an ability to deal with problems of much larger scope. There is debate as to whether necessary accuracy is also sacrificed.

There is also a third category of model, mesoscopic. In these, platoons are followed. The SCOT model (4) is the foremost example of this category.

In this paper, we shall discuss a certain subset of the macroscopic model that is, within this category, the most detailed and is capable of representing dynamic behavior well enough to allow study of dynamic traffic operations. We shall, further, describe the related simulation package, FREFLO (5, 6), a successor to the computer simulation package MACK (7).

Figure 1. Aggregate variables.



TRAFFIC VARIABLES

The freeway segment is divided into sections, defined by section boundaries at x_j, $j = 1, \ldots, N$. The peak period is divided into uniform time intervals of length Δt. Within the j th section defined by the interval (x_j, x_{j+1}), we shall define the following variables (see Figure 1):

l_j = number of lanes;
Δx_j = section length in kilometers;
ρ_j^n = section density, or number of vehicles in this section at time $t_0 + n\Delta t$ divided by the number of lanes and the section length in vehicles per lane per kilometer; and
u_j^n = section space-mean-speed, or the average of the speeds of the vehicles in section j at time $t_0 + n\Delta t$ in kilometers per hour.

At the section boundary x_j, we define

q_j^n = volume, or the rate at which vehicles pass x_j in the time interval $[t_0 + (n - 1)\Delta t, t_0 + n\Delta t]$ divided by the number of lanes in vehicles per hour per lane,

and, where appropriate,

$f_j^{ON,n}$ = on-ramp volume, or rate at which vehicles enter the on-ramp at x_j in the interval $[t_0 + (n - 1)\Delta t, t_0 + n\Delta t]$ in vehicles per hour, and
$f_j^{OFF,n}$ = off-ramp volume, or rate at which vehicles exit on the off-ramp at x_j in the interval $[t_0 + (n - 1)\Delta t, t_0 + n\Delta t]$ in vehicles per hour.

BASIC MODEL

The first equation expresses the conservation of vehicles:

$$\rho_j^{n+1} = \rho_j^n + (\Delta t/l_j \Delta x_j)(l_{j-1} q_j^{n+1} - l_j q_{j+1}^{n+1} + f_j^{ON,n+1} - f_j^{OFF,n+1}) \tag{1}$$

where $n = 0, 1, 2, \ldots, N$ and $j = 1, \ldots, J$.

Note that we have adopted the convention that a change in the number of lanes is assumed to take place slightly downstream of a section boundary. Consequently, the total freeway volume at x_j is $l_{j-1}q_j$. The off-ramp volume is taken to be given by

$$f_j^{OFF,n+1} = \beta_j q_j^{n+1} \tag{2}$$

Under uniform conditions within a section, the volume, density, and speed are related precisely by

$$q_{j+1}^{n+1} = \rho_j^n u_j^n \tag{3}$$

We adopt this as our second equation. The final equation of the model is derived from a continuous-space model by spatial averaging (2).

The dynamic speed-density relationship is

$$u_j^{n+1} = u_j^n - \Delta t \left\{ \underbrace{u_j^n [(u_j^n - u_{j-1}^n)/\Delta x_j]}_{\text{convection}} + \underbrace{1/T_j [u_j^n - u_e(\rho_j^n)}_{\substack{\text{relaxation to}\\ \text{equilibrium}}}) + \underbrace{(\nu_j/\rho_j^n)[(\rho_{j+1}^n - \rho_j^n)/\Delta x_j]}_{\text{anticipation}} \right\} \quad (4)$$

where j and n proceed as in Equation 1, $T_j = k_T \Delta x_j$, and $\nu_j = k_\nu \Delta x_j$. The parameters k_T and k_ν are termed the relaxation time and anticipation coefficients, respectively. The three groups of terms express three physical processes. The first of these, $[(u_j^n - u_{j-1}^n) \div \Delta x_j]$, is convection, i.e., the fact that vehicles traveling at speed u_{j-1} in the upstream section (section j-1) will tend to continue to travel at that speed as they enter section j. The second, $u_j^n - u_e(\rho_j^n)$, represents the tendency of drivers to adjust their speeds to the equilibrium speed-density relationship. The third, $[(\rho_{j+1}^n - \rho_j^n)/\Delta x_j]$, is a model of anticipation of changing travel conditions ahead; i.e., drivers tend to slow down if the density is seen to be increasing.

In addition, boundary conditions and the initial values of the speeds and densities in each section must be defined. One "dummy" section at each end of the freeway segment is added so that $u_1^n = u_0^n$ and $\rho_1^n = \rho_0^n$.

In the simulations, we have taken

$$u_e(\rho) = \min [88.5 (172 - 3.72\rho + 0.0346\rho^2 - 0.001\,19\rho^3)] \quad (5)$$

where $u_e(\rho)$ is in kilometers per hour. This speed-density relationship is a rescaled version of a least-squares fit to data taken from the Harbor and Hollywood Freeways in Los Angeles (see Figure 2). It is generally necessary to develop a new speed-density relationship for each distinct freeway facility.

Associated with this speed-density relationship, there is a nominal section capacity, defined by

$$c = \max_\rho [\rho u_e(\rho)] \quad (6)$$

This nominal capacity is the largest volume that can be sustained under spatially and temporally uniform conditions. It should be realized, however, that under nonuniform conditions, e.g., in the vicinity of a geometric bottleneck, volumes may exceed nominal capacity.

Note that capacities specific to sections can be obtained by appropriately scaling the speed-density relationship.

Good choices for the parameters k_T and k_ν are 46 s/km (75 s/mile) and 40 km/h (25 mph), respectively (8). The ratio of these parameters to one another is closely related to the phenomenon of slow-down and speed-up cycles in traffic (9). Below the critical speed, u_c, defined by

$$u_c = \sqrt{k_\nu/k_T} \quad (7)$$

traffic, as simulated by the model, exhibits this phenomenon. With the parameters indicated, u_c equals 56 km/h (34.6 mph). Generally, larger values of k_ν and k_T lead to a more sluggish modeled response.

TRAFFIC INCIDENTS

An incident may be reflected in the aggregate variables by (a) a reduction in the number of available lanes, (b) a restriction in the volume flowing past the incident site, and (c) an alteration in parameters such as k_T and k_ν.

The first effect can be represented by placing all vehicles in the affected section in the available lanes. This is manifested as an instantaneous adjustment in density through obvious relationships. The second effect can be represented by noting the expression $q_{j+1}^{n+1} = \rho_j^n u_j^n$ and adjusting the speed u_j^n to limit the flow to the specified volume flowing past the incident site. The third effect has been investigated, but has not proved effective (8).

Figure 2. Speed-density relationship.

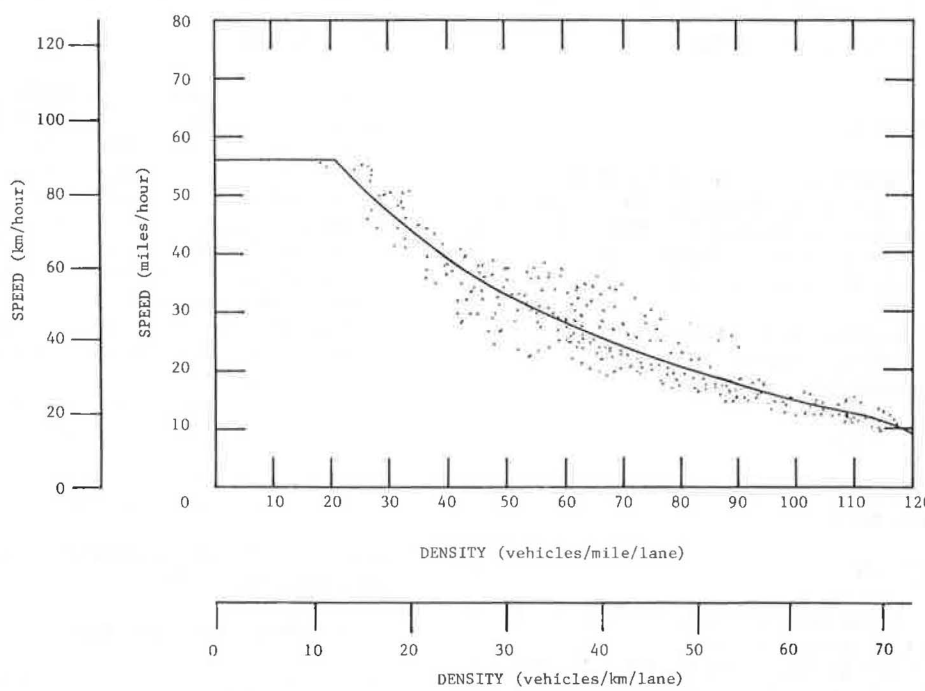


Figure 3. Modeling of on-ramps.

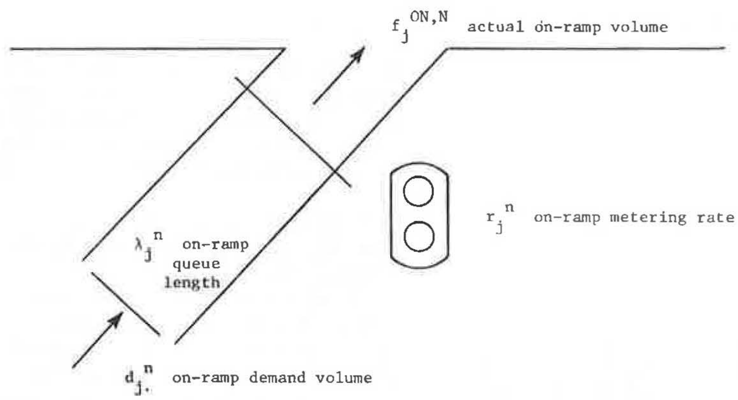
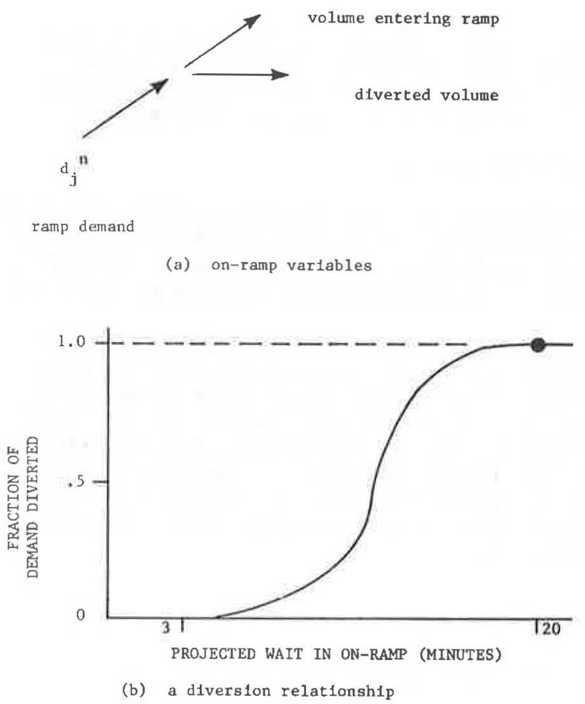


Figure 4. Diversion at on-ramps.



ON-RAMP METERING

The variables associated with on-ramp and ramp metering are illustrated in Figure 3. The ramp demand, d_j^n, is metered at the rate r_j^n. When d_j^n exceeds r_j^n, a queue, λ_j^n, is generated. When there is no λ_j^n and d_j^n is less than r_j^n, the actual on-ramp volume, $f_j^{ON,n}$, will equal d_j^n.

In other circumstances, the total demand for the interval Δt can be expressed as $\lambda_j^n/(\Delta t + d_j^n)$; the actual on-ramp rate is then given by

$$f_j^{ON,n} = \min[r_j^n, \lambda_j^n/(\Delta t + d_j^n)] \tag{8}$$

There is now also the need to maintain the queue variable through the expression

$$\lambda_j^{n+1} = \lambda_j^n + (d_j^n - f_j^{ON,n})\Delta t \tag{9}$$

where λ_j^n is the queue length in vehicles on the ramp entering section j at the time $t_0 + n\Delta t$.

As queues build up, there is a tendency for a portion of the drivers arriving at the on-ramp to divert to alternate routes. This effect can be modeled by making this fraction a function of the estimated waiting time, computed as λ_j^n/r_j^n. Figure 4 illustrates the related variables and a candidate diversion relationship. Application of this concept requires that we modify the actual demand on the ramp accordingly.

TRAFFIC CONTROL

Traffic control or ramp metering may be open loop (time of day) or feedback (traffic responsive). In the case of open-loop control, ramp-metering rates are specified for each on-ramp as a function of the time of day. The metering rates function as constraints on the actual on-ramp volume.

With feedback control, the ramp-metering rates depend on traffic conditions as measured by the surveillance system. A local-occupancy feedback mode is illustrated in Figure 5 (10). In this mode, metering rates depend on the occupancy measured at a detector station on a neighboring freeway, usually the station immediately upstream of the ramp. The solid line in Figure 5 applies if the last change in occupancy was positive; the dashed line applies if the last change in occupancy was negative.

We will provide sample outputs involving this ramp-metering scheme in a subsequent section.

SURVEILLANCE

Freeway surveillance is generally accomplished through the use of presence detectors, usually induction loops, placed in each lane of the roadway (8). To simulate occupancy and volume measurements, each measurement is associated with a simulated section. Then the smoothed occupancy is determined from the corresponding section density by a scale factor (here taken as G). The smoothed volume is taken directly from the associated simulated volume.

The smoothing performed in each case is single exponential. The specific formulas are in the form

$$\begin{aligned} \text{SOCC(time n + 1)} &= \text{SOCC(time n)} \times (1 - \alpha) \\ &+ \alpha \times \text{current density/G} \end{aligned} \tag{10}$$

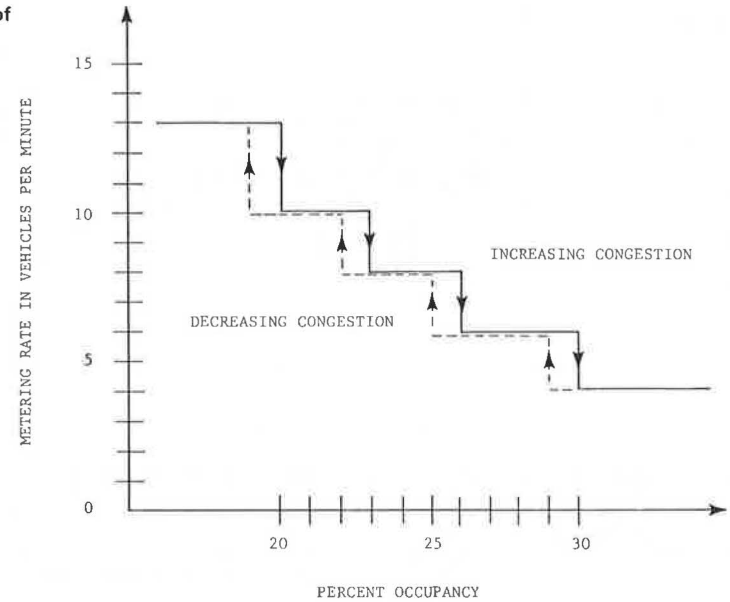
$$\begin{aligned} \text{SVOL(time n + 1)} &= \text{SVOL(time n)} \times (1 - \alpha) \\ &+ \alpha \times \text{current volume} \end{aligned} \tag{11}$$

where α controls the effective time interval over which averaging takes place.

PERFORMANCE MEASURES

Two performance measures generally produced from

Figure 5. Discrete metering rates as a function of percentage occupancy.



the aggregate variables are service and travel time.

Service

The service rate for the freeway partitioned into sections indexed J = 1, . . . , J is given by

$$\text{total service rate} = \sum_{j=1}^{J} l_j \Delta x_j q_{j+1}^n \tag{12}$$

and has units of vehicle kilometers per hour. The total service performed by the freeway over the time interval (t_0, $t_0 + N\Delta t$) is then simply

$$\sum_{n=1}^{N} \sum_{j=1}^{J} l_j \Delta x_j q_{j+1}^n \Delta t \tag{13}$$

and has units of vehicle kilometers.

Travel Time

Total travel time on the freeway is given by the expression

$$\sum_{n=1}^{N} \sum_{j=1}^{J} \ell_j \Delta x_j \rho_j^n \Delta t \tag{14}$$

and has units of vehicle hours. In the presence of ramp queueing, there is a ramp component of total travel time, given by

$$\sum_{n=1}^{N} \sum_{j=1}^{J} \lambda_j^n \Delta t \tag{15}$$

Fuel consumption and pollution emissions are important further measures of performance. Relationships suitable for use with the aggregate variables are not yet firmly established, but some present relationships may be useful and others currently under development certainly will be. Here we shall describe the form the computations take.

Fuel consumption and pollution emissions (HC, CO, NO_x) can be computed for an average automobile from tables (10). Each table provides a rate for a specified speed and acceleration. Thus the total rate for the freeway is in the form

$$\sum_{j=1}^{N} \rho_j^n \ell_j \Delta x_j \times F(u_j^n, a_j^n) \tag{16}$$

where the vehicle acceleration is given by the latter two terms of Equation 3, i.e., the relaxation-to-equilibrium and anticipation terms. Rates for the ramps are also computed from a relationship of the form

$$\sum_{\text{ramps}} \lambda_j^n F(0,0) \tag{17}$$

FREFLO

FREFLO is a FORTRAN program that incorporates all the model features detailed in the following. It is a successor to the program MACK (7). Documentation in the form of a user's guide (5) and program documentation (6) are available.

FREFLO can do the following:

1. Provide a basic model,
2. Perform input data diagnostics,
3. Represent incidents,
4. Model on-ramps,
5. Control time-of-day,
6. Represent surveillance,
7. Represent two traffic-responsive metering modes,
8. Provide standard measures of travel and travel time,
9. Include fuel consumption, and
10. Include pollution emissions.

FREFLO requires such geometric data as number of lanes ℓ_j, j = 1, . . . , d; section lengths Δx_j, j = 1, . . . , d; on-ramp and off-ramp locations; and nominal section capacities. The traffic data it requires are densities ($\rho_1^0, \rho_2^0, \ldots, \rho_N^0$) and speeds ($u_1^0, u_2^0, \ldots, u_N^0$) for the initial state, upstream freeway volume (q_1^n, n = 1,2, . . . , N) and on-ramp rates ($f_j^{ON,n}$, n = 1, 2, . . . , N for each section j with an on-ramp) for input volumes and β_j^n n = 1, 2, . . . , N for each section j with an off-ramp for off-ramp fractions.

The simulation parameters of FREFLO are k_μ, an anticipation parameter; k_T, a relaxation parameter; Δt,

Figure 6. Freeway segment simulated in the example.

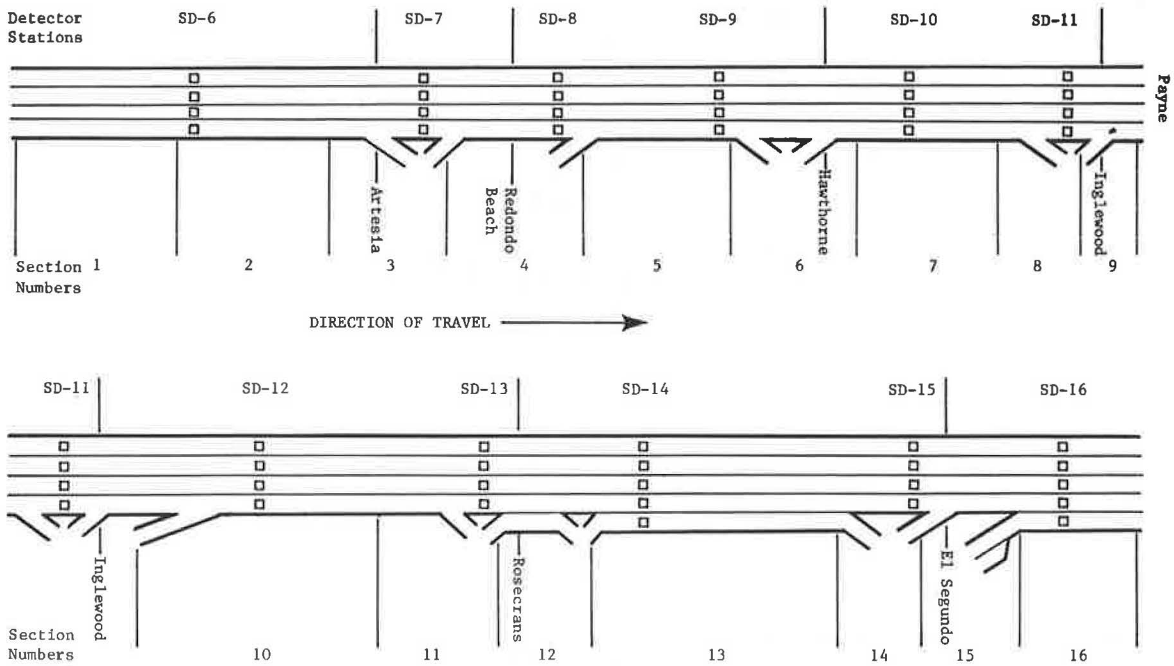


Figure 7. Freeway geometry and link capacities.

FREEWAY GEOMETRY AND LINK CAPACITIES

TOTAL LENGTH	5.40 MI	(8.7 KM)
TOTAL LANE MILES	22.50	(36.2 LANE-KM)
NUMBER OF ON RAMPS	9	
NUMBER OF OFF RAMPS	6	

SECTION NO	LENGTH (MI)	(KM)	LANES	SEGMENT CAPACITY (VEH/LN-HR)	ONRAMP CAPACITY (VEH/HR)	OFFRAMP CAPACITY (VEH/HR)
1	0.50	0.80	4	1800.	0.	0.
2	0.40	0.64	4	1800.	0.	0.
3	0.30	0.48	4	1800.	0.	1800.
4	0.30	0.48	4	1800.	1800.	0.
5	0.40	0.64	4	1800.	1800.	0.
6	0.30	0.48	4	1800.	1800.	1800.
7	0.20	0.32	4	1800.	0.	0.
8	0.30	0.48	4	1800.	0.	1800.
9	0.20	0.32	4	1800.	1800.	0.
10	0.40	0.64	4	1800.	1800.	0.
11	0.38	0.61	4	1800.	0.	1800.
12	0.22	0.35	4	1800.	1800.	1800.
13	0.40	0.64	5	1800.	1800.	0.
14	0.38	0.61	4	1800.	0.	1800.
15	0.22	0.35	4	1800.	1800.	0.
16	0.50	0.80	5	1800.	1800.	0.

Figure 8. Initial freeway states.

INITIAL FREEWAY STATES

SECTION NO	INITIAL DENSITY (VEH/LANE-MI)	(VEH/LANE-KM)	INITIAL SPEEDS (MI/HR)	(KM/HR)
1	40.	25.	45.	72.
2	40.	25.	45.	72.
3	40.	25.	45.	72.
4	40.	25.	45.	72.
5	40.	25.	45.	72.
6	40.	25.	45.	72.
7	40.	25.	45.	72.
8	40.	25.	45.	72.
9	40.	25.	45.	72.
10	40.	25.	45.	72.
11	40.	25.	45.	72.
12	40.	25.	45.	72.
13	40.	25.	45.	72.
14	40.	25.	45.	72.
15	40.	25.	45.	72.
16	40.	25.	45.	72.

Figure 9. Simulation parameters and constants and incident scenario.

SIMULATION PARAMETERS AND CONSTANTS

TIMING DATA

INTEGRATION INTERVAL	6.0 SEC
OUTPUT INTERVAL	1.0 MIN
STARTING TIME	730.
ENDING TIME	800.

INTEGRATION CONSTANTS

TEE = 75.0 SEC/MI (46.6 SEC/MI) VEE = 25.0 MI/HR (40.2 KM/HR)

COEFFICIENTS FOR SPEED-DENSITY RELATIONSHIP

0.10700E+03 -0.23100E+01 0.21500E-01 -0.74000E-04

VMAX= 55. MI/HR (88.5 KM/HR)

INCIDENT SCENARIO

INCIDENT SUBINTERVAL	1	2	3
ITH INTERVAL BEGIN TIME	740.	750.	750.
LINK AFFECTED	12	12	0
LANES AVAILABLE	3	4	4
INCIDENT CAPACITY	1600.	1600.	0.

a time step; duration of simulation; and speed-density relationship, $u_e(\rho)$. Its incident scenario parameters include number of lanes available and capacity at incident site. Specifications for its ramp-control parameters depend on choice of mode. Surveillance-data processing parameters are detector-station locations and averaging time. Finally, FREFLO offers the output options of diagnostics only or simulation and a choice of detailed outputs.

SAMPLE SIMULATION

To illustrate the functioning of and outputs provided by FREFLO, we consider an example involving the local occupancy-metering mode with an incident. The freeway segment simulated is illustrated in Figure 6. It is a portion of northbound I-405 in Los Angeles. Figures 7-17, the program printouts, contain the complete input

Figure 10. Output options.

OUTPUTS SELECTED INDICATED BY 1

TIME HISTORY PLOT	0
TABLES	1
SPECIAL DENSITY MAP	1
SPEED	1
DENSITY	1
FREEWAY VOLUMES	1
SERVICE RATES	0
ON RAMP DEMANDS	1
OFF RAMP RATES	1
ACTUAL ON RAMP RATES	1
ONRAMP METERING RATES	1
ON RAMP QUEUES	1
DIVERTED TRAFFIC VOLUMES	1
ACCELERATIONS	1
HC EMISSIONS	0
CO EMISSIONS	0
NOX EMISSIONS	0
FUEL CONSUMPTION	0
SMOOTHED OCCUPANCY	1
SMOOTHED VOLUME	1
SIMULATION TO BE EXECUTED	0

SPATIAL AND TEMPORAL BOUNDS FOR COMPUTATION OF SUMMARY PERFORMANCE MEASURES

FIRST SECTION	2
LAST SECTION	15
FIRST TIME	0.
LAST TIME	9999.

Figure 11. Surveillance and diversion parameters.

SURVEILLANCE DATA PARAMETERS

AVERAGING TIME	60. SEC
SMOOTHING CONSTANT	.1000
G-FACTOR	2.5000

DETECTOR STATIONS/SECTION CORRESPONDENCE

DETECTOR STATION	SECTION FOR OCC	VOL
1	1	1
2	1	1
3	1	1
4	1	1
5	1	1
6	2	3
7	3	4
8	4	5
9	5	6
10	7	8
11	8	9
12	10	11
13	11	12
14	13	14
15	14	15
16	16	17

DIVERSION PARAMETERS

DIVP1 = 3.00
DIVP2 = 20.00
DIVP3 = 0.00

Figure 12. Ramp-metering plan parameters.

RAMP METERING PLAN PARAMETERS

METERING MODE 2 SELECTED
PARAMETERS FOR LOCAL OCCUPANCY PLAN

NUMBER OF METERING LEVELS	6
UPDATE INTERVAL	1.00 MIN

ON RAMP	CONTROLLING DETECTOR STATION
4	7
5	8
6	9
9	11
10	11
12	13
13	14
15	15
16	15

OCCUPANCY THRESHOLDS (PER CENT) INCREASING	DECREASING	METERING RATE IF GREATER THAN THRESHOLD (VEH/HR)
		1800.
15.	15.	780.
20.	20.	600.
23.	23.	480.
26.	26.	360.
30.	30.	240.

Figure 13. Traffic demand data.

TRAFFIC DEMANDS BEGINNING AT 730.

UPSTREAM FREEWAY VOLUME 7116. VEH/HR

SECTION NO	ONRAMP VOLUME (VEH/HR)	OFFRAMP FRACTION
1	0.	0.000
2	0.	0.000
3	0.	0.046
4	288.	0.000
5	372.	0.000
6	624.	0.034
7	0.	0.000
8	0.	0.102
9	420.	0.000
10	168.	0.000
11	0.	0.019
12	636.	0.093
13	960.	0.000
14	0.	0.110
15	180.	0.000
16	732.	0.000

Figure 14. Summary simulation results.

SIMULATION RESULTS

TOTAL TRAVEL TIME	0.5104E+03 VEHICLE-HOURS
TOTAL FREEWAY TRAVEL TIME	0.4804E+03 VEHICLE-HOURS
TOTAL RAMP QUEUE WAITING TIME	0.2998E+02 VEHICLE-HOURS
TOTAL SERVICE	0.1508E+05 VEHICLE-MILES (0.9370E+04 VEHICLE-KMS)
TOTAL DIVERTED VOLUME	0.4264E+03 VEHICLES

	TOTAL	FREEWAY	RAMPS
HC EMISSIONS (GMS X 100)	0.2957E+03	0.2751E+03	0.2057E+02
CO EMISSIONS (GMS X 100)	0.3623E+04	0.3216E+04	0.4071E+03
NOX EMISSIONS (GMS X 100)	0.7549E+03	0.7265E+03	0.2838E+02
FUEL CONSUMPTION (GALS)	0.1162E+04	0.1104E+04	0.5780E+02
(LITERS)	0.4398E+04	0.4179E+04	0.2188E+03

Figure 15. Detailed simulation results on speed.

SPEED(MI/HR)*

SECTION INDEX →

TIME	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
730.	45.	45.	45.	45.	45.	45.	45.	45.	45.	45.	45.	45.	45.	45.	45.	45.
731.	44.	45.	45.	43.	43.	43.	44.	44.	44.	44.	45.	46.	45.	43.	44.	45.
732.	45.	45.	45.	43.	41.	41.	43.	44.	44.	44.	45.	46.	46.	44.	44.	46.
733.	45.	45.	44.	43.	40.	40.	41.	43.	43.	44.	45.	47.	46.	45.	45.	47.
734.	45.	45.	44.	42.	40.	39.	40.	42.	42.	43.	45.	47.	47.	45.	46.	47.
735.	45.	45.	44.	42.	39.	38.	39.	41.	42.	43.	45.	47.	47.	46.	47.	48.
736.	45.	45.	44.	42.	39.	37.	39.	40.	41.	42.	44.	47.	47.	46.	47.	49.
737.	45.	45.	44.	41.	38.	37.	38.	39.	40.	41.	44.	47.	47.	47.	48.	49.
738.	45.	45.	44.	41.	38.	36.	37.	39.	39.	41.	44.	46.	47.	47.	48.	50.
739.	45.	45.	44.	41.	37.	36.	37.	38.	39.	40.	43.	46.	47.	47.	48.	50.
740.	45.	45.	44.	41.	37.	35.	36.	37.	38.	40.	43.	46.	47.	47.	49.	50.
741.	45.	45.	43.	40.	36.	35.	36.	37.	38.	38.	27.	19.	37.	45.	50.	51.
742.	45.	45.	43.	40.	36.	34.	35.	36.	36.	33.	19.	16.	37.	47.	52.	53.
743.	45.	45.	43.	40.	36.	34.	35.	35.	34.	27.	15.	14.	37.	47.	52.	53.
744.	44.	45.	43.	40.	35.	33.	34.	34.	31.	23.	13.	13.	36.	47.	52.	53.
745.	44.	45.	43.	39.	35.	33.	33.	32.	28.	19.	12.	13.	36.	47.	52.	53.
746.	44.	45.	43.	39.	34.	32.	32.	29.	24.	17.	11.	12.	36.	47.	52.	53.
747.	44.	44.	42.	39.	34.	31.	30.	26.	21.	15.	10.	12.	36.	47.	52.	53.
748.	44.	44.	42.	38.	33.	30.	28.	23.	19.	13.	10.	13.	36.	47.	52.	53.
749.	44.	44.	42.	38.	32.	28.	26.	21.	17.	12.	9.	13.	36.	47.	52.	53.
750.	44.	44.	42.	37.	31.	27.	24.	18.	15.	10.	9.	13.	36.	47.	52.	53.
751.	44.	44.	41.	36.	30.	25.	22.	16.	13.	10.	19.	28.	40.	49.	54.	54.
752.	44.	44.	41.	35.	28.	23.	20.	14.	12.	14.	23.	30.	38.	45.	51.	53.
753.	44.	43.	40.	34.	26.	20.	18.	13.	13.	18.	25.	32.	38.	43.	48.	51.
754.	44.	43.	39.	32.	24.	19.	17.	14.	15.	20.	26.	32.	38.	42.	47.	50.
755.	43.	43.	38.	30.	22.	18.	17.	16.	18.	21.	28.	33.	38.	42.	46.	49.
756.	43.	42.	36.	28.	20.	18.	18.	17.	19.	23.	28.	34.	38.	42.	45.	49.
757.	43.	41.	35.	26.	20.	18.	19.	19.	20.	23.	29.	35.	39.	42.	45.	48.
758.	42.	40.	33.	25.	20.	19.	20.	20.	21.	24.	30.	36.	39.	42.	45.	48.
759.	42.	39.	32.	24.	20.	19.	20.	21.	22.	25.	31.	37.	40.	42.	45.	48.
800.	41.	38.	31.	24.	20.	20.	21.	21.	23.	26.	32.	37.	40.	43.	45.	48.

Figure 16. Detailed simulation results on on-ramp metering rates.

ON RAMP METERING RATES(VEH/HR)

SECTION INDEX →

TIME	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
730.	1800.	1800.	1800.	1800.	1800.	1800.	1800.	1800.	1800.	1800.	1800.	1800.	1800.	1800.	1800.	1800.
731.	0.	0.	0.	780.	780.	780.	0.	0.	780.	780.	0.	780.	780.	0.	780.	780.
732.	0.	0.	0.	780.	780.	780.	0.	0.	780.	780.	0.	780.	780.	0.	780.	780.
733.	0.	0.	0.	780.	780.	780.	0.	0.	780.	780.	0.	780.	780.	0.	780.	1800.
734.	0.	0.	0.	780.	780.	780.	0.	0.	780.	780.	0.	780.	780.	0.	780.	1800.
735.	0.	0.	0.	780.	780.	780.	0.	0.	780.	780.	0.	780.	780.	0.	780.	1800.
736.	0.	0.	0.	780.	780.	780.	0.	0.	780.	780.	0.	780.	780.	0.	780.	1800.
737.	0.	0.	0.	780.	780.	780.	0.	0.	780.	780.	0.	780.	780.	0.	780.	1800.
738.	0.	0.	0.	780.	780.	780.	0.	0.	780.	780.	0.	780.	780.	0.	780.	1800.
739.	0.	0.	0.	780.	780.	780.	0.	0.	780.	600.	0.	780.	780.	0.	1800.	1800.
740.	0.	0.	0.	780.	780.	780.	0.	0.	780.	600.	0.	780.	790.	0.	1800.	1800.
741.	0.	0.	0.	780.	780.	780.	0.	0.	780.	600.	0.	780.	780.	0.	1800.	1800.
742.	0.	0.	0.	780.	780.	780.	0.	0.	600.	600.	0.	780.	780.	0.	1800.	1800.
743.	0.	0.	0.	780.	780.	780.	0.	0.	600.	600.	0.	780.	600.	0.	1800.	1800.
744.	0.	0.	0.	780.	780.	780.	0.	0.	600.	600.	0.	600.	360.	0.	1800.	1800.
745.	0.	0.	0.	780.	780.	780.	0.	0.	600.	600.	0.	480.	240.	0.	1800.	1800.
746.	0.	0.	0.	780.	780.	780.	0.	0.	600.	600.	0.	360.	240.	0.	1800.	1800.
747.	0.	0.	0.	780.	780.	780.	0.	0.	600.	600.	0.	240.	240.	0.	1800.	1800.
748.	0.	0.	0.	780.	780.	780.	0.	0.	600.	480.	0.	240.	240.	0.	1800.	1800.
749.	0.	0.	0.	780.	780.	780.	0.	0.	600.	480.	0.	240.	240.	0.	1800.	1800.
750.	0.	0.	0.	780.	780.	780.	0.	0.	600.	360.	0.	240.	240.	0.	1800.	1800.
751.	0.	0.	0.	780.	780.	780.	0.	0.	600.	360.	0.	240.	240.	0.	1800.	1800.
752.	0.	0.	0.	780.	780.	780.	0.	0.	480.	360.	0.	240.	240.	0.	1800.	1800.
753.	0.	0.	0.	780.	780.	780.	0.	0.	480.	240.	0.	240.	240.	0.	1800.	1800.
754.	0.	0.	0.	780.	780.	780.	0.	0.	480.	240.	0.	240.	240.	0.	1800.	1800.
755.	0.	0.	0.	780.	780.	780.	0.	0.	360.	240.	0.	240.	240.	0.	780.	1800.
756.	0.	0.	0.	780.	780.	780.	0.	0.	360.	240.	0.	240.	240.	0.	780.	1800.
757.	0.	0.	0.	780.	780.	780.	0.	0.	240.	240.	0.	240.	360.	0.	780.	1800.
758.	0.	0.	0.	780.	780.	780.	0.	0.	240.	240.	0.	240.	360.	0.	780.	1800.
759.	0.	0.	0.	780.	780.	780.	0.	0.	240.	240.	0.	240.	360.	0.	780.	1800.
800.	0.	0.	0.	780.	780.	780.	0.	0.	240.	240.	0.	240.	480.	0.	780.	1800.

Figure 17. Detailed simulation results on CO emissions.

CO EMISSIONS(GM X 100/HR)

SECTION INDEX →

TIME	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
730.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
731.	451.	359.	261.	303.	420.	356.	189.	253.	224.	387.	342.	270.	516.	353.	228.	606.
732.	452.	362.	259.	304.	426.	363.	197.	257.	224.	385.	340.	267.	537.	348.	227.	600.
733.	451.	361.	259.	306.	431.	369.	201.	263.	226.	387.	338.	268.	560.	343.	226.	596.
734.	451.	361.	260.	308.	438.	370.	206.	269.	229.	389.	338.	266.	583.	340.	223.	591.
735.	451.	361.	259.	307.	441.	376.	210.	272.	228.	395.	340.	266.	608.	336.	220.	589.
736.	450.	361.	260.	309.	443.	381.	213.	278.	230.	398.	343.	266.	630.	333.	218.	584.
737.	450.	361.	260.	309.	448.	384.	217.	282.	233.	402.	344.	267.	653.	329.	218.	584.
738.	450.	362.	261.	311.	453.	389.	221.	286.	234.	408.	346.	268.	678.	329.	216.	583.
739.	450.	357.	262.	310.	457.	393.	223.	290.	235.	411.	350.	267.	703.	328.	217.	577.
740.	450.	358.	261.	311.	458.	397.	226.	295.	238.	416.	354.	268.	724.	328.	216.	582.
741.	450.	358.	261.	313.	462.	400.	229.	299.	240.	420.	425.	412.	651.	258.	184.	556.
742.	451.	358.	262.	314.	466.	404.	232.	302.	243.	452.	607.	520.	783.	236.	165.	498.
743.	451.	359.	263.	316.	470.	407.	235.	309.	250.	517.	775.	600.	802.	228.	160.	473.
744.	451.	359.	264.	315.	474.	412.	239.	319.	264.	612.	924.	676.	830.	220.	155.	458.
745.	452.	360.	265.	317.	478.	416.	245.	335.	286.	725.	1046.	574.	826.	214.	151.	447.
746.	452.	360.	265.	318.	482.	422.	252.	360.	318.	846.	1140.	620.	838.	212.	150.	441.
747.	452.	361.	266.	318.	488.	429.	263.	395.	357.	962.	1221.	669.	847.	211.	149.	438.
748.	450.	362.	267.	320.	495.	441.	278.	440.	402.	1070.	1294.	718.	853.	211.	149.	438.
749.	451.	360.	267.	323.	505.	456.	299.	498.	450.	1182.	1364.	750.	857.	211.	149.	437.
750.	451.	361.	269.	326.	518.	477.	324.	563.	502.	1322.	1421.	908.	866.	211.	149.	438.
751.	452.	362.	269.	329.	539.	510.	355.	638.	568.	1421.	982.	642.	902.	287.	187.	473.
752.	453.	363.	270.	335.	564.	546.	396.	726.	628.	1274.	739.	640.	933.	322.	202.	531.
753.	454.	363.	274.	344.	602.	596.	440.	794.	621.	1038.	675.	640.	934.	336.	208.	546.
754.	455.	365.	277.	356.	648.	648.	472.	795.	558.	907.	628.	647.	933.	348.	213.	556.
755.	453.	368.	282.	372.	705.	689.	480.	738.	509.	834.	595.	651.	928.	350.	217.	566.
756.	455.	370.	289.	394.	760.	705.	465.	681.	480.	785.	569.	654.	922.	351.	218.	569.
757.	458.	373.	300.	421.	791.	708.	449.	635.	469.	745.	548.	655.	965.	352.	219.	570.
758.	458.	377.	312.	446.	815.	696.	436.	604.	472.	711.	526.	654.	989.	351.	220.	575.
759.	462.	382.	324.	462.	826.	686.	423.	575.	477.	686.	508.	652.	1006.	347.	219.	577.
800.	465.	388.	336.	478.	831.	677.	411.	550.	484.	659.	493.	652.	1046.	347.	219.	577.

and selected outputs. Full examples of the runs, which were carried out in miles originally, are available in the user's guide (5).

From Figure 7 it can be seen that the simulated segment consists of 16 sections that range in length from 0.3 to 0.8 km (0.2 to 0.5 miles), and have four or five lanes each. Nominal section capacities are taken to be 1800 vehicles/lane-h.

From Figure 9 it can be seen that the simulation covers the half-hour interval between 7:30 and 8:00 and involves an incident. This incident occurs in section 12, lasts from 7:40 to 7:50, reduces the number of available lanes to three (from four), and reduces the nominal capacity to 1600 vehicles/lane-h.

Figure 10 details the relationship between detector stations and sections, also illustrated in Figure 6. Figure 12 provides details of the ramp-metering plan. The plan selected was the local occupancy plan, as illustrated in Figure 5.

Traffic demand data are provided in Figure 13. The indicated off-ramp fractions are to be associated with the parameters β_j defined earlier.

Summary simulation results are provided in Figure 14. The remaining Figures 15-17 provide three of the available detailed outputs. Each of the detailed outputs is in the form of an array of values specific to a time instant and section. In the figures, values are provided at 1-min intervals. The effects of the incident are clearly in these detailed outputs as reduced speeds (Figure 15), reduced metering rates (Figure 16), and increased CO emissions (Figure 17).

CONCLUSION

The macroscopic simulation model as represented by FREFLO has undergone only limited calibration and validation but has shown considerable promise (8). Present research is involved in further validation efforts and will be the subject of a future paper.

Applications of the model described here have been made in several studies of the development and evaluation of ramp-metering strategies (12, 13). FREFLO is currently being used in two national studies. The first of these is a Federal Highway Administration study on control strategies in response to freeway incidents; the second is a study concerned with analytic and field evaluations of ramp-metering strategies.

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Discussion

E. Hauer and V. F. Hurdle, University of Toronto

To examine the validity of the FREFLO package, we used it on a simple example: a freeway section with no ramps and a bottleneck in its middle. The entering travel demand had a peak that exceeded the capacity of the bottleneck for an appreciable length of time (Figure 18).

The freeway has been divided into 15 sections 1 km (0.6 mile) long. Section 9 served in both cases as a bottleneck; in case 1 the bottleneck was due to a capacity restriction, in case 2 to a lane drop. The demand pattern used is artificially simple but still representative of demand served by urban freeways. It was further assumed that it is appropriate to use the default values built into FREFLO. This is probably equivalent to assuming that the example freeway is similar to the Harbor and Hollywood Freeways in Los Angeles, which seem to form the empirical basis of FREFLO.

The initial conditions were selected to ensure a steady state; the velocity was specified as 88 km/h (55 mph) and the density was selected to satisfy the equation flow = density × speed.

Regardless of the numerical values of speed, flow, and density that the program might generate, we expected to observe the following general features:

1. When the demand exceeded the capacity of section 9, it would become a bottleneck and begin to flow at capacity;
2. Once the bottleneck reached capacity, a congested region of high density and low speed would begin forming upstream of section 9;
3. After demand dropped below the capacity of section 9, the extent of the congested region would begin to diminish;
4. The flow in the bottleneck would remain at capacity until the congestion upstream had cleared; and
5. The flow downstream of section 9 would never exceed the capacity of the bottleneck.

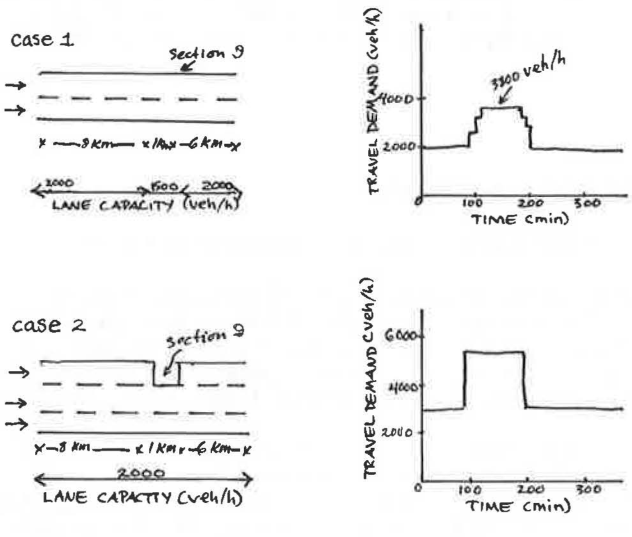
None of this happened. In all test runs, the model produced bottleneck flows substantially in excess of the specified bottleneck capacity for long periods of time. There is no indication of congestion upstream of the bottleneck, nor does the bottleneck restrict flow in sections 10 and beyond.

In short, the output we obtained does not seem to reflect what really happens even in a qualitative manner. There may be three reasons for our failure to obtain sensible results from the FREFLO model.

First, we may have made a mistake in preparing the input. This is somewhat unlikely in view of the simplicity of the input and the fact that we managed to reproduce exactly the results of the example provided in the paper. The latter fact also seems to diminish the possibility that the version of FREFLO we used is faulty.

Second, the program may contain some easily corrigible "bug". We hope that Payne in his closure can demonstrate that, indeed, the problem was of the first

Figure 18. Geometry, capacity, and travel demand for test cases 1 and 2.



or second kind and that, when used on the above simple test examples, FREFLO can be shown to produce sensible results without any need to tamper with the TEE and VEE parameters.

Third, the program may be built on a faulty theoretical foundation. Whether this is in fact so cannot be ascertained without either Payne's help or a reexamination of the program statement by statement. Consequently, we can only comment on two points that may serve to explain the difficulties we encountered.

The difficulties with the test runs seem to stem from the failure of the program to recognize that there is a bottleneck. Payne uses the term "nominal capacity". This is defined to be "the largest volume that can be sustained under spatially and temporally uniform conditions". He cautions, "It should be realized, however, that under nonuniform conditions, e.g., in the vicinity of a geometric bottleneck, volumes may exceed nominal capacity". It is certainly true that the flow on any highway can exceed its capacity, for a short period of time. However, the possibility of flows exceeding the capacity of a section by 40 percent for 80 min, as in case 2, runs counter to the very definition of capacity. Specifically, if there is congestion upstream of a bottleneck, the average flow in the bottleneck is its capacity.

Another plausible fundamental cause for the apparent failure of FREFLO to replicate traffic flow on a freeway may lie in a common misconception caused by the mathematical formulation of the process (2, 14). Theorists of traffic flow have failed to emphasize sufficiently the discontinuity in the process that occurs at the instant at which a freeway section reaches capacity. This has misled some students of the theory into believing that congestion arises mainly from some instability in the microscopic car-following behavior. In contrast, we believe that freeway congestion arises because travel demand exceeds the capacity of bottlenecks.

Papers that describe computer programs are notoriously difficult to discuss. We can only point to difficulties encountered and speculate about possible explanations. It is hoped that Payne in his closure will be able to demonstrate (using the same test examples) that FREFLO is capable of producing sensible answers about speed, flow, and density under conditions when demand exceeds capacity.

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Author's Closure

Before addressing the issues raised in the discussion, it is useful to state the purpose of the paper: to bring to the attention of the traffic engineering community a potentially useful tool for design and evaluation of freeway operational procedures.

It was noted in the paper that model validation efforts have been limited (though promising), and, as they did not yet form a sufficient basis for establishing the validity of the model, they were not described in any detail. I will elaborate here somewhat on these efforts.

In addition to my use of the model in various forms, over the past 10 years, two substantial research projects have employed it. In each case, a limited model-validation effort preceded application. One of these projects was the FHWA-sponsored study on Control Strategies in Response to Freeway Incidents.

In that study, the model parameters were successfully identified to achieve agreement with predictions made by the microscopic simulation model INTRAS (1). In the second, more recent, NCHRP study, model parameters were again successfully identified to gain excellent correspondence with each of four different real-traffic scenarios, two that used Los Angeles data and two Dallas data. Discussions of this effort will be available in the final reports from that project.

The parameters adjusted to achieve agreement in each case involved the section nominal capacities, which were found to be in the range of 1600-1800 vehicles/lane-h.

The purpose of the discussion appears to be to suggest that the model may have fundamental shortcomings. This suggestion is based on a single execution of the model for a simple scenario that produced predictions that, I would agree, are not even qualitatively meaningful.

One will note the contrast between the nature of the validation efforts I have described and the counter-example produced by Hauer and Hurdle. In the former efforts, an attempt was made to adjust parameters to achieve agreement with another simulation or real surveillance data.

The specific shortcomings of the example presented in the discussion is that the nominal section capacities selected (2000 vehicles/lane-h) are too large. The choice made by Hauer and Hurdle may have resulted from too close an association between the model parameters' nominal capacities and the traditional concept of roadway capacity. The model does not directly impose a capacity. Rather, capacity depends on model parameters, including the nominal capacity, and on local spatial variations in density. Thus, in order to obtain a capacity that is sensible—or that corresponds to observations—it is generally necessary to adjust the model parameters.

Generally, one finds the appropriate value of the model parameter nominal capacity to be 10-20 percent less than the capacity observed under ordinary circumstances. The discrepancy in the example of the discussion, 28 percent of the maximum flow rate observed (i.e., the capacity), is larger than generally observed. However, this discrepancy is decreased by appropriate selection of other model parameters (k_T and k_ν as defined by $T_j = k_T \Delta x_j$ and $\nu_j = k_\nu \Delta x_j$, respectively. Such adjustments—that is, reduction of the nominal capacity and decreases in k_T and k_ν—will yield a lower value of roadway capacity and, consequently, produce the effects expected by Hauer and Hurdle.

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Evaluation of the I-35 Route Redesignation in San Antonio

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This paper presents the results of studies conducted in San Antonio, Texas, to evaluate the effectiveness of the redesignation of I-35 to an alternate freeway route. The redesignation was designed as a temporary measure to reroute traffic from a congested freeway to one with adequate available capacity. Therefore, only the advance guide signs and gore signs on the approaches to the diversion points were modified. Diversion potential was estimated by using planning-survey and license-plate origin-destination data. Changes in route choice were identified through license-plate origin-destination studies. Mailed questionnaires used to identify characteristics of through and diverting drivers indicated that, although not all through drivers were expected to divert, a significant number shifted from their original routes.

The Texas State Department of Highways and Public Transportation (SDHPT), working in cooperation with the San Antonio Corridor Management Team (CMT), has initiated programs aimed at alleviating congestion and reducing accidents on I-35 in San Antonio near the central business district (CBD). Included among the programs are (a) the redesignation of the I-35 route around the CBD and (b) use of changeable message signs for incident management and freeway diversion.

The Texas Transportation Institute (TTI) was contracted to evaluate the effectiveness of these two programs as part of research sponsored by the Federal Highway Administration (FHWA) on human factors requirements for real-time motorist information displays. In this paper the effects of the I-35 route redesignation are evaluated.

BACKGROUND

Facility Description

The Interstate and other major highway routes through and around the San Antonio metropolitan area are shown in Figure 1. I-35 in San Antonio is the major facility in the Austin-Laredo corridor. It also forms the western and northern boundaries of the CBD. This section of the freeway was completed in 1957. Design standards at the time, coupled with the presence of major drainage tributaries and proximity to the CBD, dictated sharp horizontal alignment of the four-lane facility. Retaining walls and rigid structures prohibit expansion along the existing roadway surface. Because of capacity constraints and alignments, considerable congestion and relatively high accident rates are experienced (1).

I-10 in the southeast part of the city and I-37 were constructed in the late 1960s and early 1970s as eight-lane facilities according to higher design standards. The I-10/I-37 route around the CBD has considerable available capacity and is seldom congested.

Object of I-35 Route Redesignation

The object of the route redesignation was to encourage through drivers in the I-35 Austin-Laredo corridor to travel on the wider I-10/I-37 route in order to reduce congestion and accident rates on I-35. The redesignation was designed as a temporary measure until I-35 could be reconstructed. The I-35 route from the I-35/I-10/US-90 interchange to the I-35/I-37 interchange is about 7.7 km (4.8 miles). The I-10/I-37 route is about 9.0 km (5.6 miles), 1.3 km (0.8 mile) longer.

Sign Changes

SDHPT modified the advance guide signs and gore signs on the freeway sections shown on Figure 2. The sign modifications, completed in November 1977, included moving the destination names (Austin or Laredo) and the I-35 shields so that northbound (NB) and southbound (SB) I-35 traffic would follow the I-10/I-37 route around the CBD. Figure 3 illustrates a typical signing change, which was made at the NB I-35 exit to eastbound (EB) I-10.

For ease of discussion, the two routes will be referred to as route A and route B throughout the remainder of this paper. Route A is the original I-35 route; route B is the newly redesignated route that follows I-10/I-37 around the CBD (see Figure 2).

Study Scope

This study addresses only NB travel in terms of on-site data collection and questionnaires. Cost constraints limited the field data collection to only one direction, and NB was chosen because more appropriate data-collection sites existed there. Some overall conclusions about SB travel are drawn when appropriate.

For the purposes of this study, a through trip is any trip whose origin and destination require that the vehicle

Figure 1. Major highways in the San Antonio area.

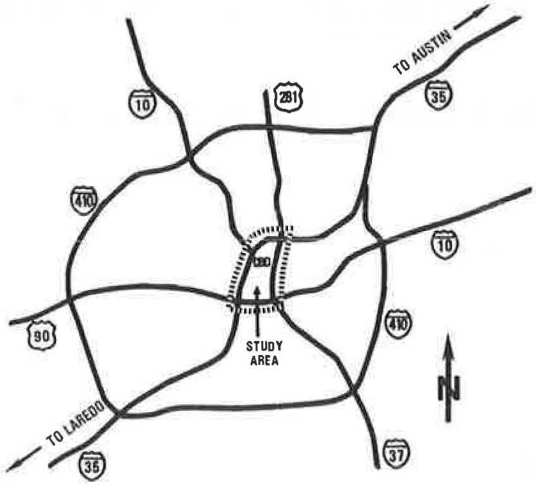
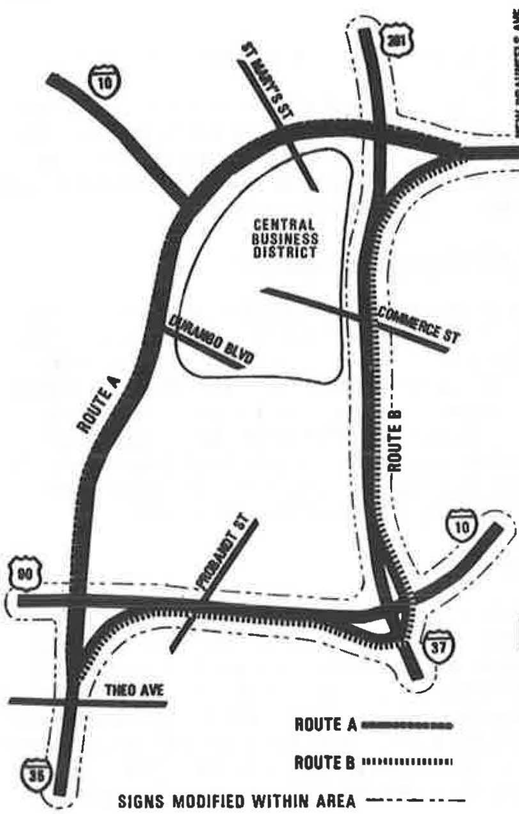


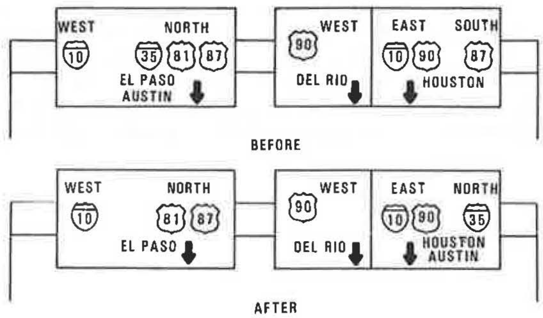
Figure 2. Primary and diversion routes.



travel completely through the study area. That is, a through trip is one originating south or southwest of the I-35/I-10/US-90 interchange and destined for north or northeast of the I-37/I-35/US-281 interchange (see Figure 1).

This paper is a summary of an evaluation of the I-35 route redesignation in San Antonio, prepared under the project on human factors requirements for real-time motorist information displays. Further details on the topics and analyses may be found in Stockton, Dudek, and Hatcher (2).

Figure 3. Typical sign changes.



BASE CONDITIONS

Objectives

The primary object of this portion of the study was an estimate of the number of through trips that could potentially be diverted to the redesignated route (B). This was to be accomplished by estimating the existing number and distribution of through trips on routes A and B immediately before the sign changes.

Approach

Because it was impractical to actually measure the number of daily through trips, daily through volumes were estimated by determining the number of through drivers from previous planning survey origin-destination (O-D) data, developing annual traffic volume growth factors in the corridor, and extrapolating the planning survey data to present (1977) volumes by using the growth factors. The distribution of drivers by route (A or B) was obtained from a license-plate O-D study.

The most recent study providing information from which through trips could be estimated was a 1969 O-D survey prepared by the San Antonio-Bexar County Urban Transportation Study (SABCUTS) (3). This survey gives detailed information on the number of daily trips among various external stations (at the county line) and internal districts (within the county).

The two external stations and internal areas (several internal districts combined) that are most relevant to the route redesignation study are shown in Figure 4. These stations and areas were assumed to include virtually all O-D trip combinations that would require drivers to travel on route A or B.

Figure 5 shows the locations of the four permanent automatic traffic counters located within the study area and four other automatic counters installed for this project. A traffic-volume growth factor was developed based on traffic volumes collected from 1969 to 1977 at a counter located on route A near St. Mary's Street (Figure 5, counter A2). The 1969 through volumes from the planning survey O-D data were then extrapolated to 1977 conditions by using the growth factor.

License-plate O-D studies were conducted before the sign changes to estimate the distribution of through drivers between routes A and B. Study days and time periods were selected to include a good sampling of nonlocal drivers, because it was anticipated that this classification of drivers would be most influenced by the sign changes. The before O-D studies were conducted on Friday and Saturday, September 23 and 24, 1977, at 10:00-11:00 a.m., 1:00-2:00 p.m., and 3:00-5:00 p.m.

The O-D study stations for the before survey are shown in Figure 6. License-plate numbers of all NB

Figure 4. O-D stations and areas.

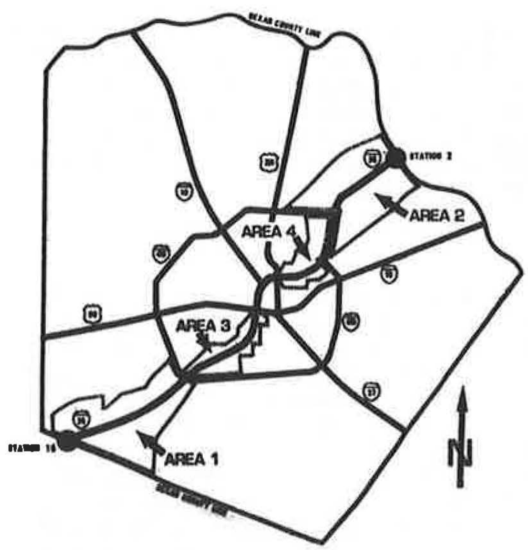
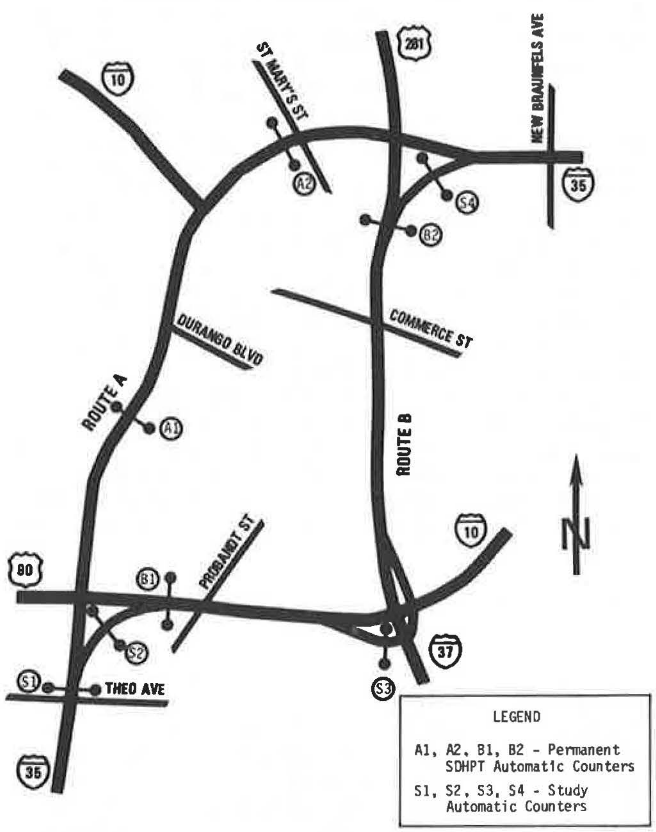


Figure 5. Automatic counter locations.



vehicles were recorded at the origin location of the study area (station A) and at the destination location on each of the two possible routes (stations B and C). At station C (the I-37 to I-35 connector ramp) personnel were able to read plate numbers from ground level and record them on cassette tape recorders. Stations A and B were high-speed, high-volume freeway locations (I-35 at Theo Avenue and I-35 at St. Mary's Street). At these stations personnel had to sit on overhead bridge structures and use binoculars to read the license plates. After the data were reduced from the tapes, the license-

Figure 6. License-plate O-D study locations.

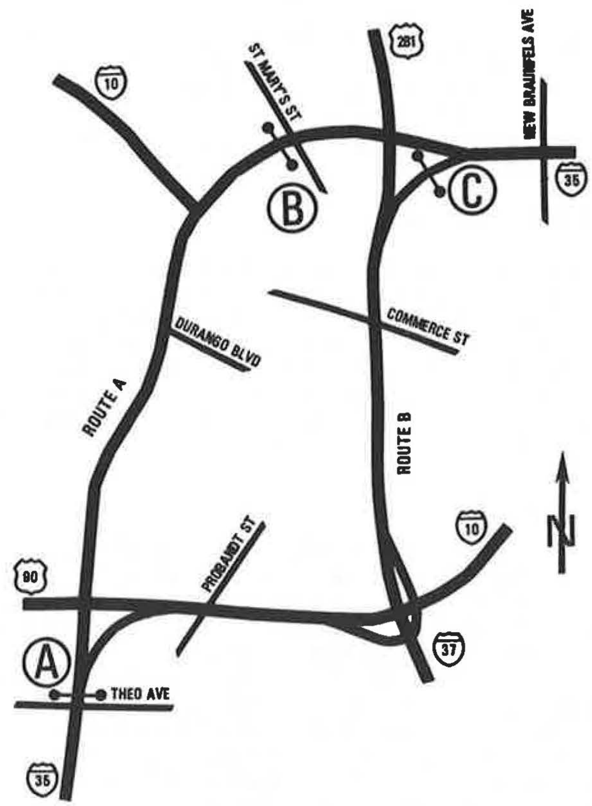


plate numbers for vehicles passing the two destination situations (stations B and C) were computer-matched against the plate numbers recorded at the origin (station A) to identify the total through traffic on each route. This technique was previously used successfully by TTI in similar studies conducted in Dallas (4).

Results

1969 Through Drivers

The results of the 1969 SABCUTS O-D data analysis are shown below. The analysis indicates that an average of 4811 vehicles/day (total both directions) traveled through the study area in 1969.

Trips	Number (N=4811)
External-external	
Between station 2 and station 15	388
External-internal	
Between station 2 and area 1	13
Between station 2 and area 3	213
Between station 15 and area 2	61
Between station 15 and area 4	133
Internal-internal	
Between area 1 and area 2	50
Between area 1 and area 4	187
Between area 2 and area 3	839
Between area 3 and area 4	2927

A conservative estimate is that 75 percent (388) of the actual 518 drivers that traveled between station 2 and station 15 used I-35, while 25 percent (130) traveled on the I-410 east loop.

Traffic Volume Growth

The annual average daily traffic (AADT) in the Austin-Laredo corridor for the years 1969 through 1977 was estimated from counts taken on route A near St. Mary's Street. The results show a change in AADT from 49 358 vehicles/day in 1969 to 61 085 in 1977—a 24 percent increase in volume.

Estimated 1977 Through Volumes

If it is assumed that the percentage increase in through trips was identical to the percentage increase in AADT, the 24 percent growth can be applied to the 1969 O-D data. This results in an estimated average of 5950 vehicles/day (total both directions) traveling through the study area in 1977. By further assuming an equal distribution of through trips in both directions, 2980 through vehicles/day are estimated in each direction.

License-Plate Study

Although the license-plate O-D technique used in this study increases the sample size compared to other study approaches, not all license plates can be read. This is because, among other things, capabilities and experience of the survey technicians vary. Thus, it was important to compare the actual volumes at the origin (input) station upstream of the I-35/I-10/US-90 interchange recorded by the survey party with those obtained from automatic counters located near the survey station (see Figure 5). Adjustments of survey counts could be made should any discrepancies among the automatic counts be noted.

A comparison of the two counts showed that the survey crew located at the freeway input station recorded on cassette tapes the licenses of an average of 88 percent of all vehicles recorded on the automatic counters. The license-plate volume data were therefore adjusted upward by 12 percent to obtain a more accurate estimate of the route choice by through drivers.

The results of the license-plate survey (Table 1) reveal that, on the average during the study period, NB through traffic represents 7.6 percent of the total traffic entering the area upstream of the I-35/I-10/US-90 interchange; 5.9 percent of the total entering traffic used route A, whereas 1.7 percent used route B.

As can be seen in Table 1, it is estimated that an average of 78 percent of all through drivers used route A, while 22 percent used route B before the sign changes. The distribution of through drivers by route is of primary importance, because these data reflect the volume of traffic that could potentially use redesignated route B.

Estimated Through Trips on Route A

It was noted earlier that 5960 estimated average through trips were made each day in 1977 (2980 in each direction). By using the route distribution found in Table 1, we can estimate that 78 percent, or approximately 4650 through trips/day, were made on route A. This volume amounts to approximately 7 percent of the total daily traffic on I-35 in the study area.

SHORT-TERM EFFECTS OF SIGN CHANGES

Object

Studies were conducted to determine the immediate effects of the sign modifications in terms of through drivers changing their route choices.

Approach

Short-term effects of the sign changes in terms of route choice were determined by conducting after studies of license plates on Friday and Saturday during the same time periods as the before studies. Because of the Christmas and New Year's holidays, the after studies were delayed until January 13 and 14, 1978, to reduce any possible bias in the after results.

Results

The license-plate O-D data were again adjusted to reflect the differences between the total freeway volumes obtained from the license plate survey and those obtained from the permanent counters. The license-plate freeway volume data represented an average of 89 percent of the total counted volume, compared with 88 percent for the before study.

A summary of the through traffic as a percentage of total NB traffic entering the study area is given in Table 2. The data show that during the after study the through traffic represented 8.3 percent of the total NB traffic; 6.0 percent used route A and 2.3 percent route B. These values reflect a 0.5 percent increase in the percentage of through traffic compared to the before study period. The data also show that in the after period 28 percent of the through drivers used route B, a 6 percent increase compared to the before study in which 22 percent of the through drivers used route B (Table 1).

The data were further analyzed to estimate the actual volume of traffic influenced by the sign changes (increased use of route B) during the study days in January. In order to estimate the volumes, two assumptions were made: First, it was assumed that the average percentage of arriving freeway traffic on NB I-35 determined from the license-plate studies to be traveling through the study area is valid for the entire day; second, the average percentage distribution of through drivers on routes A and B obtained from the field studies would hold true for the entire day.

Another factor that was considered was the seasonal variation in traffic volume between the study months (September and January). Thus, the volume data were normalized in terms of AADT. An estimate of the through traffic by using a specific route (ETT) during one of the four study days was obtained from the following equation:

$$\text{ETT} = \text{total traffic on I-35 at Theo} \times \text{seasonal correction factor} \times \text{fraction of total traffic using route} \qquad (1)$$

Data used in estimating the average daily change in through NB traffic on routes A and B are presented in Table 3. The total northbound volumes on I-35 were obtained from automatic counters located near Theo Avenue. Seasonal correction factors were computed from data documented in the SDHPT annual summary of freeway volumes (5). The fractions of total traffic using each route were derived from the license-plate O-D data summarized in Tables 1 and 2.

The results shown in Table 3 indicate that an estimated average of 2727 NB through vehicles/day (normalized to AADT) used route A during the September study days, whereas 2710 through vehicles/day traveled route A during the January studies—essentially no change. In contrast, 751 NB vehicles/day used route B during the September study days and 1028 vehicles/day during January—an increase of 277 vehicles/day (normalized to AADT).

If it can be assumed that the change in the SB direction is the same as that of the NB, then it is estimated

Table 1. NB through traffic before sign changes.

Study Period: Day	Time	Total NB I-35 Traffic at Theo (vehicles/h)	Through Traffic: No.[a]	Through Traffic: % of Total Traffic	Route A: No.[a]	Route A: % of Total Traffic	Route A: % of Through Traffic	Route B: No.[a]	Route B: % of Total Traffic	Route B: % of Through Traffic
Friday, 9/23/77	10:00-11:00 a.m.	2 210	130	5.9	100	4.5	77	30	1.4	23
	1:00-2:00 p.m.	2 530	133	5.3	111	4.4	83	22	0.9	17
	3:00-4:00 p.m.	3 020	248	8.2	175	5.8	71	73	2.4	29
	4:00-5:00 p.m.	3 130	252	8.1	197	6.3	78	55	1.8	22
Subtotal		10 890	763	7.0	583	5.4	76	180	1.6	24
Saturday, 9/24/77	10:00-11:00 a.m.	2 480	196	7.9	162	6.5	83	34	1.4	17
	1:00-2:00 p.m.	2 680	227	8.5	173	6.5	76	54	2.0	24
	3:00-4:00 p.m.	2 420	168	6.9	120	4.9	71	48	2.0	29
	4:00-5:00 p.m.	2 410	236	9.8	203	8.4	86	33	1.4	14
Subtotal		9 990	827	8.3	658	5.9	80	169	1.7	20
Total		20 880	1590	7.6	1241	5.9	78	349	1.7	22

[a] Adjusted.

Table 2. NB through traffic after sign changes.

Study Period: Day	Time	Total NB I-35 Traffic at Theo (vehicles/h)	Through Traffic: No.[a]	Through Traffic: % of Total Traffic	Route A: No.[a]	Route A: % of Total Traffic	Route A: % of Through Traffic	Route B: No.[a]	Route B: % of Total Traffic	Route B: % of Through Traffic
Friday, 1/13/78	10:00-11:00 a.m.	2 100	148	7.0	103	4.9	70	45	2.1	30
	1:00-2:00 p.m.	2 510	212	8.4	170	6.8	80	42	1.6	20
	3:00-4:00 p.m.	3 070	266	8.7	204	6.6	77	62	2.1	23
	4:00-5:00 p.m.	3 050	226	7.4	147	4.8	65	79	2.6	35
Subtotal		10 730	852	7.9	624	5.8	73	228	2.1	27
Saturday, 1/14/78	10:00-11:00 a.m.	2 270	176	7.7	128	5.6	73	48	2.1	27
	1:00-2:00 p.m.	2 690	236	8.7	176	6.5	75	60	2.2	25
	3:00-4:00 p.m.	2 510	237	9.4	167	6.6	70	70	2.8	30
	4:00-4:39 p.m.[b]	1 684	155	9.2	103	6.1	66	52	3.1	34
Subtotal		9 154	804	8.8	574	6.3	71	230	2.5	29
Total		19 884	1656	8.3	1198	6.0	72	458	2.3	28

[a] Adjusted.
[b] Tape recorder malfunctioned.

Table 3. Estimated NB through traffic during study days.

Study Day	Total NB I-35 Traffic at Theo (vehicles/h)	Seasonal Correction Factor	Route A: Fraction of Total Traffic Using Route	Route A: Estimated Through Traffic (vehicles/h)	Route B[a]: Fraction of Total Traffic Using Route	Route B[a]: Estimated Through Traffic (vehicles/h)
Friday, 9/23/77	47 430	0.975	0.054	2497	0.016	740
Saturday, 9/24/77	45 510	0.984	0.066	2956	0.017	761
Average				2727		751
Friday, 1/13/78	46 190	1.024	0.058	2743	0.021	993
Saturday, 1/14/78	41 220	1.031	0.063	2677	0.025	1062
Average				2710		1028

[a] Average increase in NB through volumes on Route B = 1028 - 751 = 277 vehicles/h.

that approximately 550 through vehicles/day on I-35 were influenced by the static sign changes during the January study days.

In summary, the before-and-after data revealed that the percentage of NB drivers traveling through the study area increased during the January studies. In addition, there was a significant increase in the number of through drivers that used route B.

CHARACTERISTICS OF THROUGH DRIVERS

Object

The object of this portion of the study was to determine the characteristics of through drivers in terms of familiarity and knowledge of alternate freeway routes around the downtown area.

Approach

Before-and-after questionnaires were mailed to NB through drivers identified from the license-plate studies. The questionnaires were coded by study time periods and driver travel route. Addresses of the through drivers were obtained from the Motor Vehicle Division of SDHPT. Unfortunately, addresses for out-of-state residents could not be obtained. In addition, questionnaires were not mailed to businesses and automobile rental companies because of the difficulty of establishing actual drivers of the vehicles.

It should be noted that the amount of license-plate data that was reduced from the tapes (45 000 plate numbers from the before study and 130 000 from the after study) dictated a relatively long time between conducting the study and receiving questionnaires from the drivers. This lag of from six to eight weeks may have diminished the individual drivers' ability to recall some particulars of the trips they made on the study day.

Results

Approximately one-fourth of all through drivers identified on the study days responded to the mailed questionnaires (25 percent in the before study, 24 percent in the after).

Frequency of Route Use

Questions related to frequency of use of routes A and B were included in the questionnaires in the belief that use frequency would reflect driver familiarity with each route, which would then give some clues as to the characteristics of drivers switching to route B after the sign changes.

Drivers were asked to indicate how often they used each route: from one to five times a week, from one to three times a month, less than once a month, or never before. It may be inferred that drivers who traveled the route one to five times a week could be considered as very familiar drivers, those using the facility from one to three times a month as familiar drivers, less than once a month as somewhat familiar, and never before as unfamiliar.

The table below compares driver familiarity based on the frequency of route use.

Familiarity with Route B	Familiarity with Route A: Very Familiar to Familiar	Familiarity with Route A: Somewhat Familiar to Unfamiliar
Before sign change (N = 394)		
Very familiar to familiar	62	8
Somewhat familiar to unfamiliar	21	9
After sign change (N = 405)		
Very familiar to familiar	57	8
Somewhat familiar to unfamiliar	21	14

The data reveal that there was a 5 percent reduction in the proportion of through drivers who may be considered very familiar or familiar with both routes (62 percent before, 57 percent after). Conversely, there was a 5 percent increase in the proportion of drivers somewhat familiar and unfamiliar with both routes between the before study (9 percent) and the after study (14 percent).

Because the freeway sign changes are directed at through drivers less familiar with the routes, the data indicate that the increased use of route B by through drivers after the sign changes is a result of a greater percentage of less familiar drivers traveling through the city during the after study. This indicates that the sign changes were successful in attracting through drivers to route B.

Local Versus Nonlocal Drivers

Another analysis was performed to determine which types of drivers (i.e., local or nonlocal) traveling through the city shifted to route B. The license-plate studies provided data about which route drivers selected, so plate numbers on each route could be matched with the addresses of the drivers obtained from the Motor Vehicle Division of SDHPT.

Those drivers residing in Bexar County were categorized as local drivers, whereas those living outside Bexar County were categorized as nonlocal drivers. Even though addresses were not obtained for out-of-state drivers, the mere fact that the license-plate numbers were available allowed these drivers to be included in the analysis. Thus, the analysis includes all the through drivers (license-plate matches) for both the before and the after studies.

The route selection, based on driver residence, is clearly reflected in the table below.

Route	Local Drivers (%): Before Sign Change (N = 1103)	Local Drivers (%): After Sign Change (N = 1166)	Nonlocal Drivers (%): Before Sign Change (N =298)	Nonlocal Drivers (%): After Sign Change (N = 317)
A	75	76	90	67
B	25	24	10	33

Before the sign changes, the route choice by local drivers traveling through the study area was 75 percent on route A and 25 percent on route B. After the sign changes, 76 percent of the local through drivers selected route A and 24 percent route B—a slight but probably insignificant increase toward route A. The results, however, show a definite increased use of route B by nonlocal drivers. The route selection by nonlocal drivers before the sign changes was 90 percent on route A and 10 percent on route B. After the sign changes, 33 percent of the nonlocal drivers traveled on route B during the study periods.

SUMMARY OF RESULTS

The results of the analysis of base conditions showed that there was an average of approximately 5960 through trips/day (total both directions). O-D studies showed that approximately 78 percent or 4650 of the through trips were made on route A.

The comparison of before and after license-plate O-D data revealed that, in the short term, approximately 6 percent of the through trips had shifted to the new route after the sign changes (route B: 22 percent before, 28 percent after).

Approximately one-fourth of all through drivers identified in the license-plate survey responded to questionnaires sent out after each of the two studies. The questionnaire studies showed that there was a 5 percent reduction in the number of drivers very familiar or familiar with both routes (62 percent of the before sample, 57 percent after). There was also an increase of 5 percent of drivers somewhat familiar or unfamiliar with both routes (9 percent before, 14 percent after).

When route choice was stratified by local versus nonlocal driver residence, it was found that route choice by local drivers remained fairly consistent. However, after the sign changes, 23 percent more nonlocals used route B than previously (10 percent before, 33 percent after).

We conclude that the redesignation of I-35 to the I-10/I-37 route significantly reduced expected volumes on the original I-35 route. An estimate of before and after study days indicated that approximate diversion for those days was 550 vehicles/day.

ACKNOWLEDGMENT

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paper does not constitute a standard, specification, or regulation.

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Improved Air Quality Through Transportation System Management

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Many cities must reduce total automotive emissions if they are to meet the national ambient air quality standards set by the U.S. Environmental Protection Agency under the authority of the Clean Air Act. This paper summarizes the results of two recent transportation air quality analyses in the Denver metropolitan area: first, an examination of implementation experience with six program measures in Denver's 1973 Transportation Control Plan and, second, a more in-depth examination of the potential role of parking management in reducing vehicle kilometers of travel (VKMT). Conclusions are that meaningful VKMT reductions are possible (in the order of 6-8 percent), that air quality measures are cost effective, that few real cost or administrative barriers exist to impede implementation, and that most measures are within the current authority of one or more agencies. These jurisdictions often overlap, and support action and institutional cooperation are therefore greatly needed. Successful implementation is impeded by political and institutional unwillingness to generate controversy or to go against vested interests that conflict with the agency's priorities.

To meet the national ambient air quality standards promulgated by the U.S. Environmental Protection Agency (EPA) under the authority of the Clean Air Act, many American cities will have to reduce total automotive emissions. The development and implementation of air quality transportation control plans, which began in 1973, has been a frustrating experience for most people. Too often, potential transportation measures to improve air quality are viewed as ineffective or not implementable. They are also considered as disincentives, incompatible with ongoing state and local programs, that will incur large direct and indirect costs.

Our own conclusions, however, are much more positive. The tight deadlines for the 1970 Clean Air Act and the severity of the air quality problem in many cities have combined to make it impossible to meet the ambient air standards on time, but both the transportation control plan requirement and, more recently, the consistency requirement of Title 23 have contributed significantly to the initiation of studies and the implementation of measures that will improve air quality. These requirements have forced state and local transportation agencies to give air quality explicit and thorough consideration and have prodded the agencies to take reasonable steps toward improvement.

The provisions of the 1977 amendments to the Clean Air Act and the resulting implementing guidelines (1) provide significant opportunities to build on previous successes and to accelerate implementation of measures.

The amendments, which provide new deadlines for attainment of the air standards, have set in motion a second generation of air quality transportation plans. Initial revisions to the state implementation plan were due on January 1, 1979; cities that cannot meet the standards by 1982 must complete a more systematic and comprehensive alternatives analysis by July 30, 1980 (1). Emphasis by both EPA and the U.S. Department of Transportation (DOT) is on a truly coordinated and integrated planning process in which air quality measures are routine actions undertaken by state, regional, and local agencies to better manage their multimodal transportation systems.

Two recent studies in Denver provide an opportunity to assess the realism of this objective and in particular to examine issues of effectiveness, cost, institutional acceptance, and consistency. The first study examined implementation experience with six program measures contained in Denver's 1973 Transportation Control Plan (2); the second is a more in-depth examination of the potential of one particular form of transportation system management—parking management—to contribute to improved air quality (3).

Our answer to the question of whether the second generation of air quality transportation measures will be more successful than the first is one of guarded optimism. A realistic appraisal simply does not support the negative impressions mentioned above. While there certainly are problems and dangers, we are hopeful that these can be overcome. Keys to success, though, are, first, positive, open attitudes on the part of all participants, especially city and urban area transportation officials. Also of great import is a participatory planning and implementation process that stresses the need for careful, high-quality design; systematically analyzes available alternatives and their potential impacts; and is supported by satisfactory analytical techniques rather than relying exclusively on subjective judgment.

NEW PROGRAMS VERSUS BETTER MANAGEMENT OF EXISTING SYSTEMS

Air quality transportation measures frequently are viewed as overlays of entirely new programs on top of already implemented urban policy. In reality, they should be viewed more as a periodic reappraisal of the continued desirability of existing policy. Responsible management of public resources requires maximizing efficient use of existing systems. In practical terms, this means looking at the movement of people rather than the vehicles in which they are placed.

Informal carpooling already is common practice in most cities; ride sharing that accounts for 20 percent of modal choice for the home-work trip is not uncommon. Employer-based and areawide ride-sharing programs are not new; they have built on already existing practice. Their object is to help people make better use of a rather expensive household commodity—the personal automobile—and to facilitate a wider variety of ride-sharing forms, such as vanpooling.

As a second example, parking management typically is viewed as something new and, as such, something to be feared. But we know of no city that has no form of a parking management program.

In Denver, as in most other U.S. cities, providing parking spaces has historically been a response to the demand for parking. If there has been a perceived need for vehicle parking, the spaces generally have been provided. Thus, commonly accepted policy has been to ensure an equilibrium between the demand for parking, both regionwide and in specified locations, and the amount of parking available in these areas. Extension of this policy in Denver, in conjunction with anticipated changes in population, income, and automobile ownership, has led to the estimate that by 1985 the current regional parking inventory should be expanded by over 300 000 spaces. Within the Denver central business district (CBD), parking would expand by 21 percent, or about 7300 spaces. This corresponds to a projected growth in regional population of 19 percent.

An important question of public policy in Denver and other urban areas, then, is whether it is desirable to continue these parking management policies by initiating construction of much new parking or whether in fact there are other more preferable policies.

Further, it is improper to view parking management primarily as a disincentive. Parking management can be broadly defined as the control of parking supply, location, or rates in a manner that

1. Affects parking in certain areas, during certain times of the day, or for certain purposes;
2. Encourages transit use or other ride sharing by providing incentives or a convenient gathering point at a peripheral location; or
3. Establishes review procedures and criteria for the construction of new parking facilities to meet a variety of goals, including minimization of carbon monoxide hot spots, conservation of energy, and improvement of urban aesthetics.

Thus, parking management is an umbrella category for a variety of measures such as park and ride, preferential parking for residents in neighborhoods, removal of on-street spaces in conjunction with a transit mall or automobile-restricted zone, parking supply freezes, price increases, long- versus short-term rate-structure changes, off-street parking bans, and restrictions on the size or location of new parking facilities. Parking management strategies are intended to help areas establish control over their parking supplies to meet a variety of development objectives. As a result, a parking management program may contain several elements and is not limited to negative or disincentive measures.

POTENTIAL EFFECTIVENESS

The initial transportation control plans and the initial rounds of transportation system management (TSM) plans generally were developed with little or no supporting transportation analyses. Today, however, the available analytical capabilities are sensitive to the kinds of low-capital, short-range policies being proposed. These techniques, unlike traditional aggregate urban transportation planning models, are referred to as sketch-planning techniques because they are low cost and have a short response time. The use of these new analytic techniques greatly increases the realism of a transportation analysis both by providing differentiation among alternate designs and by decreasing the need for overly simplistic assumptions.

The travel demand analyses in the two Denver studies were based on a set of disaggregate travel demand models that are policy sensitive to a broad range of socioeconomic, transportation, location, and mode-specific variables (Table 1) (5, 6). A complete set of behavioral decisions is examined (5) on a household basis (Figure 1):

1. Mode choice for the work trip for both primary and secondary workers, including drive-alone, public transit, shared-ride, and vanpool alternatives;
2. Non-work-trip frequency, destination, and mode choice, differentiating between shopping or personal business trips and social and recreational trips; and
3. Automobile ownership.

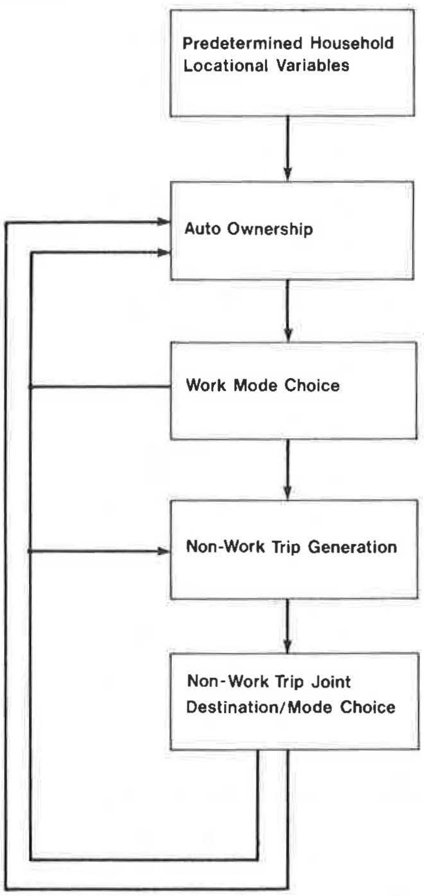
Appropriate submodels have been combined with the travel demand models to translate the predicted changes in travel behavior into changes in automobile emissions and fuel consumption. The emissions submodel predicts the amount of carbon monoxide, hydrocarbons, and nitrogen oxides emitted on a trip-by-trip basis as a function of vehicle-fleet composition, ambient temperature, trip length, cold start, and average speed and accumulates totals for these emissions on an areawide basis (5). The fuel-consumption submodel predicts the amount of gasoline consumed on a trip-by-trip basis as a function of the distribution of automobile types within the vehicle fleet, automobile occupancy (i.e., increased vehicle weight), the cold-start and trip-length factor, and average trip speed (5).

From the Denver home interview survey 2027 households were randomly selected, and costs and socioeconomic and transportation characteristics were updated to reflect both 1977 and 1985 conditions (5). Each

Table 1. Independent variables incorporated in travel demand model systems.

Variables	Automobile Ownership	Work-Trip Mode Choice	Nonwork Trip Generation: Shopping	Nonwork Trip Generation: Social and Recreational	Nonwork Destination and Mode: Shopping	Nonwork Destination and Mode: Social and Recreational
Socioeconomic						
Household income	X	X	X	X	X	X
Automobile ownership and availability	X	X			X	X
Primary worker		X				
No. of workers		X				
No. of nonworkers				X		
Household size	X	X	X	X		
Residence type	X					
Automobile ownership costs	X					
Level of service						
In-vehicle travel time		X			X	X
Out-of-vehicle travel time		X			X	X
Out-of-pocket travel cost		X			X	X
Walk versus automobile access to transit		X				
Location						
CBD destination		X			X	X
Employment density	X		X	X	X	X
Employment					X	X
Population density						X
Population						X
Composite						
Work-trip accessibility	X					
Shopping-trip accessibility	X		X			
Social and recreational trip accessibility				X		

Figure 1. Interrelations among travel demand models.



household's travel response to one or more candidate control measure then was simulated probabilistically by sequentially proceeding through the individual demand models (4).

To illustrate the kinds of analysis results that can be obtained, the potential travel effectiveness of alternate parking management strategies for Denver are summarized in Table 2. Each policy was described by identifying applicable changes in modal or parking availability or in transportation level-of-service variables. In the table the projected change in VKMT was tabulated in terms of a percentage change relative to

1. VKMT of the particular population group affected (i.e., CBD workers, those employed by large employers, etc.),
2. Regionwide work-trip VKMT,
3. Total regionwide non-work-trip VKMT for those measures affecting nonwork travel, and
4. Total regionwide VKMT.

Changes in emissions and fuel consumption are roughly similar, though generally somewhat smaller, and depend on the characteristics of the vehicle affected, trip lengths (i.e., cold-start effects), and vehicle occupancy. In addition, for carbon monoxide one should examine changes in emissions on other than a regionwide basis. This more detailed output, though available, is not provided as part of this paper.

An assessment of these particular parking management strategies, based on data collected and analyzed during the two studies, is provided in Figure 2. Overall findings of the travel behavior analyses include aspects of effectiveness, availability, mode choice, and measures analyzed and combined.

Effectiveness

The effectiveness of a particular parking management strategy in achieving reductions in areawide VKMT is directly related to two factors: the severity of the strategy and the number of people affected by the strategy. The parking strategies analyzed have produced regionwide work-trip VKMT reductions that ranged from 0.04 to 11.30 percent. Within the particular segment of the population most directly affected, however, reduction in VKMT ranges from 0.4 to 43.3 percent.

For example, the strategy that restricts the 10:00 a.m. occupancy of commercial parking facilities to 60 percent achieves a 3.1 percent decrease in VKMT for those work trips directly affected by this measure. In

Table 2. Effectiveness of parking management strategies in reducing VKMT.

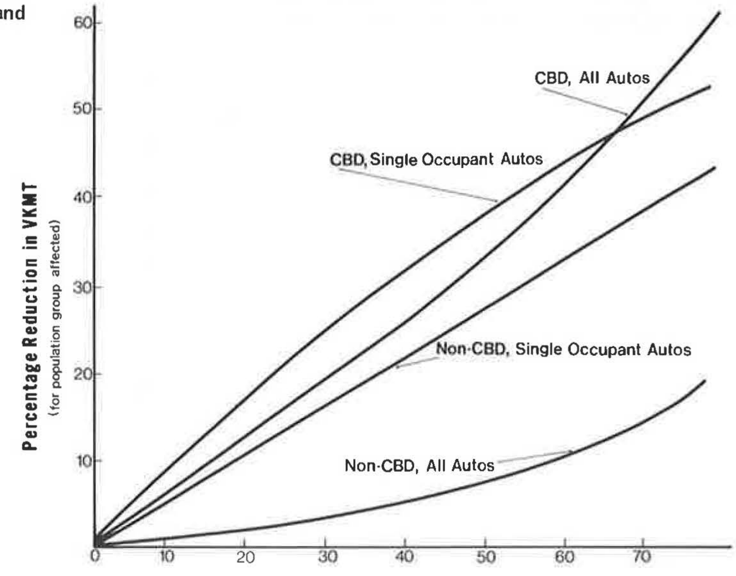
Strategy	Percentage Change in VKMT: VKMT of Group Affected	Areawide Work VKMT	Areawide Nonwork VKMT	Areawide Total VKMT
Short-term supply				
10:00 a.m. occupancy restricted at commercial facilities, 50 percent	-8.7	-1.0	-	-0.5
Employer-provided spaces restricted to HOVs[a] at large employers	-15.8	-4.1	-	-1.9
Long-term supply				
New parking construction restricted	-22.7	-6.8	-	-3.2
Pricing				
$3.00 parking charge at large employer-provided facilities	-3.2	-0.8	-	-0.4
100 percent price increases for long-term parking at commercial facilities	-2.4	-0.3	-	-0.13
Rate structure at commercial facilities altered to $4.00/day and $0.25/half hour	-3.7	-0.4	+0.13	-0.05
Parking charge for all parking of daily $1.00 surcharge/space	-0.9	-0.9	-1.8	-1.4
Ride-sharing incentives				
Preferential employer-based parking locations for HOVs	-3.4	-0.9	-	-0.4
Employer-based carpool program for employers of at least 50 employees	-3.1	-1.4	-	-0.7
For large employers of more than 250 employees	-3.1	-0.8	-	-0.4
Employer-based carpool-vanpool programs	-14.4	-3.7	-	-1.8

[a] High-occupancy vehicles.

Figure 2. Assessment of parking management strategies.

Assessment		Short Term Supply: 10 AM occupancy restrictions at all commercial facilities	Short Term Supply: Restrict parking to HOV at large employers	Long Term Supply: Restrictions on new parking constructions	Pricing: Parking charge at large employers	Pricing: Long term rate increase at commercial facilities	Pricing: Alter rate structure at commercial facilities	Pricing: Surcharge for all parking	Preferential Treatment: Preferential parking for carpools	Preferential Treatment: Employer based carpool program	Preferential Treatment: Employer based carpool and vanpool programs
Potential Effectiveness	Effective on an areawide basis		●	●							
Potential Effectiveness	Moderately effective on an areawide basis							●			●
Potential Effectiveness	Effective within individual market segments	●			●				●	●	
Potential Effectiveness	Not effective					●	●				
Ease of Implementation	Definitely practicable			●					●	●	●
Ease of Implementation	Potentially practicable					●	●				
Ease of Implementation	Probably not practicable	●	●		●			●			

Figure 3. Percentage reduction in work VKMT versus type and severity of supply restriction.



terms of areawide work-trip VKMT, this translates into a 0.34 percent reduction and is further diluted to -0.16 percent when expressed in terms of a percentage change in total areawide VKMT. By comparison, the pricing strategy that imposes a $3 daily parking charge at facilities provided by large employers achieves a similar percentage change in VKMT for that group affected (-3.2 versus -3.1 percent), but, because this group is more than twice as large, the effectiveness in terms of areawide work-trip VKMT is much greater (-0.8 versus -0.3 percent).

Availability

The availability of alternate modes of travel that offer levels of service comparable to that offered by the automobile is an important factor in determining the effectiveness of parking management strategies in reducing VKMT.

In situations where alternate modes are characterized by relatively poor service levels, travelers would resist a shift from automobile in response to parking management strategies much more vigorously than in those where travel alternatives with relatively good levels of service are available.

To demonstrate this effect, curves were developed that gave the percentage change in work-trip VKMT (relative to VKMT of the population group affected) as a function of increasing severity of parking management strategies. Using the accessibility of parking at the work site as one possible dimension of parking availability, separate curves were developed for each of two population groups: one with relatively good transit service (such as that available to those working within the Denver CBD), the other with relatively poor transit service (similar to that experienced by commuters working outside the CBD). Further, for each of these population groups, different curves were developed to represent measures aimed at all automobiles versus those affecting single-occupant automobiles only (Figure 3).

The most striking comparison is that between workers well served by transit and those poorly served for those measures that apply to all automobiles. In this case, the curves indicate that strategies aimed at those groups well served could be as much as five times as effective in reducing VKMT of the particular group affected as strategies reaching groups poorly served (i.e., reduced VKMT expressed in terms of a percentage change relative to the VKMT of the group affected). Note, however, that in most situations the group well served by transit will be much smaller than that not well served, and therefore the relative scale of these curves would be quite different if the percentage changes in VKMT were expressed in terms of areawide work-trip VKMT.

Another interesting comparison can be made between supply restrictions applied to all automobiles versus single-occupant automobiles only for that group receiving relatively poor transit service. In the former situation, commuters have no good alternative and, therefore, continue to use the automobile mode despite severe decreases in levels of service. On the other hand, if only single-occupant automobiles are affected, carpooling appears as an increasingly attractive alternative as supply restrictions increase in severity for single-occupant automobiles.

Mode Choice

Choice of mode for work travel (and therefore work-trip VKMT) appears to be relatively insensitive to most of the strategies that are designed to discourage automobile use by making parking more expensive or less conveniently located or both. On the other hand, strategies that limit the number of spaces available can be quite effective.

The effectiveness of parking supply constraint measures ranged from a 0.4 to a 24.3 percent reduction in VKMT relative to the group affected. For those measures designed to discourage automobile use, VKMT reductions ranged from 0.5 to 3.7 percent (relative to VKMT of the particular group affected).

Measures

The parking measures analyzed result in transit ridership increases varying from 0.6 to 43 percent for the trip between home and work, relative to a 1977 base modal share of 3 percent. The majority of the policies analyzed, though, produced increases in transit ridership of less than 10 percent.

Other modal use changes occur as well with a maximum of a 62 percent increase in ride sharing and an ab-

Figure 4. Percentage reduction in work VKMT based on effectiveness of combining employer-based incentive and disincentive measures.

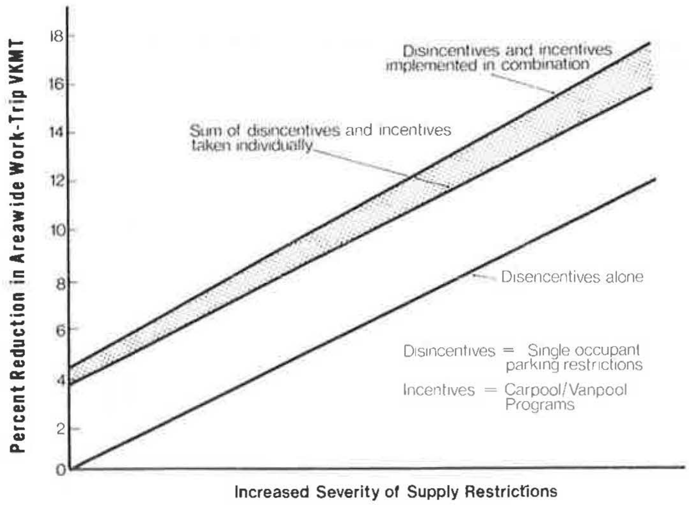


Table 3. Cost-effectiveness of ride-sharing projects.

Location	Annual Project Cost ($)	Estimated Annual Cost/New Carpooler ($)	Estimated Cost/New Carpooler Trip[a] ($)	Estimated Cost/Vehicle Kilometer Reduced ($)
Tucson[b]	58 000	7	0.015	0.005
Los Angeles	660 000	85	0.18	0.143
Sacramento	85 000	32	0.07	0.018
San Diego	210 000	98	0.21	0.048
Denver	125 000	88	0.19	0.068
Connecticut	65 000	23	0.05	0.008
Boise[b]	45 000	75	0.16	0.069
Louisville[b]	65 000	9	0.02	0.008
Boston[b]	325 000	37	0.08	0.034
Minneapolis	60 000	13	0.028	0.005
Omaha	84 000	69	0.15	0.061
Raleigh	20 000	26	0.06	0.029
Portland[b]	190 000	26	0.06	0.021
Pittsburgh	134 000	71	0.15	0.055
Rhode Island	70 000	46	0.10	0.026
Dallas	60 000	38	0.08	0.024
Ft. Worth	30 000	15	0.033	0.011
Houston	220 000	112	0.24	0.061
San Antonio	160 000	34	0.07	0.027
Seattle	215 000	99	0.22	0.103
Washington, D.C.	110 000	11	0.024	0.010
Milwaukee[b]	100 000	12	0.027	0.016
Average[c]	140 000	47	0.10	0.039

[a] At 2 trips/day for 230 days/year, or 460 annual trips to or from work.
[b] Based on broad impacts of ridesharing programs; impacts for other locations are directly attributable to carpool matching.
[c] Arithmetic averages of the individual city data. If averages are computed from total annual project costs for all above cities divided by number of new carpoolers and annual VKMT reductions, the cost-effectiveness indicators are $35/new carpooler, $0.75/new carpooler trip, and $0.014/VKMT reduced.

solute vanpool modal share of 2 percent.

Measures can also be combined into program packages that are more effective in terms of reducing VKMT than the sum of the individual measures. For example, if employer-based carpool and vanpool programs are combined with measures restricting employer-provided spaces to carpools, the resulting percentage change in VKMT is about 14 percent greater than the summed VKMT reduction of the measures taken individually.

This is demonstrated by the three curves shown in Figure 4, which relate a percentage change in areawide work-trip VKMT to increasing severity of supply constraints. The lower curve represents the effectiveness of the supply restrictions alone; the middle curve represents the summed effect of the supply restriction and employer-sponsored carpool and vanpool programs, if we assume that each is implemented individually and that the two programs do not interact. The upper curve represents the effectiveness of these measures implemented in combination; the shaded area represents the increased effectiveness attributable to the synergistic effect of the combined implementation of these measures.

PROGRAM COSTS

Many VKMT reduction measures are relatively inexpensive, and most are within the current authority of one or more agencies (3, 6). This does not necessarily imply, though, that successful implementation can be easily accomplished; a variety of other administrative and institutional barriers may exist.

A legal authority for implementation of parking measures is the police power delegated by states to local areas to protect the public health, safety, and welfare. Zoning ordinances and on-street parking regulations are commonly accepted uses of this police power.

Most of the parking measures analyzed in Denver can be planned and implemented within the existing staff resources of Denver-area city agencies (4). While costs of enforcement vary according to design details, consideration of enforcement aspects at the time of planning will increase the likelihood that low-cost, self-enforcing designs will be developed. As one example, long-term limitations on parking supply emerged as an effective policy on an areawide basis and would involve minimal enforcement costs, since violations would be in the form of unauthorized construction rather than daily-use violations.

An examination of ride-sharing programs performed by Wagner (5) demonstrates the relatively good cost-effectiveness of a second type of TSM policy. The Denver Regional Council of Governments has been funded at an annual average level of approximately $125 000, which corresponds to an annual cost of $0.10 per capita.

As shown in Table 3, this corresponds to funding levels in a number of other urban areas. Shown are various cost-effectiveness measures for 22 of the Federal Highway Administration's carpool demonstration projects. For example, annual project cost per new carpooler averaged $47 for the cities shown. Most of the available impact data, however, include only direct impacts of carpool matching. For the six cities where broader impact estimates were made, the cost-effectiveness indicators—$28 annually/new carpooler, 6 cents/new carpooler trip, and 2.7 cents/VKMT reduced—are much better.

For purposes of comparison with a capital-intensive transportation measure, a recent analysis of one proposed heavy rail rapid transit extension (not in Denver) derived an annual cost per new transit passenger in the range of $3000. This does not necessarily imply that more capital-oriented transit expenditures are a poor investment but rather that many air quality transportation measures are highly cost-efficient means of improving the short-range management effectiveness of existing transportation systems.

Transit, if properly designed and coordinated, may contribute to a number of long-range objectives not only for air quality improvement and energy conservation, but also for improved spatial distribution of urban activities and a stronger long-range economic infrastructure (2).

INSTITUTIONAL RESPONSIBILITIES

Given an assessment of the effectiveness of a TSM measure in changing travel behavior patterns, it is equally important to examine institutional, legal, enforcement, and other administrative issues that would be associated with implementation to determine a measure's actual potential (Table 3). These considerations influence not only the overall effectiveness of a program of action but also the acceptability of the measures to the public and to the various institutions and interest groups involved in the implementation effort.

Whereas a variety of transportation measures can be shown to be cost effective in terms of improving air quality, experience in Denver and elsewhere has shown that the primary barriers to successful implementation can be characterized as being institutional or political in character (3). For example, only one of the alternate parking management strategies examined—restrictions on new parking construction—that proved to be effective on an areawide basis also was judged to be definitely practicable in terms of ease of implementation (Figure 2).

One of the difficulties in implementing transportation management programs for air quality improvement (as well as for energy conservation) is that a sizable number of the population, as well as many agency officials and elected representatives, do not understand the nature of air pollution and are not convinced that a problem exists or that the problem is serious enough to justify special action. However, well-designed public information programs can be combined with the actual planning and implementation of control measures in order to increase general awareness and appreciation of air quality as an issue. This consciousness raising may in turn stimulate additional voluntary action that will contribute to air quality goals.

The lack of consensus on growth and development policy for the Denver region, as evidenced by recent intense debates over highway, transit, and water projects, has further complicated all planning and implementation efforts, including population and land-use forecasting, highway and transit systems development, and water resource and air quality programs. Competing recommendations on each of the programs and the state of flux inherent in a rapidly growing region have made the always-difficult job of predicting future conditions especially fraught with uncertainty. This uncertainty further complicates the problem of getting support for a particular course of action.

Many air quality transportation measures do not easily coincide with the traditional way in which the state, regional, and local transportation agencies have done things, which necessitates a degree of change in organizational procedures, responsibilities, and objectives. Experience has shown that these changes may be actively and successfully resisted by numerous forces.

The requirements for institutional cooperation often are relatively greater for air quality transportation measures, particularly those that are more innovative, than they are for many large-capital projects. Jurisdictions often overlap and support actions often are needed. The cost of administration and coordination also can be a higher percentage of the total costs of such a project than of a more traditional highway or transit project. These factors tend to make agencies reluctant to take on responsibility for measures that are intended primarily to improve air quality. They also make it easier for agencies to pass the buck with regard to implementation responsibility.

Because air quality transportation measures are usually both short range and low capital, they often are viewed as being simple. Experience has shown just the opposite: Each measure has numerous design details and problems to be worked out that require the skills and knowledge of a trained professional.

Active, inspired leadership and continued follow-up by a small number of individuals are needed to get a measure going and to keep it going. Unfortunately, these conditions may not exist, and even when they do success may be slow. Barriers exist in terms of political and institutional unwillingness to generate controversy or to go against vested interests and in the sense that implementation of the measures may not match the cognizant agency's priorities.

These general observations are derived from an examination of Denver-area institutional responsibilities and responses but are felt to be equally applicable to many other large urban areas as well (2). Before the Denver region's present effort on the state implementation plan revision, operations of the Denver Regional Council of Governments (DRCOG), the Regional Transportation District (RTD), the Colorado Department of Highways (CDH), the city and county of Denver, and the Colorado Department of Health, the state's air quality agency, were characterized by an atmosphere of cooperative autonomy. The agencies' activities and plans occurred largely independently of each other. While recent changes in Denver's institutional arrangements are improving this coordination, it is still too early to determine the degree of long-term success associated with effective implementation that will actually be achieved.

Each of the Denver area's transportation agencies that has a responsibility in transportation and air quality has had its own agenda of actions programmed for implementation, some of which are compatible with air quality objectives. These agencies have multi-million-dollar annual budgets, and implementation of transportation air quality measures is within their financial capabilities, although in most cases it would require shifting their priorities and reallocating existing staff resources. The agencies frequently have been unwilling to change their priorities to projects that would improve air quality, particularly when such measures are not consistent with agency objectives.

Many potential transportation air quality measures, particularly automobile disincentives, are perceived as having a low level of public acceptability. Agencies with high visibility and accountability, such as a mayor's office, are skeptical about implementing such measures because of anticipated adverse public reaction. Thus, many transportation measures that initially appear to be implementable in the short run can become long-term efforts when institutional and political factors are taken into account.

THE FUTURE

The 1977 amendments to the Clean Air Act and the resulting implementing guidelines jointly developed by DOT and EPA acknowledge both the potential effectiveness of air quality transportation measures and the institutional problems that can impede their successful implementation. Methods for avoiding many of the problems faced by previous transportation control plans are provided. The responses of city transportation officials and metropolitan planning organizations (MPOs) to these opportunities will in large part determine the degree to which future efforts will be more successful than those during the past five years.

Air Quality Transportation Planning Guidelines

The amendments and resulting guidelines emphasize the establishment of a process for transportation air quality planning characterized by leadership responsibility at the regional level; shared and agreed-on state, regional, and local authorities and obligations; requirements for consideration of alternative measures and programs and for evaluation of social and economic impacts; a standard of reasonable incremental progress in implementing measures to improve air quality; and shared federal responsibility for ensuring that transportation air quality measures are in fact introduced.

In addition, this process is to be integrated with other metropolitan planning efforts, and the assessment of the results will no longer be made on the basis of a separate transportation air quality plan but instead on the overall achievements in air quality improvement taking all relevant activities into consideration.

The emerging process, then, would have self-enforcing provisions for the compatibility of air quality planning and transportation planning: The separate state implementation and transportation plans would be, simply, documentation of the same planning process and implementation activities. And the process would include a system of procedures, incentives, and sanctions designed to ensure that reasonable progress in implementing air quality improvements actually would occur.

Changing Role of the Urban-Area Traffic Engineer

If the objective of shifting away from merely considering air quality to actual implementation of air quality improvement measures is to be achieved, it is essential that those who have traffic engineering and transportation responsibilities in a metropolitan area assume major and visible leadership positions. In most large urban areas, the city traffic engineer's role has evolved into that of the principal transportation advisor to the mayor. Concerns have shifted from traditional intersection signalization, vehicle flow, and on-street parking to those associated with the overall movement of people, including transit, off-street and residential-area parking, preferential treatment for particular vehicle types, and the distribution and density of spatial activity patterns. To cite but four examples: Boston's automobile-restricted zone is being coordinated by the mayor's commissioner of traffic and parking; Cambridge's director of traffic and parking is responsible for administering on-street commuter parking restrictions in residential neighborhoods; Seattle's ride-sharing and parking management programs are being managed by their traffic engineering office; and the city of Berkeley has an appointed transportation commission that has helped develop that city's successful programs of transportation for the handicapped and restricted automobile movement in residential areas.

Mentioning these broader transportation responsibilities does not imply, however, that traditional traffic engineering measures cannot also contribute to improved air quality. An important focus of concern in developing revised state implementation plans will be the identification and correction of carbon monoxide (CO) "hot-spot" problems. Unlike photochemical oxidants that require areawide measures, violations of CO standards frequently occur on a highly localized basis. Because vehicle CO emissions decrease with vehicle operating speed and smoother traffic flow, traffic engineering design changes that affect the operation of streets, intersections, and parking facilities can make an important contribution to achieving ambient CO standards on a localized level.

An integrated program of localized traffic engineering improvements that concentrates on arterial streets may lead, as well, to overall regional air quality improvements. For example, in Denver an approximately 12 percent decrease in regionwide automobile travel-time rates results in about a 3 percent short-term decrease in vehicle emissions. This takes into account the 1 percent projected increase in VKMT as a result of an improved level of service and leaves a net improvement in emissions comparable to that obtainable with many other types of air quality transportation measures.

MPO Capabilities and Relations with Other Agencies

Many MPO staffs have been limited to performing feasibility studies and conducting certain areawide planning efforts that give primary emphasis to summarizing and compiling planning and programming activities of local and state agencies and operators. State and local agencies retain the primary responsibility for implementation.

In order to achieve the goal of regional and subarea air quality improvement, it will be necessary to mobilize these disparate agencies and organizations for coordinated action. As planning agencies, MPOs simply do not have the authority or the mandate to direct other agencies to conduct studies or implement measures. Therefore, if coordinated action on air quality is going to occur, it will be largely dependent on the MPO in its role as a consortium of local (and state) officials, orchestrating an agreement that the members individually will assume the appropriate responsibilities. MPO actions including "lend-a-planner" programs, pass-through funding earmarked for particular studies or projects, and MPO-conducted studies of promising actions would be valuable. Nevertheless, the proposed process will necessitate goal-oriented commitment and responsibility on the part of local agencies and officials to a greater degree than has typically occurred in the past, plus stronger direction from the MPO as a forum and as a planning staff than has been common.

In summary, the outlook for improved consideration of air quality in transportation planning is promising, but there remains the potential for controversy and conflicting objectives. The emerging process should correct many of the problems that occurred in the past, but a great deal of work is ahead for regional, local, and state agencies and officials in establishing the process and for EPA and DOT in managing it.

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Results of Implementing Low-Cost Freeway Incident-Management Techniques

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The purpose of this paper is to report on the demonstration of a low-cost freeway incident-management methodology. The results of the demonstration indicated that the delay caused by accidents, spills, breakdowns, and other incidents was reduced by 45 percent by implementing three different solutions. This paper presents before-and-after incident data and estimated versus actual delay savings. The major finding of this research was that the method developed to estimate delay caused by incidents and to evaluate proposed solutions is appropriate and merits further testing.

One of the research projects in the federally coordinated program of research and development deals with analysis and remedies of freeway traffic disturbances (1). It is concerned with the planning, design, and operation of traffic-responsive incident-management systems. Emphasis has been placed on the use of low-cost freeway incident-management systems as well as on the more expensive electronic surveillance and control systems.

It is known that freeway incidents occur with sufficient regularity to be considered a major problem. In fact, approximately half the delay on urban freeways is caused by unexpected incidents such as spilled loads, collisions, and stalled vehicles (2). It is estimated that in the United States motorists lose 750 million vehicle-hours/year waiting for freeway incidents to be cleared (3). Although such incidents occur randomly with respect to time and place, they are predictable in the sense that they will occur sufficiently often during peak-period flow conditions to further complicate the continued movement of traffic in the traffic stream.

One of the advantages of using low-cost freeway incident-management approaches is that, by using preplanned incident-response procedures, the delay in detecting, responding to, and removing the incidents can often be significantly reduced. Electronic surveillance and control systems may also achieve the same end result. However, many agencies may not have the extensive freeway distance and associated operation problems needed to justify the installation of these advanced systems.

The purpose of this report is to describe the selection, planning, operation, and results of several low-cost technical options for providing for freeway incident management (FIM). These particular options and the deployment methods used for them were developed jointly by the Federal Highway Administration (FHWA), Florida Department of Transportation (DOT), and Peat, Marwick, Mitchell and Company in a project on alternative surveillance concepts and methods for freeway incident management. This report describes the specific application of three low-cost freeway incident management options evaluated on the I-275 Howard Frankland Bridge (HFB) in Tampa, Florida.

HISTORY AND SITE DESCRIPTION

HFB, which spans upper Tampa Bay, was built in 1959 as part of I-4 and is now part of I-275. It serves as the major artery between Tampa and St. Petersburg, Florida. Since its completion, the bridge has had a history of traffic problems such as high accident rates, insufficient servicing of disabled vehicles, and long delays associated with capacity-reducing incidents. The bridge, 4.8 km (3 miles) in length, has 3.2 km (2 miles) of causeway at each end and is a four-lane structure with two 3.7-m (12-ft) lanes eastbound and two 3.7-m lanes westbound.

Since 1959, a barrier wall has been built to separate the two directions of traffic, a stalled-vehicle warning system has been implemented, a glare screen has been installed atop the barrier wall, and the highway lighting has been upgraded and improved. These improvements have had a positive effect on reducing accidents on the bridge, but a significant problem still exists when an incident occurs.

The general feeling of both the public and the Florida agencies, whose operational responsibilities include the bridge, has been that the number of accidents occurring on HFB is not the total problem. From the public agency's point of view, all reasonable technology short of continuous electronic surveillance has been applied to prevent accidents: The median barrier was constructed to prevent head-on collisions and the overhead motorist-actuated signs aided in the reduction of rear-end collisions. However, as was found during subsequent interviews with users of the facility, most of them still considered the HFB a significant bottleneck and felt that action should be taken to further minimize the problems associated with incidents.

In 1977 the Florida DOT, in response to the public's desires, began to plan and program for a surveillance-and-control system (SCS) for HFB. They published a request for engineering services that indicated that the consultant would be required to design and optimally integrate into a total SCS the following items:

1. Lane-use signals,
2. Speed and condition displays,
3. Incident-detection equipment,
4. Visibility-detection equipment,
5. Television surveillance equipment,
6. Route-diversion displays,
7. Communication equipment, and
8. Central control equipment.

The budget for the project was $1 000 000, and the project was scheduled for construction in FY 1979/80. From preliminary plans to final construction, the installation was to take about two years. This planning and construction time was offered to FHWA as a possible testing period for low-cost FIM options. FHWA recognized that this opportunity had advantages: An identified problem did exist even though it was not quantified in terms of vehicle hours of delay; the ensuing construction project would probably add to the congestion problem; and to justify the SCS project Florida DOT would carry out a significant data-collection effort, which could be valuable to the FIM pilot test.

Florida DOT at the same time wanted an immediate solution to the problem, which the FIM project had the potential of achieving, and realized that the FIM study results might affect the design of the SCS project. The potential existed for both agencies to realize significant benefits. Therefore, an agreement was reached on conducting a demonstration of low-cost FIM techniques on HFB.

THE DEMONSTRATION

Committee Formulation

The demonstration of low-cost FIM options or solutions began with an examination of the problem as described in the previous section and a comparison of the problem with the options developed by the project (4).

Based on Florida DOT's knowledge of the problem, the options were reduced to the first 15 in the list below.

1. Increased police patrol frequency
2. Aircraft
3. Dedicated freeway patrol
4. Fast vehicle removal
5. DOT service patrol
6. Accident investigation sites
7. Hazardous materials manual
8. DOT communication training
9. CB radio (stationary)
10. Rest area telephone
11. Media ties
12. Private services coordination
13. Traffic operations training
14. Dispatcher's manual
15. Alternate route planning
16. Peak-period patrol motorcycles
17. Call boxes
18. Transit ties
19. Professional observers
20. Closed-circuit television
21. Loop detectors
22. Volunteer observers
23. Citizen service patrol
24. Flashing-lights policy
25. Highway agency relationship
26. Wrecker contracts
27. Other public agency relationships
28. Citizen group liaison
29. Police relationships

This list was then presented to a Tampa-based technical advisory committee made up of representatives of 13 local organizations directly or indirectly affected by the FIM problem. The committee was asked to review this list, generate new options, and assist with data collection.

Data Collection

Data collection for the project had several purposes. The most obvious was the need to evaluate the magnitude of the existing problem. Of equal importance was the need to support or refute the need for the indicated options. Finally, the data-collection effort was to generate new options if necessary.

The project team collected data by having certain committee members review their historical files; by having the Florida Highway Patrol collect real-time data; by using temporary electronic sensors, time-lapse photography, and floating automobile runs; and by interviewing users (limousine operators, taxi drivers, tow operators, etc.) of the HFB. The quantitative findings of this data-collection effort are summarized in Tables 1 and 2 and the table below.

Category	Result
Average detection time, min	6.0
Average response time, min	13.5
Average clearance time, min	12.0
Approximate beat duration, min	60.0
Approximate wrecker response, min	30.0
Average daily traffic, vehicles	50 000
Peak flow, vehicles/h	2000-2500

The data-collection effort eliminated the first eight options in the list and generated two additional ones. The details for eliminating each can be found in Urbanek and Colpitts (5). Essentially, they were eliminated because the field data indicated that they were not problem areas or that they were impractical for the HFB site. The two new options were constructing stopping bays on the bridge and creating an underpass for Florida Highway Patrol vehicles beneath the approaches.

The fast-vehicle-removal option was defined to mean a law that would make it a civil offense to run out of fuel on HFB.

Option Evaluation

The remaining options to be evaluated were to contribute to the solution of the HFB FIM problem in some manner (Table 3). An indication of how each would contribute as well as a description of the option itself follows.

Bridge Underpass

A bridge underpass would be constructed for official vehicles as a roadway for traveling beneath the ends of HFB. Thus the highway patrol's response time to incidents would be shortened.

Mobile CB Radios in Patrol Vehicles

Highway patrol vehicles would be equipped with CB radios so that detection time could be improved. More on-site incident information would also be obtained before arriving at the incident scene. This additional information would have the potential of expediting detection and response times and reducing false alarms and gone-on-arrival incidents.

Table 1. Incident causes.

Incident Type	Number	Percentage	Direction	
			Eastbound	Westbound
Gas and fuel related	17.5	37	10.5	7
Tire	14	30	8.5	5.5
Cooling system	8	17	4	4
Mechanical	1	2	0	1
Accident	2	4	1	1
Other	4.5	10	2	5
Total	47	100	26	23.5

Rest Area Telephone

Telephones would be installed in the rest areas immediately off the ends of HFB in the causeway area. This option would affect the detection time of passing motorists.

Fuel Law

An ordinance would assess fines on motorists if they were stopped on HFB by lack of fuel. This action would probably reduce the number of fuel-related incidents.

Traffic Operations Training

Training of Florida troopers and DOT maintenance personnel would focus on minimizing clearance time. This option involves training centered on the material found in Urbanek and Owen (6).

Alternate Route Planning

Trail-blazer signs would clearly mark alternate routes to be used in lieu of HFB. This option would potentially reduce the number of vehicles using HFB during an incident.

Media Ties

A direct communications link would be created between the patrol dispatcher and the media. The most important aspect of this option would be the potential of causing motorists to take alternate routes based on radio broadcasts. This option may also affect detection time, in that certain media use aircraft to make traffic reports.

Stopping Bays

Permanent structures would be constructed on HFB that would store disabled vehicles until after the peak period. This option can potentially reduce the total number of incidents.

Table 2. Incident report sources.

Source	Incidents: Actual No.	Actual Percentage of Total	False No.	False Percentage of Total	All Reported No.	All Reported Percentage of Total
CB radio	19.5	71	8	29	27.5	39
Commercial radio station	12.5	71	5	29	17.5	25
Passing motorist	12	52	11	48	23	32
State trooper	3	100	0	0	3	4
Other	0	0	0	0	0	0
Total	47	66	24	34	71	100

Table 3. Option evaluation.

Option	Assumed Impact	Annual Delay Reduction (vehicle hours)	Estimated Capital Cost ($)
Bridge underpass	Reduce response time 4 min	59 308	20 000
Mobile CB radios in patrol vehicles	Reduce detection time 1 min	16 600	600
Fuel law	Reduce incident rate 17 percent	48 371	7 000
Rest area telephone	Reduce detection time 1 min	14 177	1 000
Traffic operations training[a]	Improve clearance time	-	-
Alternate route planning[b]	Reduce demand	-	-
Media ties	Reduce demand 5 percent	46 000	1 000
Stopping bays	Reduce incident 34 percent	94 082	1 000 000

[a] Not evaluated due to lack of data pertaining to the option's merits.
[b] Would be implemented with the media-tie option. The benefits would have to be allocated between the media-tie and alternate-route options.

Evaluation Method

Each option was evaluated by means of a delay computation worksheet that was developed during earlier phases of the FHWA contract. Urbanek and Bruggeman (7) fully describe the mechanics of the evaluation. Each option was evaluated under the assumption that it would improve some aspect of the base-case problem. The major assumed impacts are indicated in Table 3. This list of options was presented to the committee, who in turn ranked the options in the order of preference indicated in the first column of Table 3 and changed the focus of two options as described below.

The committee redefined the CB option because it was highway patrol policy not to have CB radios in police vehicles. Instead, the committee felt that the CB motorists should be made aware by information signing of the fact that the patrol dispatcher monitored channel 9. Therefore, the redefined CB option involved constructing signs to inform CB users that channel 9 was being monitored.

The fuel law option as originally presented involved creating an ordinance whereby it would be a civil, fine-receiving traffic offense to become disabled on HFB for running out of fuel. Although Dade County, Florida, has such an ordinance, the committee felt that the advantages of such a law are not worth the effort required to administer it. The committee also expressed the opinion that such a solution was not in keeping with the symptoms of the problem. Several committee members felt that the lack of proper exit and information signing was one of the primary contributions to the problem. The committee rated this option third priority. As revised, it involved constructing information signs warning motorists of the distance to the next fuel area.

IMPLEMENTATION CYCLE

The funds for this demonstration project were limited, and FHWA and Florida DOT wanted to be able to measure the impact of each option, so the first three options of Table 3 were chosen for implementation. This choice stayed within the budget and offered independent measurement, in that each option affected a different aspect of the problem: The underpass was to affect response time, the fuel signs the total number of incidents, and the CB signs detection time and false alarms.

Implementation of the options required six months for the signs and an additional four months for the underpass option. This cycle took somewhat longer than estimated but was still within a reasonable construction timetable.

EVALUATION

Post-implementation data were collected by means of incident logs kept by the highway patrol. The logs provided for response time, incident-type distribution, clearance time and recorded request, and arrival and departure times for all other service vehicles.

An analysis of the data revealed that fuel-related incidents decreased enough to cause the total incident rate to decrease by 21 percent, response time to decrease by 7 min, clearance time to decrease by 3 min, and the gone-on-arrival and false-alarm rate to decrease by 4 percent.

The effect of these changes is indicated in the table below, which compares the predicted estimated impact of the option of the pre-study with the post-study results.

	Annual Delay Reduction (vehicle hours)	
Option	Estimated Pre-Study	Actual Post-Study
Bridge underpass	59 308	39 147
CB signs	16 600	48 982
Fuel signs	48 371	46 989
Combined effect	109 471	124 388

Of particular significance is the fact that the estimate of the combined effect of the options is within about 13 percent of the actual. Of equal importance is the fact that the low-cost FIM options reduced the annual vehicle hours of delay from an estimated 276 404 to 152 066, a reduction of about 45 percent.

Predicted Versus Actual Results

The fact that the predicted results vary with the actual results merits explanation.

The variance between the underpass post- and pre-study values can be explained by the difference in response times. The estimated reduction in response time was 4 min, while the actual response time was reduced about 7 min. However, this response time in itself is confounded by the fact that, as will be indicated below, the fuel signs also contributed to the response reduction. This occurred because the fuel signs reduced the incident rate, which allowed the troopers to spend more time patrolling, which in turn allowed them to detect more incidents and thus reduce total average response time. But the average detection time increased by 5 min because they spent more time detecting incidents.

This chain reaction was unanticipated and the fuel signs were installed before the underpass was built. Therefore the after data-collection mechanism was not able to differentiate between response reduced by signs or by the underpass. In an attempt to compare similar before and after effects, all response time was assigned to the underpass option.

The fuel signs, which said Long Bridge Ahead—Check Gas, were designed to reduce the number of incidents caused by vehicles running out of fuel. The gasoline-caused incident rate dropped from 37 to 20 percent, which translates into a normalized reduction in the total incident rate of 21 percent. Therefore, the primary reason for the difference between the predicted and actual study results was the fact that the predicted estimate was a 17.5 percent reduction.

The CB sign option was aimed at improving detection time, reducing false alarms, and giving the troopers more information before arrival at an incident site (for those incidents not previously detected).

From the data collected it was not possible to discern any improvement in detection time, although the CB user's detection rate increased from 39 to an estimated 64 percent. The false-alarm rate decreased from 34 to 30 percent. More significant, however, is the fact that clearance was reduced from 12 to 9 min.

That clearance time decreased at first seems inconsistent with the option's purpose. Upon reflection, however, it is consistent, because with more advance information about complex incidents the trooper is able to both call for a response before arriving and have a more organized cleanup plan after arriving at the incident. Another contributing factor is that, during this demonstration, volume 5 (6) of the FIM series was distributed to the HFB technical advisory committee for review. Since the highway patrol had representatives on the committee, it is a possibility that some of the on-site clearance concepts of volume 5 were adapted to improve clearance times.

The CB signs produced a savings of 48 982 vehicle-h. This compares with a pre-study estimate of 16 600 vehicle-h. The variance is explained by the fact that the option produced a 3-min savings, whereas a conservative 1-min pre-implementation estimate was used.

The previous three-option discussions served the purpose of indicating how the results of the individual options would compare with the pre-implementation estimates, had only one been implemented. As a result, some of the reduction is double counted because the options tended to have overlapping effects. For example, the fuel signs reduced the total incident rate, which gave the highway patrol more time to patrol and resulted in more detected incidents and a smaller average response rate. As indicated previously, however, the net effect of these offsetting facts is that the combined estimate was within 13 percent of the post-study value.

Implications

This section contains a discussion of significant findings that may be of benefit to departments of transportation, police agencies, citizen groups, or FIM groups considering low-cost techniques. The previous sections have focused on quantifying the results of the demonstration. The conclusions and recommendations go beyond this and add a subjective dimension to the project.

Any organization considering improving its FIM environment must take into account the difficulties that will be encountered in evaluating the existing system and the potential worth of solutions to the problem. Although it cannot be quantified, it was found that Florida DOT engineers grasped the concept readily and that the committee was comfortable with making decisions based on the output of the methodology. These observations are supported by the facts that Florida DOT has subsequently used the methodology to support at least one other project and that the nontechnical members of the committee were able to ask questions about and discuss an option's worth in terms of vehicle hours of delay.

Because Florida DOT was the lead agency in the demonstration, it was their decision that a committee should be used. In reviewing this decision, it is noted that this was one of the key decisions that had a major influence on the success of the demonstration. The committee members developed the underpass option and revised both signing options. At the time the latter decision was made, it appeared that informing local commuting residents that there was a long bridge ahead was illogical. However, as was seen earlier, the fuel signs had a definite impact.

Another aspect of the committee selection that merits noting is the fact that wrecker and tow-truck operators were not members. In administering the underpass option this was a sound decision because it limited the number of electronic gate-opening devices to those for the highway patrol. In addition, it would have been impossible to invite all tow operators, because the committee would have become unmanageable. But selecting only certain operators would have posed a political problem.

To avoid conducting a demonstration without the advice and ideas of this important group of FIM actors, a number of them were selected at random and interviewed. The results of the interviews were then incorporated into the decision-making process. This type of a solution to a problem that may have arisen and affected the outcome of the demonstration merits recognition by others considering the use of local committees to assist with finding solutions to FIM problems.

The data were collected for evaluating the demonstration by means of incident logs. Several problems in making use of volunteer data gatherers were encountered that merit note. First, it is difficult to enforce quality control and, second, it becomes a great effort to have others volunteer to collect data for long periods of time.

An aspect of the decision making regarding choice of options is also interesting. In fact an outsider or nonlocal decision maker would not have implemented the options that were finally put into the field. This finding strongly supports Florida DOT's decision regarding a local committee and the FIM team concept (6).

Another important finding is that this methodology is a practical means of estimating the impact of improvements with a degree of accuracy that decision makers can be comfortable with and appreciate.

Unexpected results were also obtained from the demonstration. That is, it was not anticipated that the sign options would affect response time. This result was explained previously as a secondary effect of the signs. The FIM methodology (7) only considers primary effects of the options caused by the conservative approach taken by the manual development. This should not preclude others from estimating secondary effects, which the methodology can evaluate. As more experience is gained with the actual application of it, this type of refinement will probably occur.

RECOMMENDATIONS

The evaluation of the low-cost FIM methodology in Tampa was a success. The Pennsylvania Department of Transportation and FHWA are now in the process of evaluating this technology on the Schuylkill Expressway in Philadelphia. We believe that the use of low-cost FIM procedures has applicability in many locations throughout the United States and that the use of the easily applied delay computation worksheet methodology will permit the selection of the appropriate low-cost options.

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Discussion

W. R. McCasland, Texas Transportation Institute, Texas A&M University, Houston

Urbanek, Tignor, and Price have proposed and implemented a methodology for evaluating options in planning, design, and operations of traffic-responsive incident-management systems. Their emphasis on low-cost FIM approaches is well taken: Not only do they have application to small freeway networks, but they are also compatible with the more extensive surveillance and incident-response systems.

The development and implementation of large, complex systems of surveillance and incident-response systems also require large amounts of time and money. Very often funds are not adequate and the project is delayed to subsequent budget periods when funding resources can be increased. The time lost in implementing some type of FIM system is measurable in lost benefits to the motorists. This project illustrates how limited funds can be effectively applied through the selection of several low-cost options that require short times of implementation.

This paper is limited to a discussion of the results of implementing the methodology for freeway incident management as developed in previous studies. To follow the process for making the calculations and assumptions presented in the tables, the reader is directed to several referenced reports. The basis on which the technical advisory committee ranked the eight options by using the results of FIM is not discussed in the paper.

However, much more important than the numbers and procedures for setting priorities for the FIM options are the facts that, one, there is a process that describes the problem and presents alternate solutions in comparable terms on which to make a decision, and, two, there is an official group, the committee, designated to study the options and make decisions. It has been our experience that a well-informed group representing the appropriate operating agencies can ensure the successful implementation of freeway management projects such as those presented as FIM options.

Some urban areas have established permanent groups that not only act in an advisory capacity but actively engage in conducting studies, collecting information, and supervising the implementation of projects.

The urban area of San Antonio has such a group, the freeway corridor management team (FCMT). FCMT is composed of representatives from the district office of the Texas State Department of Highways and Public Transportation; the city of San Antonio's Departments of Police, Public Works, and Traffic and Transportation; County of Bexar; and the City Transit Company. The team has regular monthly meetings but meets in extra sessions when emergencies arise. FCMT has developed a continuing long-range program of freeway-corridor study but devotes half the meeting time to the review and analysis of hot spots—problem areas that require immediate attention. FCMT reviews the problems, offers solutions for discussion, and develops a consensus of the team for implementation by the appropriate agency. The low-cost FIM process described in this paper would be of great assistance to FCMT in this activity.

The results of the study, the comparison of predicted to actual delay reductions, were very good. However, the authors point out the difficulty in obtaining these data for developing and evaluating the options of FIM. Although most urban areas do not have an extensive freeway surveillance system, a large quantity of data is collected yearly by several agencies for various reasons. Much of the data, designed to be used for long-range planning, can be used for short-term operational objectives. One of the functions of the freeway management team should be to monitor the data-collection procedures so that historical data can have a meaningful input in day-to-day decisions. New data required to make FIM techniques more effective could be identified. Moreover, the use of volunteers to collect data can be improved if members of the committee are involved.

Many low-cost freeway management techniques considered in the study have been successfully demonstrated but have not been extensively implemented. Others, such as dedicated freeway patrols and police patrols, are often the first activities to be curtailed when operation budgets run low. It is to be hoped that the successful evaluation of the options implemented in Tampa by the methodology developed here will encourage other urban areas to test these techniques and to rate them on a comparable basis with other activities.

Richard J. Murphy, California Department of Transportation, Los Angeles

Urbanek, Tignor, and Price have shown that the methodology developed in the research project on analysis and remedies of freeway traffic disturbances can be applied to reduce undesirable impacts of a real-life traffic problem.

Although the demonstration site selected might be considered a special situation (the bridge over Tampa Bay), the methodologies used would be valid for any capacity-reducing incident problem. Because data presented in the report indicate peak flow rates at 2000-2500 vehicles/h, the four-lane bridge obviously only becomes a problem when an incident blocks one or more lanes. The lack of shoulders for emergency parking complicates the problem but is not that atypical.

The authors note that freeway incidents occur randomly with respect to time and place but are predictable in number over time and in impact on normal traffic flow. Data collected on the Los Angeles freeway system in January 1973 illustrate the potential for FIM as an areawide application versus the specific roadway-section application used by the authors:

Item	Vehicle Hours Per Year (000s)	Percent
Recurrent delay		
Weekday peak periods	9300	43-57
Nonrecurrent delay		
Capacity-reducing incidents	5000-10 000	30-47
Weekends	1400	6-8
Holidays	600	3-4
Other	100	1
Total	16 400-21 400	

Thus, delay of up to 10 000 000 vehicle-h/year [shown above as a range because these figures are expanded from data on a 68-km (42-mile) electronic surveillance project] occurs as a result of 2000 or more capacity-reducing incidents per month on the 1086-km (675-mile) freeway system. This amounts to a delay of about 9200 vehicle-h/km (15 000 vehicle-h/mile) annually.

However, during the data-gathering phase (five years), certain travel patterns, or locations where incidents tended to happen more frequently, were observed. In addition, magnitude of congestion, or number of secondary accidents due to incidents at certain geometric locations, tended to be significantly higher than the per-kilometer norm.

This leads one to conclude that, if FIM options or solutions were examined on a total system basis only, the chosen solutions could differ from those selected for a specific problem area and might not solve the specific problems that accumulate to form the "system" problem.

For these reasons, I believe the approach taken in the demonstration project illustrates several important steps necessary in the application of material presented in the several volumes of Alternate Surveillance Concepts and Methods for Freeway Incident Management (4, 6, 7) used in this project.

1. Identification of the problem as truly a freeway-incident problem is covered very well in the report, in that capacity was adequate (other than during incidents) and numerous safety projects had been undertaken to reduce the accident problem to a low level.
2. The problem to be solved must be identified by technicians in a manner that allows system users (motorists) and operators (highway patrol, maintenance forces, emergency forces, etc.) to understand and to participate in offering workable solutions. Formation of the technical advisory committee was, in my opinion, the most important step taken in the project. This set up an easy and natural channel of communications that alerted all system operators of options and techniques of FIM available to them (via the FIM manuals). Results showing reduced incident-clearance time clearly indicate that additional options were being implemented by the system operators during the project. This has been common in California FIM experience. Knowing what is expected of them by the user and operator results in a synergistic effect by which the cooperative action has a greater total effect than the sum of the individual actions.
3. Data collected must be reasonable but do not need the proof of rigid statistical methods. The report is seriously lacking in hard data. However, FIM involves, by nature, an unpredictable event that would require unreasonable resources in equipment and manpower to gather accurate data. As evidenced in the report, the simplistic data gathered by the people involved (users and operators) was acceptable to make decisions.
4. Solutions chosen must be those agreed on by the technical advisory committee. The report notes that, of the three options chosen, one was developed and two were modified to match committee input. That the manual's options are strictly to get the thinking process going is another critical point well covered in the report.
5. Project, process, and technique implemented must be evaluated and updated as time and conditions change. The evaluations presented in the report tell the story. As time passes, the technical advisory committee, if it is still in existence, should take another look at how this FIM project is doing to determine whether any other actions are needed. I wonder if the alternate route and other options are being considered during construction operations.

Urbanek, Tignor, and Price are to be sincerely congratulated on their paper dealing with a methodology of implementing knowledge contained in the FIM guide manuals. Information presented will be of great value to traffic engineers in need of specific ideas on how to get started in solving problems by using the concept of preplanned FIM.

As W. E. Schaefer stated, "Very likely, the most difficult problem to resolve will be the coordination of the complex set of organizations that share the responsibility for the effective operations of the freeway" (8).

I believe the paper presented brings to our attention a method for moving in that direction.

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High-Occupancy Vehicle Considerations on an Arterial Corridor in Pensacola, Florida

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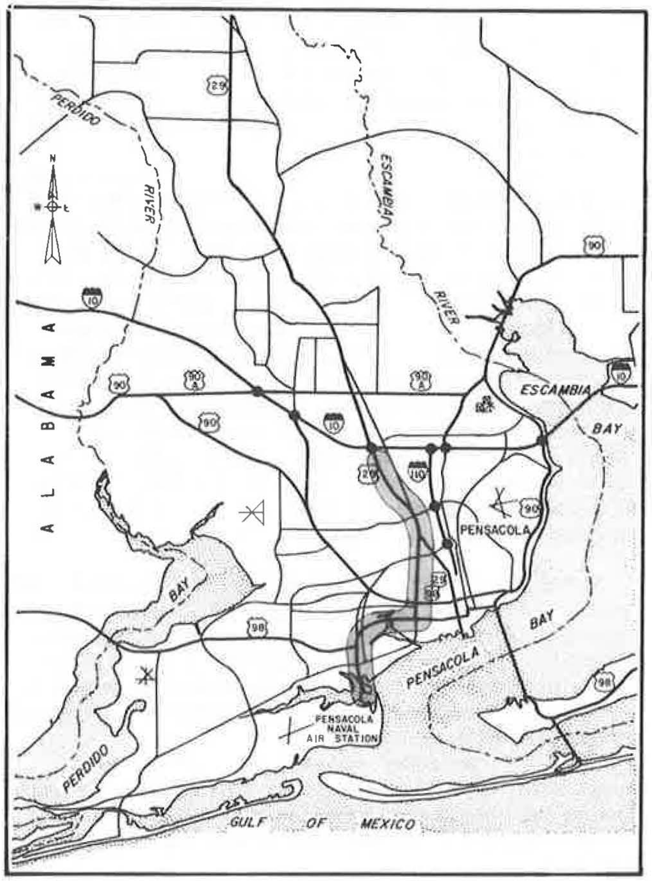
Because of the nature of the traffic using arterial corridors and the complexities of adjacent land uses, most high-occupancy vehicle (HOV) priority techniques impose restrictions on general traffic to such a degree that their implementation has met with limited success. In Pensacola, Florida, an arterial corridor was studied to determine the feasibility of implementing HOV priority techniques. The decisions made as to data

Figure 1. Corridor location in Pensacola HOV design study.



collection, data analyses, alternative selection, and the elimination of parts of the corridor from further consideration will be of general interest to others considering implementing similar projects. The final result of the study was a recommendation to implement HOV priority along parts of the corridor in combination with a lane-control system. This system permits the implementation of an HOV priority system without loss of access to the corridor and has the advantage of maintaining left-turn movements off the corridor.

This paper documents the decisions made in selecting a high-occupancy vehicle (HOV) priority technique on an arterial corridor in the Pensacola, Florida, area. The nature of the traffic using the arterial system and the nature of land uses adjacent to most arterial corridors impose restrictions on the type of techniques that can be considered. Because experience in implementing such techniques is so limited, the process used in Pensacola to select the corridor for HOV improvements should prove to be of general interest to anyone considering the implementation of a similar project.

The Florida Department of Transportation (DOT), in an effort to reduce congestion and improve vehicle-occupancy rates, selected several corridors in major cities around the state for study to determine the feasibility of implementing HOV priority techniques. The objects of these studies were to increase the person-carrying capacity of the corridor, to identify locations where transit service could be improved, to increase carpooling, and to develop a cost-effective HOV priority technique that could be implemented in the near future (three to five years).

The corridor selected in Pensacola was unique in that it was the only HOV corridor in the state located entirely on the arterial system. The decisions made and the lessons learned about data collection and analysis, alternative selection, and design considerations can provide guidance to similar arterial projects in other cities.

CORRIDOR SELECTION

The corridor selected by the DOT for study in Pensacola is US-29 (Pensacola Boulevard) between I-10 and FL-292 (Pace Boulevard), then on FL-292 to the Pensacola Naval Air Station. Although this corridor's main role is as a connector between the Naval Air Station and I-10, its length and the multitude of adjacent land uses have encouraged a variety of trip purposes and trip-making patterns. The location of the corridor in the Pensacola urban area is shown in Figure 1.

Data Collection

The data-collection effort was designed to provide information about the corridor's physical, traffic, and user characteristics. The studies undertaken are described in what follows.

Roadway Characteristics

A complete study of roadway characteristics along the corridor was made to determine where these characteristics might restrict or permit HOV priority techniques. This study consisted of recording lane and median widths, number of lanes, existence of barriers such as obstructions in medians or adverse slopes, locations of structures, and other pertinent data. This information was useful in locating not only those areas where HOV priority techniques can be implemented without major construction but also those constraints along a corridor that would prevent cost-effective implementation of a priority technique.

A record of the types and intensities of land uses along the corridor was also developed. These data are particularly important on an arterial corridor because roadside developments can cause friction on the roadway that could prevent the successful implementation of an HOV priority project.

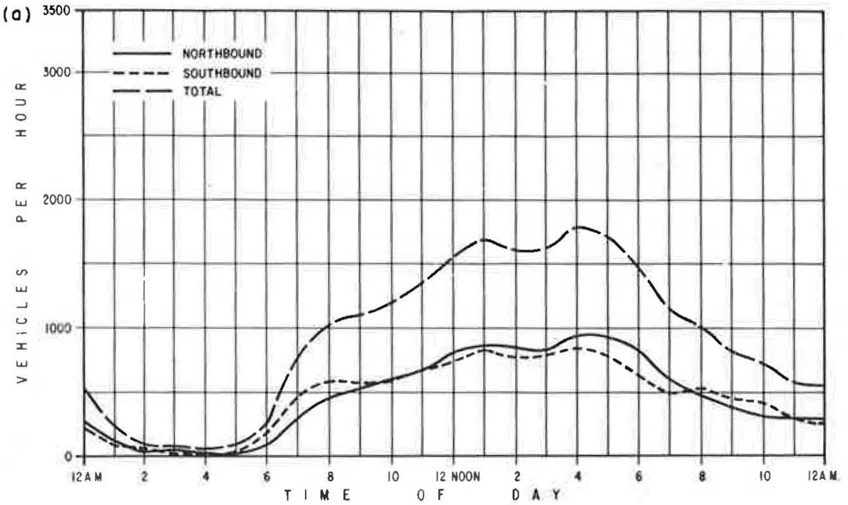
Generally along urban arterial corridors such as the one in Pensacola, the primary areas of traffic conflicts and restraints to improvements occur at intersections. For this reason, greater data-collection efforts were made at major intersections in order to gather information for intersection-capacity analyses. This also gave a more complete picture of the opportunities for and constraints on HOV priority techniques at specific intersections. Lane widths, lengths of turning lanes, median widths, and the location of obstructions such as driveways and utility poles also were determined.

An investigation of traffic-signal operations along the corridor proved useful in identifying those locations where existing equipment was inadequate and where additional equipment would be required to implement HOV priority techniques. Specifically, data collected included signal phasing, type of controllers in use, adequacy of the signal display, and condition of the signal equipment.

Planned improvements along an urban corridor have often had a drastic impact on traffic patterns. Therefore, an effort was made to determine all scheduled or planned improvements along the corridor that might affect the feasibility of HOV priority techniques. The Pensacola urban area transportation study generated several documents that were useful in identifying these projects (1). Projects that are important to identify include intersection improvements, minor widening projects, signal upgrading, and projects improving access to

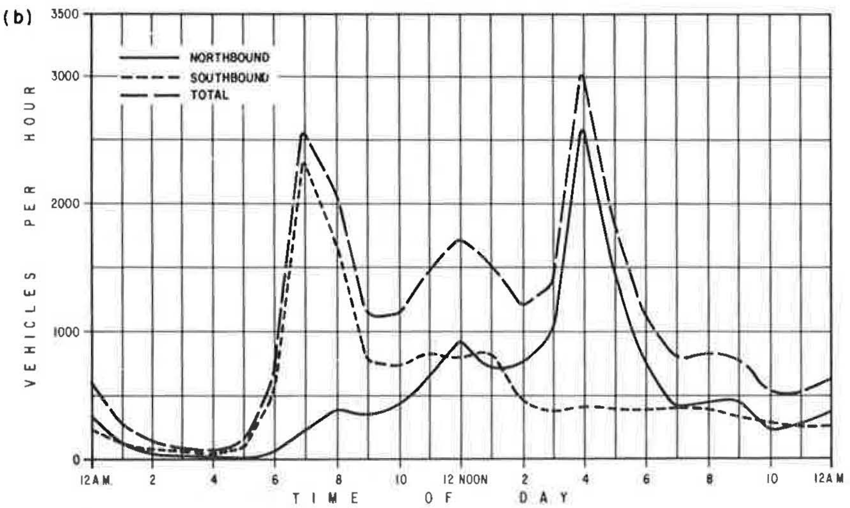
Figure 2. Hourly traffic variations.

(a)



TYPICAL TRAFFIC VARIATIONS NORTH PART OF CORRIDOR
PACE BLVD. NORTH OF CERVANTES ST.

(b)



TYPICAL TRAFFIC VARIATIONS SOUTH PART OF CORRIDOR
NAVY BLVD. NORTH OF NAVY GATE

property adjacent to the corridor. Some of these may be difficult to identify in a work program, but it is important that their impact on a proposed HOV priority technique be considered.

Traffic Characteristics

An extensive traffic-counting program was undertaken to reveal traffic variations on a daily and an hourly basis, which would give us necessary data for capacity analyses. Two counting programs were used. The first involved identifying locations that were representative of the various traffic patterns on the corridors. At these locations traffic counts were taken hourly for one week. The data obtained are useful in determining existing traffic patterns. Typical results are shown in Figure 2.

Turning-movement counts were also taken at 15-min intervals during the peak 2.5- to 3-h periods during the morning and afternoon at eight intersections along the corridor.

Observations of existing vehicle-occupancy trends were made in two locations on the corridor. The purpose of this study was to determine the effectiveness of any recommendations that might be implemented. The vehicle-occupancy studies were also used to verify the existence of current positive attitudes toward carpooling and vanpooling.

Observations were made by noting each vehicle as it passed a checkpoint and recording the number of occupants. Our experience showed that one observer per lane could record each vehicle that passed the checkpoint. The checkpoints chosen were at signalized intersections and, in the case of the Naval Air Station, at the guardhouse. The relatively slow speeds of the vehicles as they passed these points permitted 100 percent coverage.

Travel-time and delay studies were conducted along the entire length of the corridor to gather data on the locations and causes of delays and to provide information about travel speeds along the various portions. The method used to collect these data was a test-vehicle method using the maximum-car technique.

Travel-time runs were made during the morning and afternoon peak hours in good weather. No fewer than 6 runs were made in each direction of travel for each of the two time periods. Studies have shown that from 6 to 12 runs in each direction must be made to achieve an accuracy of the order of 10 percent and to estimate average travel time (2, p. 429). The amount, cause, location, and duration of delays due to traffic controls and operational restraints were obtained.

Data on past accident experience along the corridor were collected to help identify problem locations and to provide a base for measuring the effectiveness of any implemented HOV priority improvement. Data were collected by segments determined after observing traffic patterns along the corridor. The intent was to identify portions of the corridor where varying traffic patterns might indicate different trends in accident experience.

The location of each traffic characteristics study is shown in Figure 3. Those studies identified by arrows in the figure indicate that data were obtained in the direction of the arrow. Data on southbound traffic were obtained in the morning peak hours, and data on northbound traffic were gathered in the afternoon.

Transit Characteristics

Data on existing transit use in the Pensacola corridor were collected and used to locate areas where a potential for improving transit service existed. Data were obtained on ridership figures for all of the routes using any portion of the corridor. The existing route map was also studied to determine if there were routes that paralleled the Pensacola corridor or that provided service that could, in the future, be provided on the corridor. The ridership figures were collected for the same period as the data on the highway users were gathered. This provided a complete picture of all the users of the corridor during this period.

User Characteristics

A telephone survey was conducted to determine the attitudes of the users of the corridor toward bus and car-

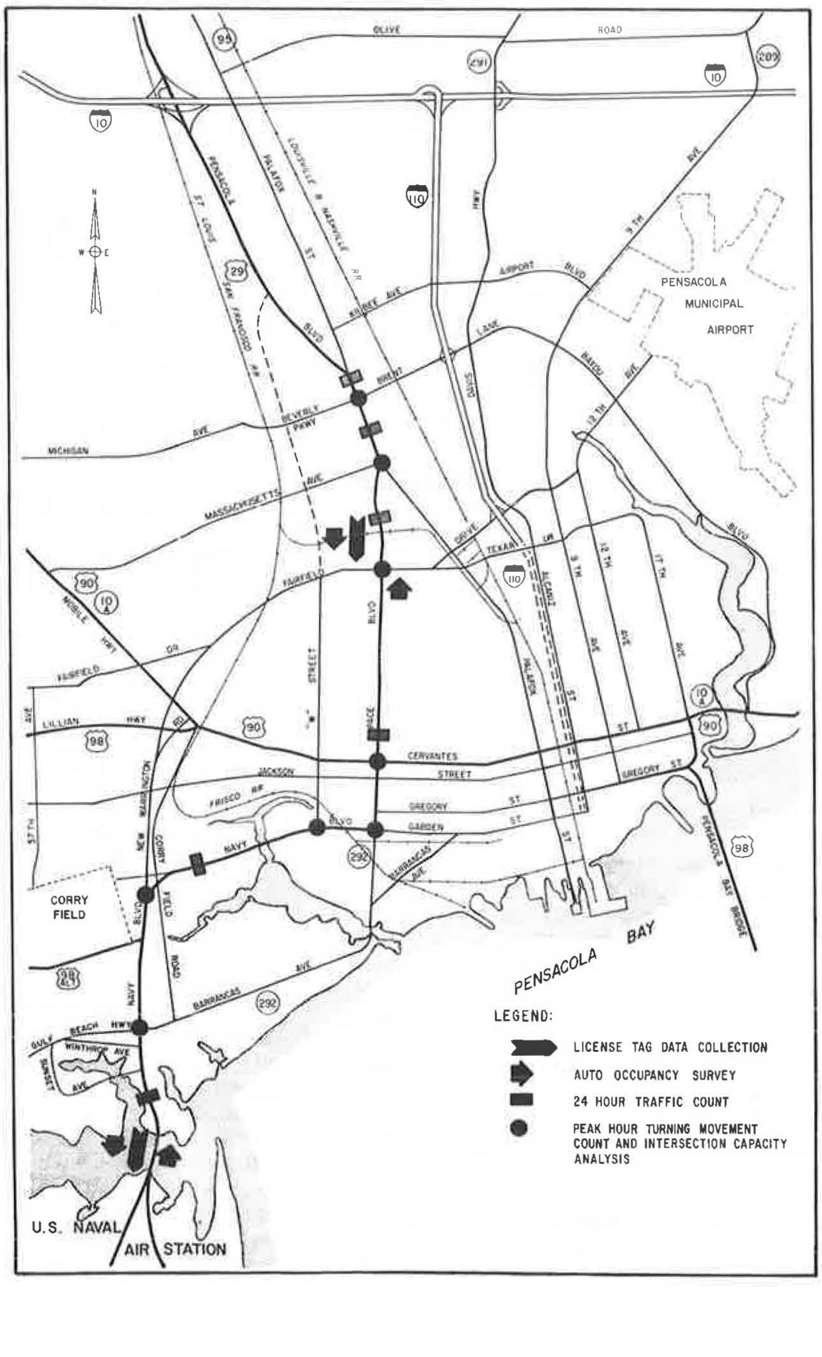


Figure 3. Data-collection locations.

pooling and also to determine existing travel characteristics on the corridor. Only those respondents who indicated that they used the corridor three times or more during the week for their home-to-work trip were considered. Telephone numbers of the interviewees were obtained by matching license-plate numbers to addresses with the cooperation of the Florida Department of Motor Vehicles.

License-plate data were collected at two locations on the corridor during the morning peak hours as shown in Figure 3. These data were collected by observers who read license-plate numbers into a tape recorder as vehicles passed. The numbers were then keypunched and placed in the proper format for further data manipulation. The sites of the data collection were carefully chosen so that vehicle speeds would be sufficiently slow to allow the tag numbers to be read. Observers were instructed to record as many numbers as possible at 15-min intervals during the morning peak hours.

DATA ANALYSIS

The data-collection effort described above represented a thorough compilation of the existing characteristics of the Pensacola study corridor. These data then had to be interpreted in light of the requirements for an HOV priority improvement. This analysis was designed both to determine the feasibility of using HOV priority techniques and to determine the proper HOV priority technique to be used on the corridor.

Roadway Characteristics

The types of HOV priority techniques that are appropriate for arterial roadways are somewhat limited. Generally, on arterials HOV priority techniques involve either reserved lanes or special techniques such as signal preemptions or turn restrictions (3). Restricting turning movements for non-HOV vehicles was deemed inappropriate for the Pensacola area because of the disruption of the normal traffic flow that would occur. For this reason only reserved lanes and signal preemption techniques were considered.

The Pensacola study corridor has several different cross-section types. Some portions of the corridor are four-lane undivided roadway, some are five-lane undivided roadway, and others are four-lane divided cross sections or six-lane divided cross sections.

On the four-lane divided cross sections of the corridor, the reserved-lane techniques that were available would have involved new construction either in the median or on the outside lanes. It was felt that removing one lane of this roadway from general use and reserving it for HOV use would not work in the Pensacola area. The other option of providing a contra-flow lane in the off-peak direction on four-lane divided roadways was only briefly considered. This was because of the lack of a clear peaking trend on most of the corridor segments that had a four-lane divided cross section.

On the four-lane undivided sections of the corridor, the options that were available included adding lanes, eliminating the left-turn lane and providing a reversible HOV lane in the center of the roadway, and remarking pavement or adding lanes to provide a lane-control and HOV system with three lanes in the peak direction, two lanes in the nonpeak direction, and a dual-use left-turn lane. The other options of removing parking to gain an extra lane for HOV use generally were not available along the corridor in Pensacola.

On the six-lane divided portion of the corridor, the options of providing an HOV priority technique included adding a lane in each direction and removing a lane from general use to provide one lane for HOV priority use and two lanes for non-HOV use in each direction. Because of the extreme width of the median on this section, many of the techniques for creating a separated reversible HOV lane in the median of the roadway were inappropriate.

A portion of the Pensacola corridor operates on a five-lane undivided roadway. The options for this portion were similar to those for the four-lane undivided sections of the corridor. The exception was the option of removing parking that would provide six lanes of travel service on this portion of the corridor without additional construction.

The HOV priority techniques that involve signal preemptions were available all along the HOV corridor, but it was felt that these techniques, because of the small demand for buses, would not be very effective in meeting the objectives of the study. While these techniques were considered, they were considered only in conjunction with reserved-lane improvements. Thus, a system wherein a reserved HOV lane would have separate actuaters at signalized intersections along the corridor was considered as an additional means of improving the attractiveness of the HOV lane.

The data study revealed that there were several physical restraints on low-capital HOV priority techniques. For example, there are three bridges along the corridor, two of which are on the east-west portion of Navy Boulevard between Pace Boulevard and New Warrington Road. The third bridge is the structure over Bayou Grande leading directly to the Naval Air Station gate.

Existing traffic on the last of these three bridges is now handled by a lane-use control system put into operation by placing cones along the bridge. This provides three lanes of movement in the peak direction on the bridge and one lane in the off-peak direction. Navy personnel place these cones before the peak hour and remove them after the peak. This technique is very effective in moving vehicles during the peak hour at this location because of the highly directional and repetitive loading patterns near the Naval Air Station during those hours.

Traffic Characteristics

The information on the characteristics of the traffic on the Pensacola corridor was analyzed with a view to locating areas where improvements through use of HOV priority techniques were possible. Figure 2 shows hourly variation of traffic at two of these locations along the corridor. The Pace Boulevard graph (Figure 2a) shows a location that is typical of the traffic variation that occurs throughout most of the corridor. This pattern is characterized by the lack of a true morning peak and by the lack of a discernible directional peak in the afternoon. This traffic pattern developed because the corridor serves a variety of trip purposes and, during the peak hour, serves the home-to-work trips in both directions. In other words, there were as many people commuting into the downtown area of the corridor from the suburbs as were commuting in the opposite direction. This is due in part to large employment centers north of the limits of the corridor.

The traffic pattern shown in Figure 2a does not lend itself very well to the type of HOV priority techniques that can be implemented on the arterial system. Since there is no clear direction of movement during the peak hours on most of the corridor, HOV priority techniques that involve lane control or removing a lane from the nonpeak direction for HOV service would not work. The HOV priority techniques available on this part of the corridor would require the addition of a separate lane in each direction.

The Navy Boulevard graph in Figure 2b is representative of the other traffic pattern that was observed on the Pensacola corridor. This figure identifies a portion of the corridor where very definite morning and afternoon peaks with extremely heavy directional loading occur. This type of traffic pattern was observed on the portion of Navy Boulevard from the Pensacola Naval Air Station's main gate north to the intersection with New Warrington Road. Because of the highly directional peaking characteristics of this traffic pattern, the HOV priority techniques discussed earlier would be appropriate here.

The data on traffic service were also analyzed to determine the location of problem areas. This analysis indicated that the primary source of delay occurred on the section just discussed where delays at the signalized intersections averaged more than 3 min in the morning. Delays on the northern portions of the corridor were noticed only in the afternoons, and these delays were not of the magnitude noticed on the southern portion of the corridor near the Naval Air Station.

The accident data collected also were analyzed to determine whether certain portions of the corridor experienced higher accident rates than others and, if so, whether they could experience a reduction through the application of appropriate HOV priority techniques. The data, however, showed no discernible differences in the accident experiences. It was noticed that the accident rate on that portion of the corridor just north of the Naval Air Station was somewhat lower than on other portions of the corridor. This was a surprise because traffic volumes on this portion were a little higher than on other portions of the corridor. It was felt that the uniformity of the trip purposes and the fact that the same drivers use this portion of the corridor every work day at the same times help hold traffic accident rates down in this portion of the corridor.

Transit Use

As noted before, the existing transit service along the Pensacola corridor was limited. A review of the routes using the corridor indicated that major modifications in route structures would be required to improve transit service, because all of the transit routes in service use the downtown terminal. This is in direct conflict with the desires of most of the users of the corridor, who desire to travel to the Naval Air Station.

User Characteristics

The telephone survey provided valuable information both on the attitudes of the people using the corridor and on their desires for improved traffic service along it. The analysis of the results of the telephone survey proved extremely valuable in the selection of an appropriate HOV priority technique for the Pensacola corridor.

Respondents to the telephone survey were asked questions that revealed two interesting facts. First, the users of the northern portions of the corridor tend to have a variety of destinations, and those whose ultimate destination is the Naval Air Station tend to leave the corridor and use parallel routes to make their approach. Those users of the corridor who are approaching the Naval Air Station generally are only on the corridor in large numbers for a short duration, namely between the intersection of Navy Boulevard with New Warrington Road and the main gate of the Naval Air Station. The other interesting fact noticed in these responses to the telephone survey was that a large number of respondents go several kilometers out of their way to avoid the existing delay at the intersection of Navy Boulevard with Barrancas Avenue and Gulf Beach Highway.

Survey questions concerning existing carpool habits showed a very close relation to the observed vehicle-occupancy rates, particularly those at the Naval Air Station's main gate. This survey also showed that, of those people who do not now carpool, a majority have either considered it in the past or have carpooled in the past. The survey also indicated a positive attitude toward carpooling if these carpools could avoid the congestion on the Pensacola corridor.

Survey respondents also indicated that they would be favorably inclined toward two-block bus service and park-and-ride bus service. In this instance, the two-block service was preferred to carpooling and carpooling slightly preferred to park-and-ride bus service.

Questions inserted in the survey to provide an indication of those people who would or could actually use carpooling and bus service indicated that only about half of those who indicated positive reactions to carpooling and bus service would actually use them. The responses to carpooling and bus-use questions were used to provide a maximum possible bus use and carpool use that would occur on the corridor. The results of the telephone survey indicated that there was a strong potential for an HOV priority lane, at least within that portion of Navy Boulevard between New Warrington Road and the Naval Air Station (4).

SELECTION OF THE HOV PRIORITY CORRIDOR

The results of the data analyses all indicated that portions of the Pensacola corridor were not appropriate for HOV priority techniques. The physical constraints of the two bridges on the east-west portion of Navy Boulevard and of the intersections on the northern portions of the corridor indicated that such techniques would not be particularly low-capital-intensive in these areas. These factors, combined with the traffic service analysis results, indicated that HOV priority techniques would not be successful in the northern portions of the corridor.

The traffic-service indicators that led to this conclusion included the lack of definite peaking characteristics on the northern portions of the corridor and the tendency of the traffic using the northern portions of the corridor to have several destination points including the Naval Air Station, the Pensacola central business district, the various employment centers along the Pensacola corridor itself, and the destinations north of the corridor. The lack of definite traffic-service problems that could be solved by HOV priority techniques discouraged their use.

Therefore Navy Boulevard from New Warrington Road south to the Pensacola Naval Air Station was selected as the only portion of the Pensacola corridor that would be appropriate for the implementation of HOV priority techniques. Along this portion, referred to hereafter as the improvement corridor, the common destination of the traffic, the extreme peaking characteristics, and the positive attitudes toward carpooling and bus use all indicated that HOV priority techniques could be implemented successfully.

SELECTION OF THE APPROPRIATE HOV PRIORITY TECHNIQUE

The improvement corridor has four different cross sections along its length. The first, from New Warrington Road to Alternate US-98, is where the corridor operates on a six-lane divided cross section and has a median width of approximately 12 m (40 ft). South of this section the corridor becomes a four-lane divided roadway with four 3.35-m (11-ft) lanes. The median width in this section is 6.40 m (21 ft). From the intersection with Barrancas Avenue and Gulf Beach Highway south to the bridge over the Bayou Grande, the roadway is undivided and marked for five lanes of service with parking on the southbound side of the road. This section provides three lanes northbound and two lanes southbound. The last of the four cross sections is the bridge over the Bayou Grande, a four-lane bridge that is 12.8 m (42 ft) wide.

There were several alternate methods available to select from to provide a lane for HOV priority uses on the improvement corridor. Generally, these alternatives broke down to the following:

1. Remove a lane from general use, both northbound and southbound;
2. Provide HOV priority only northbound from the Pensacola Naval Air Station's main gate to the intersection of Barrancas and Gulf Beach Highway (this would also involve removing a lane from general use);
3. Use lane control to provide three traffic lanes in the peak direction, one of which is reserved for HOV traffic; and
4. Provide new construction along the entire length of the improvement corridor to provide three lanes in each direction, one of which would be reserved for HOV priority use.

These various options led to the selection of six alternatives to be considered in the selection of an HOV priority technique. The first of these concepts was the "do-nothing" alternative, which meant that there would be no new construction and that only projects already scheduled for improvements on the corridor would be implemented. No priority techniques for HOV vehicles would be used.

The next choice also involved a "no-construction" solution, but for this alternative one lane of general use would be taken away and reserved for HOV vehicles. This alternative is similar to the do-nothing alternative but with the provision for HOV priority. This is alternative 1.

Alternative 2 would involve the installation of lane control along the corridor with no provision for HOV. This alternative would require widening the roadway in the four-lane divided section to six lanes of traffic along the entire improvement corridor from New Warrington Road to the Naval Air Station. Lane control would be needed to provide three lanes in the peak direction plus dual-use left-turn lanes and two lanes in the off-peak direction. All lanes would be available for use to general traffic.

Alternative 3 would be identical in concept to alternative 2, except that one of the lanes in the peak direction would be reserved for HOV use. These two alternatives would also require improvements of the signal systems at signalized intersections.

Alternative 4 would provide for six lanes of traffic plus left-turn lanes as needed. This would be accomplished by widening the existing roadway to provide three travel lanes in each direction plus left-turn lanes. The four-lane divided portion of the corridor would be widened by an additional lane in each direction. South of the intersection of Barrancas Avenue and Gulf Beach Highway, the roadway would be widened to 25.6 m (84 ft).

Alternative 5 would use the same concept as alternative 4, but one of the lanes would be reserved for exclusive use by HOV. Both alternative 4 and alternative 5 would require right-of-way purchase and major construction of drainage facilities and curb and gutter along at least portions of the corridor. In addition, alternatives 4 and 5 would require the construction of a new bridge across Bayou Grande.

ALTERNATIVES EVALUATION

An evaluation matrix was devised to provide a means of evaluating each of the alternatives. This matrix showed how the alternatives compared in nine areas of concern. Qualitative assessments of each of the major evaluation points were made for each of the alternatives. The results of these evaluations are shown in Table 1. The qualitative measurements of the various evaluation points were based to a considerable degree on a quantitative evaluation of the traffic service provided by each of the alternatives. To a certain degree, each evaluation consideration depends on traffic service. The traffic-service evaluation was made by using the intersection-capacity analysis concept and projected traffic demands for each of the alternatives at the intersection of Navy Boulevard with Barrancas Avenue and Gulf Beach Highway.

For the alternatives that did not involve an HOV priority technique, capacity analyses were conducted based on the lane arrangement provided for each of the alternatives. For those alternatives that did involve an HOV improvement, traffic analyses at the intersection were conducted for various probable lane uses that would occur based on the results of the telephone interviews. In general, these optional concepts involved a restriction on the HOV priority lane to vehicles with three or more people or two or more people.

This method provided a quantitative assessment of the traffic service along the improvement corridor for the total range of improvements. This measurement of traffic flow in turn provided a base on which to make qualitative judgments of other impacts of the various alternatives.

SELECTION

The alternative recommended for implementation on the Pensacola improvement corridor was alternative 3, a lane-control and HOV priority concept where three lanes travel in the peak direction and one of them is for HOV priority use. This alternative provided adequate traffic service at a much lower cost than alternatives 4 or 5. The alternative also satisfied the objectives of this study by improving automobile-occupancy rates and reducing the number of vehicle trips on the corridor.

The alternatives that offered HOV priority use would provide the opportunity for implementing limited bus service to the Naval Air Station. Based on responses from the telephone survey, two areas of potential bus use were identified. By servicing these areas with the appropriate bus service, the person-carrying capability of the corridor will be further improved. This bus service is made more attractive because of the time savings on the HOV priority lane.

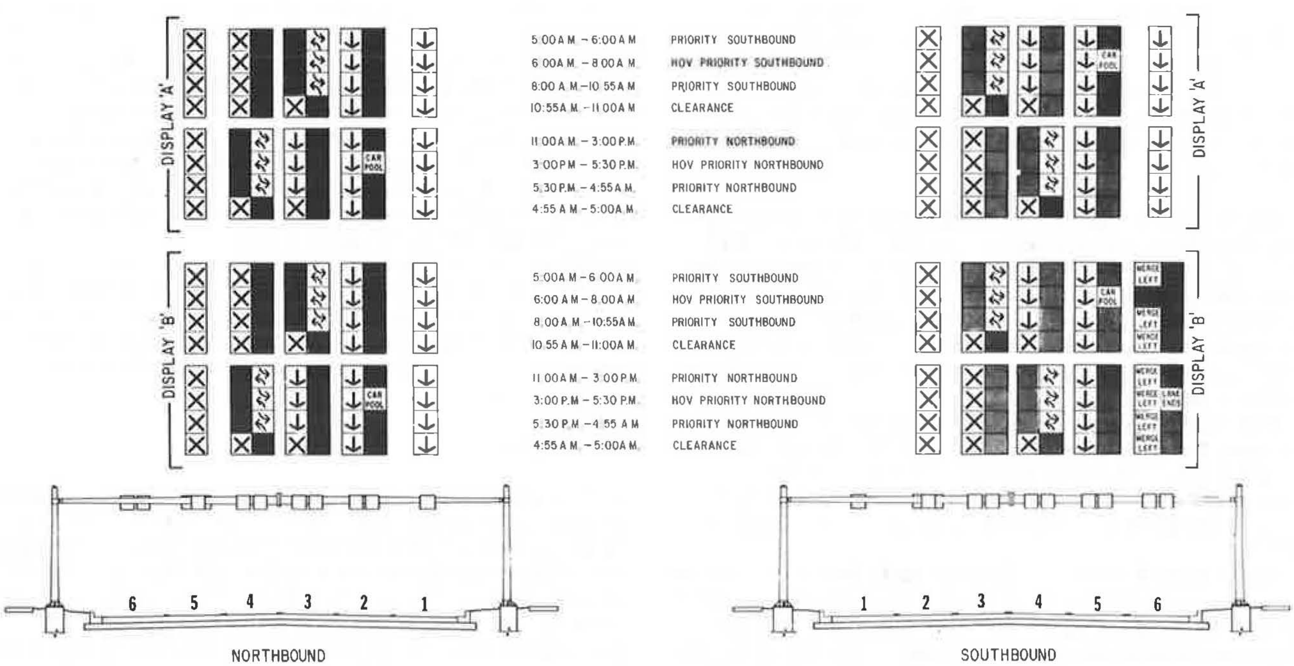
DESIGN CONSIDERATIONS

The alternative selected in the Pensacola corridor incorporates features not found on other arterial HOV priority projects in the country. The most important of these features is the use of a lane-control system with the HOV priority lane. A unique feature of this recommendation is the fact that the left-turn lanes are left in operation along the corridor, which improves access to it. The left-turn lanes also provide a buffer between traffic moving in opposite directions. Because of the nature of the land uses along the improvement corridor, it was desirable to maintain this left-turn capability.

Table 1. Results of evaluation of alternatives for improvements in corridor.

	Alternative					
Area of Concern	Do-Nothing	1	2	3	4	5
Traffic service	Poor	Fair to poor	Good	Good to poor	Excellent	Good to poor
Cost	None	Low	Mid	Mid	High	High
Environmental impact	Poor air, diversions	Poor air, diversions	Improved air, less diversion, noise	Improved air, less diversion, noise	Improved air, neighborhood encroachment, noise	Improved air, neighborhood encroachment, noise
Energy impact	Poor	Poor	Fair	Good	Fair	Good
Compatibility with planning effort	No	No	Yes	Yes	Yes	Yes
Safety	Poor	Poor	Fair	Fair	Excellent	Excellent
Ease of implementation	NA	Good	Good	Good	Poor	Poor
Enforcement	NA	Good	NA	Good	NA	Good
Compatibility with survey	Good	Questionable	Good	Questionable	Good	Questionable

Figure 4. Recommended lane-control signal indications and HOV priority system.



The recommended system operates on a six-lane undivided roadway. To make this system work, three lanes for movement in the peak direction, one lane for left turns in both directions, and two lanes for movement in the nonpeak direction are provided. During the peak traffic hours in the morning and in the afternoon, one lane in the peak direction is reserved exclusively for HOV, or, in this case, vehicles with two or more people.

Details of the recommended concept are shown in Figure 4. This figure also provides a schematic of the type of signal installations required for the proper signalization of the recommended system. Along most of the corridor, display A is used. In the southernmost part of the corridor just north of the Naval Air Station, display B is required to provide a smooth transition from the six-lane undivided roadway to the four-lane bridge over Bayou Grande. It is recommended that the existing system of cone placement on this bridge be continued. This system is recommended to be in place during the hours the HOV priority lane is in use.

CONCLUSIONS

The HOV priority system recommended for Pensacola is unique in its application of lane-control techniques with HOV priority use. This system provides a method for greatly improving traffic flow along the corridor and is cost effective because it uses the existing roadway to the maximum extent. The system also fulfills the objectives of increasing automobile occupancy along the corridor and of moving greater numbers of people with improved traffic service.

Although some of the characteristics of the Pensacola corridor are unique, particularly the extreme homogeneity of the traffic using the corridor during the peak hours, the system recommended has potential application in other urban areas as well. The combination of a lane-use control system with HOV priority use provides a system of implementing HOV priority techniques in a cost-effective manner on arterials. Access along the corridor is not adversely affected because left turns are not prohibited. This type of system has potential in other areas where the following characteristics are observed:

1. Homogeneity of traffic in terms of trip purpose and destination,
2. Distinctive peak periods that are highly directional,
3. Positive attitudes toward carpooling or bus use,
4. Extreme delays for existing travel, and
5. Available roadway widths or right-of-way for additional lanes.

While the Pensacola corridor is unique in that the corridor was a direct feeder to the Pensacola Naval Air Station, other corridors in other urban areas have the five characteristics noted above, and a system such as the one designed for Pensacola could be successfully implemented in these areas as well.

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Planning Rail Station Parking: Approach and Application

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The efforts of the Northeast Corridor Improvement Project to revitalize passenger railroad service have entailed planning numerous station improvements such as accommodating increased passenger parking requirements. Results of studies of 3 of the 11 stations along the corridor that are being upgraded to high-speed rail requirements are reported in this paper. A compendium of parking characteristics to enable planning officials to better assess the needs of rail passenger parkers is included. Topics covered are parking demand estimates, passenger trip characteristics, and fiscal considerations of providing parking at rail stations. Planning guidelines of 0.28 spaces/daily boarding Amtrak passenger and 0.32/commuter passenger are suggested. The need for subsidization to make planned parking facilities economically feasible is also emphasized.

The railroad network in the Northeast Corridor is being upgraded to offer reliable high-speed rail passenger service as an alternative to congested East Coast highways and airports. The corridor, as shown in Figure 1, extends from Washington, D.C., to Boston and includes 15 high-speed rail stations.

Every railroad station, whether located in the corridor or elsewhere, will have different factors influencing passenger parking requirements. Parking studies conducted under the auspices of the Northeast Corridor Improvement Project (NECIP) offer an opportunity to examine general relations that can help determine total parking requirements of the respective stations.

Rail station activity entails the three elements of parking demand conceptually presented in Figure 2—passenger demand for both long- and short-term spaces and nonpassenger (station employee, station visitor) demand. This paper focuses primarily on the passenger demand for long-term parking space. It addresses approaches used in determining passenger parking demand and application of the findings to define economic feasibility, as illustrated in the flowchart in Figure 3.

ESTIMATING PASSENGER PARKING DEMANDS

Parking studies were conducted at the Wilmington, New Haven, and Providence stations as part of NECIP. All cities have Amtrak (high-speed rail) as well as commuter (non-Amtrak) train service. Commuter service is provided in Wilmington by the Southeastern Pennsylvania Transportation Authority (SEPTA), in New Haven by Consolidated Rail Corporation (Conrail), and in Providence by the Boston and Maine Corporation (B&M).

Rail Passengers

Wilmington, the most centrally located of the corridor stations surveyed, has the most train activity: More than 75 trains depart daily. Only 26 trains leave from Providence, as detailed in Table 1. New Haven, however, has the most passenger activity of the three stations, primarily because of commuter trips to New York. An average of 1650 passengers depart from New Haven daily. Average daily boarding passenger volumes are 1335 and 760 at Wilmington and Providence, respectively.

New Haven is principally a commuter station; two-

Figure 1. Northeast rail corridor.

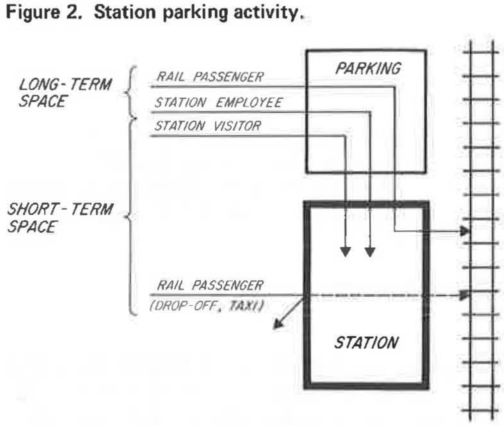


Figure 2. Station parking activity.

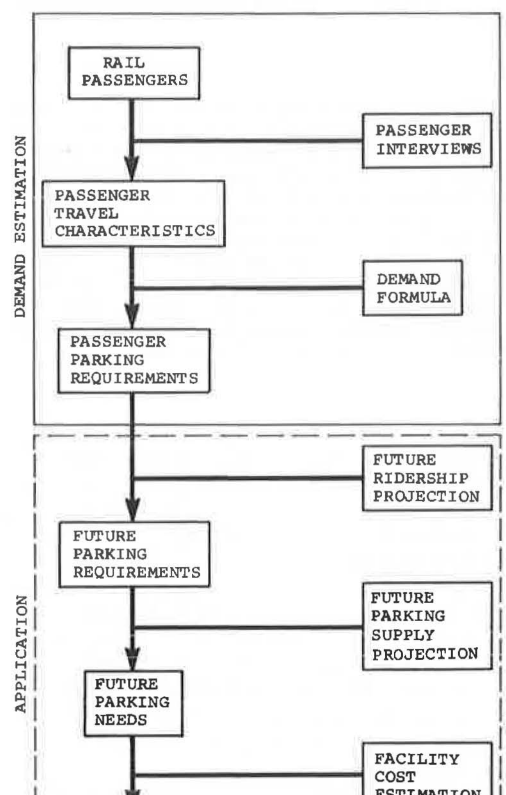


Figure 3. Study approach and application.

thirds of the daily passengers depart on Conrail trains. Conversely, Providence is primarily oriented to Amtrak service; over 70 percent of weekday travel out of Providence is on Amtrak trains. Passenger activity at Wilmington is relatively balanced, approximately 60 percent on Amtrak and 40 percent on SEPTA (commuter).

Before passenger interviews were conducted, it was determined that travel characteristics on Friday differ from those on Monday through Thursday. Passenger volumes are greater and trip durations are longer for weekend traveling. Major generators, such as the University of Delaware near Wilmington, Yale University in New Haven, and Brown University in Providence, as well as the proximity of the stations to major cultural centers such as Boston and New York, greatly influence Friday travel characteristics. Total passenger board-

Table 1. Station activity.

	No. of Daily Departing Trains			Average Daily Boarding Passengers in a Typical Week					
				Amtrak		Commuter		Total	
Station	Amtrak	Commuter	Total	No.	%	No.	%	No.	%
Wilmington	58	18	76	820	61.4	515	38.6	1335	100.0
New Haven	31	23	54	550	33.3	1100	66.7	1650	100.0
Providence	19	7	26	560	73.7	200	26.3	760	100.0

Table 2. Boarding passenger volumes.

	No. of Typical Weekday Boarding Passengers			No. of Typical Friday Boarding Passengers		
Station	Amtrak	Commuter	Total	Amtrak	Commuter	Total
Wilmington	780	510	1290	985	545	1530
New Haven	490	1030	1520	805	1370	2175
Providence	505	200	705	785	190	975

Table 3. Sample sizes.

Passenger Type	Wilmington	New Haven	Providence[a]	Total
Amtrak				
No. of boarding passengers[b]	1766	1425	786	3977
No. of interviews obtained	461	303	527	1291
Percentage of sample	26	21	67	32
Commuter				
No. of boarding passengers[b]	1057	2411	190	3650
No. of interviews obtained	205	637	83	925
Percentage of sample	20	26	44	25
Total				
No. of boarding passengers[b]	2823	3836	976	7635
No. of interviews obtained	666	940	610	2216
Percentage of sample	24	25	63	29

[a] Passenger boardings for Providence represent only Friday activity.
[b] Number of boarding passengers recorded during the survey period; passenger volumes are for two days, a typical Friday and a typical Monday through Thursday weekday.

ings for a typical Friday and a typical Monday through Thursday are presented in Table 2.

Passenger Interviews

Information pertaining to origin-destination patterns, trip purpose, mode of arrival, scheduled time of return, trip frequency, and location of parking, as applicable, was gathered by directly interviewing rail passengers before boarding. Each interview was coded by passenger type (Amtrak versus commuter) and time of departure. To ensure an adequate data base, interviews were conducted over a two-day period from 6:00 a.m. to 9:00 p.m. Friday was always selected as one of the two days, because travel initiated on that day not only incorporates weekday commute-to-work travel but also includes weekend-oriented social and recreational trips.

A predetermined number of interviews per train were conducted, according to passenger volumes. Typically, one out of every three or four boarding passengers was selected for an interview. More than 2200 interviews were conducted in the course of the studies. As outlined in Table 3, the percentage of the sample by station by passenger type was always greater than 20 percent. Approximately 30 percent of all passengers boarding trains during the survey period were interviewed.

The minimum sample size for Amtrak interviews was 300. For attribute sampling, this size is considered to yield reasonably good results. Although the commuter sample size was smaller, the somewhat homogeneous nature of commuters, and the type of survey used, suggests the acceptability of the samples for determining parking demand.

Results of the interviews were expanded to reflect total number of typical weekday and typical Friday boardings. Manual counts of the number of boarding passengers by train were used as control totals for the expansion of the sampled interviews. As a check for the reliability of the survey results and expansion techniques employed, field counts of the number of vehicles accumulated by time period in station-related parking facilities and along the curb were conducted. In all cases, results of the expanded passenger interviews in terms of numbers of parked vehicles and the actual field counts of parked vehicles were similar.

Passenger Travel Characteristics

For information purposes, characteristics of only Friday boarding passengers for each station surveyed are summarized. It should be noted that the data are presented primarily for purposes of comparison, as both Monday through Friday work and business trips and Friday social and recreational trips are represented. In determining parking requirements, characteristics of passengers boarding on all seven days of the week were considered.

Trip Purpose

Trip purposes are classified by work, business, shopping, school, and social and recreational reasons (Table 4). The majority of Friday station activity is directed to travel for reasons other than work, business, school, or shopping. At all stations more than 50 percent of Amtrak departures are for social and recreational trips. With the exception of Providence, few rail passengers use Amtrak service to commute to work. More than 10 percent of Providence Amtrak passengers are workers who frequently use the Amtrak service to

Boston that supplements the B&M commuter schedule. This facilitates the interchange of Amtrak and commuter service when trains are delayed.

Commuter service at both Wilmington and Providence principally accommodates workers. Approximately 50 percent of Wilmington SEPTA passengers and 70 percent of Providence B&M passengers are traveling to work. Although New Haven, as previously stated, is primarily a commuter station, less than one-fourth of Conrail travel is for work purposes. More than 60 percent of New Haven commuter travel is initiated after 10:00 a.m. and is oriented to weekend trips to New York.

Trip Frequency

Average trip frequency of Friday Amtrak passengers is approximately 3 trips/month; passengers on commuter lines travel more frequently; average departures range from 6 to 15/month (Table 5).

Generally, 50-60 percent of Friday Amtrak passengers use rail service less than once a month. Less than 5 percent of Amtrak passengers are daily passengers (5-6 trips/week).

Work-oriented commuter trips at Wilmington and Providence are approximately 57 and 70 percent, respectively, of Friday commuter passengers who are daily rail users. The prevalence of social and recreational commuter trips at New Haven explains the less than 20 percent of Friday rail passengers who are daily passengers.

Trip Duration

Boarding passengers were asked when they would be returning by rail to determine trip duration. Average trip duration of returning Amtrak passengers, as shown in Table 6, ranges from 43 to 49 h; commuter average trip durations are shorter, 15-26 h.

The percentage of Amtrak passengers not returning by rail to the three stations surveyed varies from 15 percent at Providence to 34 percent at Wilmington. The "not returning" category is composed primarily of workers or students traveling by train to the station in the morning and returning by bus or on foot to the station in the evening. These passengers were interviewed on the last leg of a round trip, so they are classified as "not returning." Of returning Amtrak passengers, the majority of trip durations tend to be longer than 24 h. The typical 8-h workday, plus the time for commuting, is reflected in the trip durations of commuters. Approximately one-half of Wilmington SEPTA passengers and three-fourths of Providence B&M passengers have trip durations in the 8- to 12-h range.

Mode of Arrival

Categories for mode of arrival, as detailed in Table 7,

Table 4. Friday passenger trip-purpose percentages.

	Wilmington			New Haven			Providence		
Trip Purpose	Amtrak	Commuter	Total	Amtrak	Commuter	Total	Amtrak	Commuter	Total
Work	5	52	22	5	22	16	11	70	22
Business	34	16	28	17	20	18	12	8	11
Shopping	3	2	3	1	5	4	2	-	2
School	7	4	6	5	5	5	3	3	3
Social and recreational	51	26	41	72	48	57	72	19	62
Total	100	100	100	100	100	100	100	100	100

Table 5. Friday passenger trip-frequency percentages.

No. of Departures	Wilmington			New Haven			Providence		
per Passenger	Amtrak	Commuter	Total	Amtrak	Commuter	Total	Amtrak	Commuter	Total
Less than 1 per month	54	10	39	47	30	36	59	15	50
1-2 per month	26	10	20	32	27	29	23	7	20
3-4 per month	6	4	5	1	3	3	2	1	2
1 per week	8	9	8	10	10	10	6	4	6
2-4 per week	3	10	6	7	10	8	2	3	2
5-6 per week	3	57	22	4	20	14	8	70	20
Average per month	2	13	6	3	6	5	3	15	5

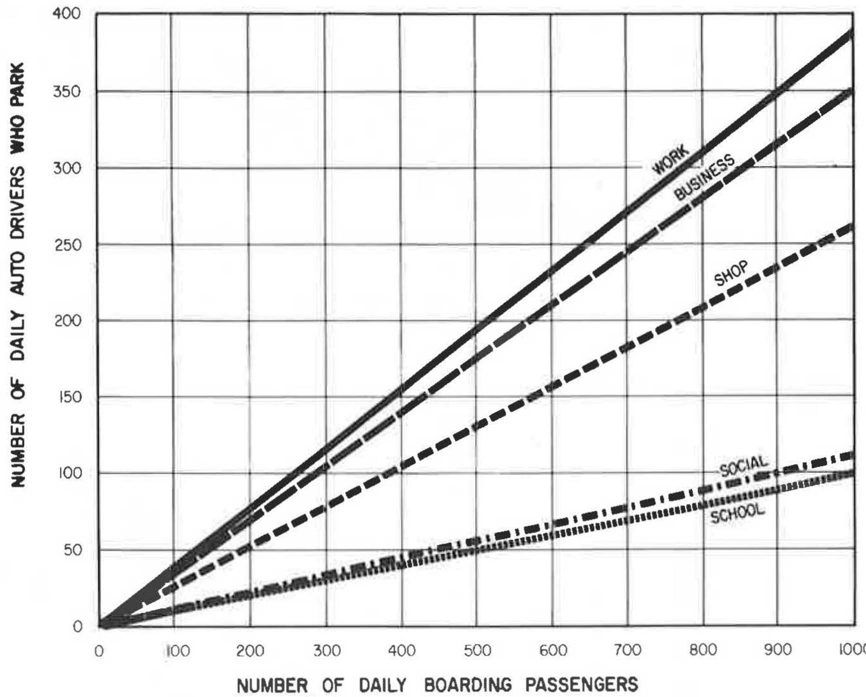
Table 6. Friday passenger trip durations.

Duration	Wilmington			New Haven			Providence		
of Trip	Amtrak	Commuter	Total	Amtrak	Commuter	Total	Amtrak	Commuter	Total
0-4 h	1	3	1	-	-	-	-	-	-
4-8 h	4	10	6	3	8	6	3	2	3
8-12 h	17	47	28	7	24	18	15	75	26
12-16 h	1	9	4	2	11	7	4	1	3
16-24 h	1	-	1	3	4	4	2	2	2
1-2 days	7	1	5	33	18	24	18	1	15
2-3 days	26	7	19	14	8	10	22	2	19
3-4 days	5	9	7	5	1	2	10	2	8
More than 4 days	4	2	3	4	2	3	11	1	9
Not returning	34	12	26	29	24	26	15	14	15
Average, h	45	24	36	43	26	33	49	15	42

Table 7. Friday passenger mode-of-arrival percentages.

Mode of Arrival of Boarding Passengers	Wilmington			New Haven			Providence		
	Amtrak	Commuter	Total	Amtrak	Commuter	Total	Amtrak	Commuter	Total
Automobile driver and park-and-ride	20	29	23	15	22	19	18	42	22
Automobile passenger and park-and-ride	8	11	9	4	6	6	12	12	12
Kiss-and-ride	45	26	38	30	28	29	35	14	31
Bus	13	19	15	16	12	13	11	11	11
Taxi	8	2	6	9	11	10	8	2	7
Walk	5	8	6	19	18	18	15	18	16
Other	1	5	3	8	3	5	1	1	1

Figure 4. Number of automobile parkers by rail trip purpose.



include automobile driver and park-and-ride, automobile passenger and park-and-ride, kiss-and-ride, bus, taxi, walk, and "other." The mode of arrival in the other category is principally by train (e.g., commuter passenger transferring to an Amtrak train).

Automobile drivers account for the mode of arrival of 19-23 percent of all passengers at the three stations surveyed. The principal mode of arrival is kiss-and-ride: 29-38 percent of all rail passengers are dropped off at the station.

New Haven and Providence stations are within a reasonable walking distance of downtown and nearby colleges and universities; approximately 16-18 percent of all passengers arrive at these stations by walking. These passengers are typically college students or workers who commute to New Haven and Providence in the morning and are walking to the station from downtown jobs or school in the evening.

A greater percentage of commuters than Amtrak passengers drive to the station. Automobile drivers account for the mode of arrival of 22-42 percent of commuter passengers as compared to 15-20 percent of Amtrak passengers. Conversely, kiss-and-ride is the mode of arrival of 30-45 percent of Amtrak passengers and 14-28 percent of commuter passengers.

Parker Characteristics

Characteristics of passengers who drive to the station and park were further investigated. As depicted in Figures 4-6, trip purpose, frequency, and duration were related to the number of private-vehicle drivers parking at the station.

There is a general relationship between the purpose of the rail trip and the choice of mode to the station. As indicated in Figure 4, people traveling for purposes of work, business, and shopping tend to drive to the station more often than those traveling for school or other purposes. Therefore, if a rail station accommodates principally the commuting worker, as opposed to the social and recreational trip maker, approximately three times more parking spaces will be required.

The number of automobile drivers and, therefore, the number of parking spaces required are a direct function of trip frequency (Figure 5). As trip frequency increases, the number of automobile drivers and the need for parking space increase.

Figure 6 shows an inverse relation between number of automobile drivers and trip duration. As trip duration increases, the number of automobile drivers decreases. The cost of parking and the risk involved in leaving an automobile unattended influence the relation.

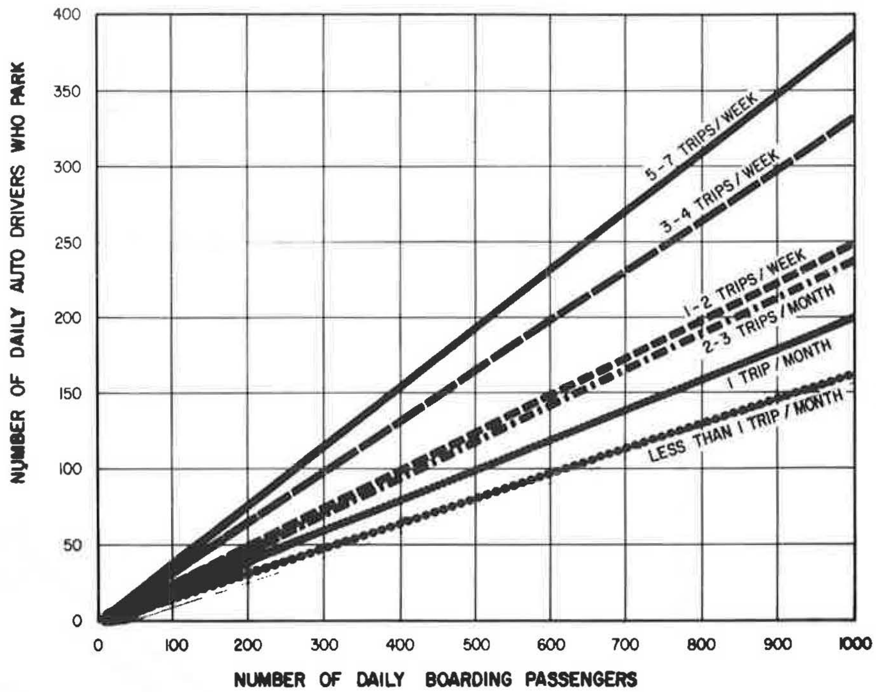


Figure 5. Number of automobile parkers by rail trip frequency.

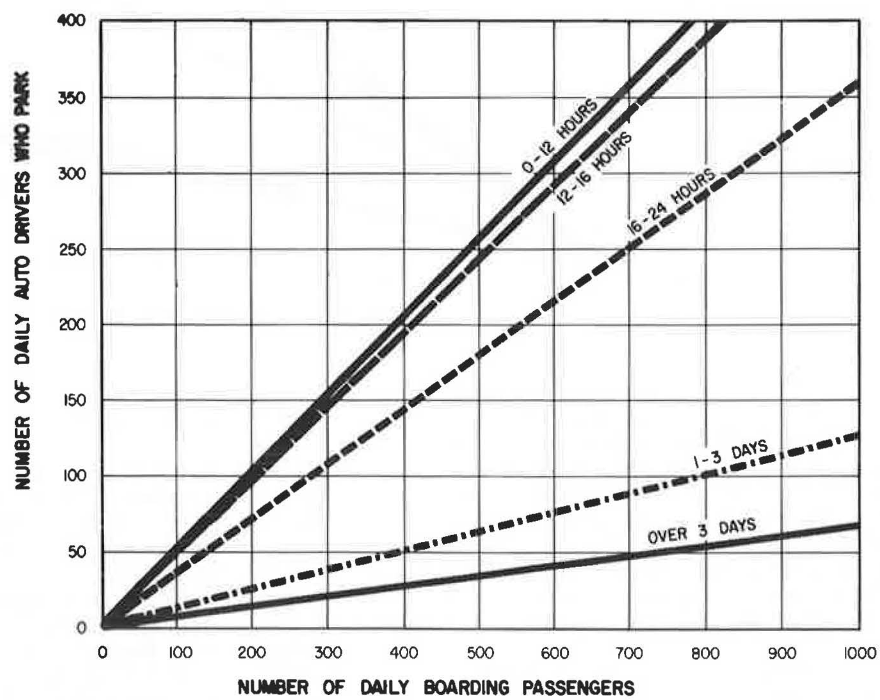


Figure 6. Number of automobile parkers by rail trip duration.

Parking Demand

Characteristics of boarding passengers were analyzed for each survey day to determine the total daily passenger parking demands. Demands were expressed in terms of passenger type (Amtrak versus commuter) for each station. A total daily parking demand of space per boarding passenger was derived based on the proportion of passenger-type volumes to total volumes.

The formula used to derive the demand is

$$N = (A \times B \times C/D)/A \tag{1}$$

where

A = number of total boarding passengers by type (Amtrak or commuter),
B = percentage of automobile drivers who park at the station,
C = maximum accumulation of parked vehicles for given day,
D = number of daily parkers on given day, and
N = peak parking demand.

The overnight parker who consumes one space for two or more days is accounted for in the C/D expression. The number of boarding passengers and the percentage of automobile drivers come from the passenger interview and count information; the maximum accumulation of parked vehicles and number of daily parkers are determined by supplemental field data gathered on the days of passenger interviews.

Parking Requirements

Table 8 presents parking demands ascertained for each station, as well as parking demand planning guidelines. Generally, commuters require more parking spaces than Amtrak passengers, and trip characteristics such as frequency and duration influence the decision on the mode of arrival to the station, which, in turn, translates into parking-space demand.

Commuter parking demands for Providence are considerably greater than for Wilmington and New Haven. This may be due in part to the availability of the relatively inexpensive ($0.75) daily parking in close proximity to the station that influences the passenger mode of arrival. Parking by Amtrak passengers, however, is not greater because of the lack of moderately priced, safe overnight parking. Providence's location at the northern end of the rail corridor may account for the fact that the majority of its Amtrak passengers are bound south on trips of long duration and require overnight parking. General guidelines of 0.32 and 0.28 spaces per daily boarding passenger are suggested for determining commuter and Amtrak passenger parking demand, respectively.

Guidelines can be interpreted in another manner, as is graphically presented in Figure 7. A railroad station offering only commuter service will require more parking spaces than a station serving a large percentage of long-distance Amtrak passengers. The general parking requirements for the majority of the nation's railroad stations, categorized somewhere in between, will be contained within the bank, as shown in Figure 7.

Table 8. Suggested parking demand guidelines.

Station	Daily Parking Space Demand[a] (space/passenger)		
	Amtrak	Commuter	Average
Wilmington	0.33	0.31	0.32
New Haven	0.27	0.32	0.30
Providence	0.20	0.42	0.24
Suggested planning guideline	0.28	0.32	–[b]

[a]Number of daily parking spaces demanded per daily boarding passenger by type.
[b]Total demand is not given, as it reflects a proportion of Amtrak and commuter ridership.

APPLICATION OF DEMAND ESTIMATES

Demand estimates were applied to projections of future rail ridership to develop future parking demands. It was assumed that current patterns of mode of arrival would not be altered in a way that would change the order of magnitude of parking demand in the projection analysis.

Future Rail Passengers

Ridership projections were provided by NECIP. Based on historical trends and speculation on future conditions, the projections were modified to produce a conservative estimate of 1982 rail patronage. Projections were expressed in terms of average daily boarding passengers.

Future Parking Demands and Needs

Results of the three station parking feasibility studies led to a recommendation that two parking garages be built, one in Wilmington and one in New Haven. Because of an abundance of inexpensive parking spaces, only a moderate increase in passenger parking demands, and other factors, a parking facility was not deemed feasible in Providence unless an adjacent office building were developed.

A decision was made to position all new parking spaces in a centrally located facility to maximize passenger convenience. Although the long-term passenger

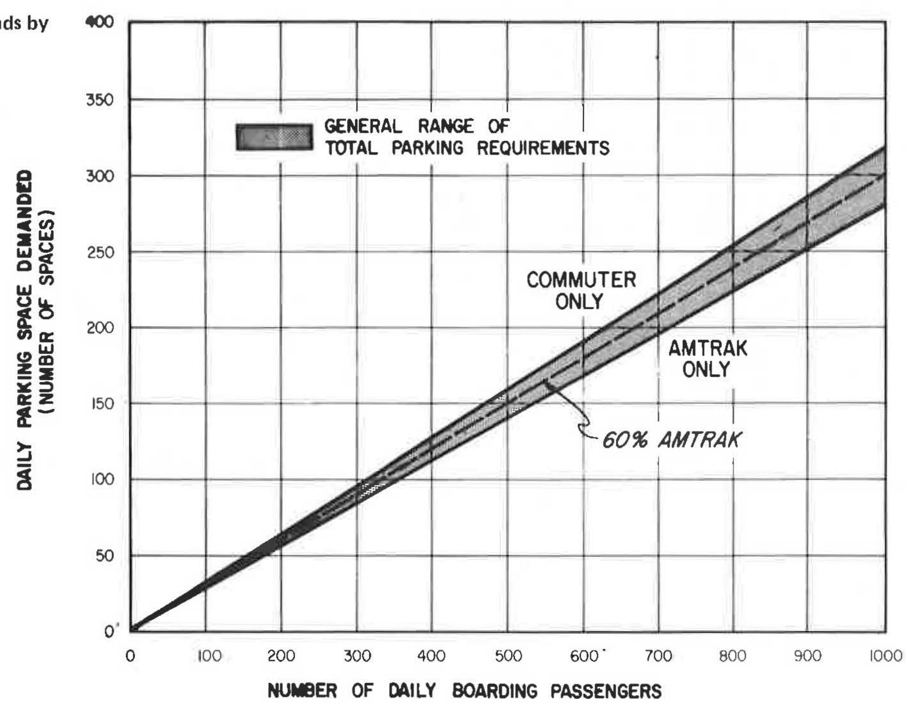


Figure 7. Estimated parking demands by type of station activity.

Table 9. Parking charges.

Prevalent Parking Charges	Station Location: Wilmington	New Haven	Providence[a]
One hour			
Existing, $	0.25	0.25	-
Recommended, $	0.25	0.25	-
Percentage change	-	-	-
Daily			
Existing, $	2.00	1.50	0.75-1.50
Recommended, $	2.00	2.50	-
Percentage change	-	66	-
Monthly			
Existing, $	15.50	15.00	12.00-25.00
Recommended, $	30.00	20.00	-
Percentage change	94	33	-

[a]Parking supply in Providence is a series of surface lots, each with differing rate schedules. No parking facility is planned for Providence; therefore recommendations for changing charges there are not made.

parking demands total 730 spaces in Wilmington and 610 spaces in New Haven, garage sizes recommended were 600 and 960 spaces, respectively. Parking requirements of other than rail passengers, i.e., visitors, employees, and non-station-related activities, were included in the estimated parking needs. Proposals for development of Union Station in New Haven include approximately 4600 m^2 (50 000 ft^2) of commercial space and a bus and limousine terminal. The parking requirements of these facilities were incorporated into the estimate of future needs. In addition, the anticipated 1982 parking supply was determined to be able to accommodate overall parking deficiencies. As stated, the result was a need for 600 spaces in Wilmington and 960 in New Haven.

Economic Feasibility

The economic feasibility of the proposed facilities was influenced by, among other factors, joint use of the facility and the net gain of parking spaces by the locality. A major determinant of economic feasibility was the Federal Railroad Administration's participation in the form of monetary contribution of 50 percent of total development costs of rail-related spaces.

Existing surface parking lots were selected as the sites for the proposed facilities. Hence, the nonfederal portion of the financing, typically from a local agency, was required to meet the 50 percent of costs to reconstruct preempted spaces. The net gain in spaces, therefore, influenced the decisions of the nonfederal participants relative to the economic feasibility of the project.

Further, it was determined earlier that, without federal monetary participation, the parking garages would not be economically feasible projects. Hence, it can be surmised that for most station situations a subsidy in some form is required to finance parking garages. A review of estimated monetary requirements and parking revenues reveals circumstances that support this premise.

Capital Requirements

As detailed below, the average construction cost per space for the two New Haven and Wilmington proposed facilities was approximately $5650.

Type of Costs per Space	Average Garage-Related Estimated Cost ($)
Average basic construction	5650
Average development	7450
Average annual operating and maintenance	225

When financing requirements and other development considerations were taken into account, the average development cost per space became $7450.

Based on financing charges, other economic considerations, and the low turnover of parkers at railroad stations (basically one parker per space per day), more than $2.00/space daily is implied as the return on investment required to operate at cost a parking garage in the order discussed.

Existing and recommended parking rates for the stations studied are summarized in Table 9. With federal participation, a daily rate of $2.00 or more is required to make the proposed parking facility economically feasible.

It is anticipated that, if monetary assistance is not available for the development of a parking garage, the cost of traveling to work by train would become great enough to discourage train use. In terms of a daily commuter, the monthly commutation ticket (about $100) plus a monthly parking charge (approximately $40.00) would result in a total monthly commutation cost of $140.00. As a planning guideline, 20-30 percent of the cost of a monthly commutation ticket is suggested as an acceptable monthly parking charge.

SUMMARY AND CONCLUSIONS

NECIP has provided the transportation planner with sufficient information to estimate the parking demands of the rail passenger. The experience of proposed projects has also identified key financial implications. The low turnover of rail parkers requires substantial parking charges to finance the facility. If the cost of parking is too high, however, an on-street spillover may occur and the garage will become a financial burden.

It can be concluded, therefore, that the provision of parking at rail stations must be considered in a similar manner as other public utilities and that outside financial assistance is required to make the project economically feasible.

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Development of Freeway Incident-Detection Algorithms by Using Pattern-Recognition Techniques

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Two incident-detection experiments were conducted on the Queen Elizabeth Way Freeway Surveillance and Control System in Ontario. A pattern-recognition approach was applied to improve incident-detection algorithms. By considering the true- and false-incident-alarm identification process as pattern-recognition in nature, the maximum-likelihood decision principle was applied to develop an optimum incident-duration persistence test. The false-alarm rate fell from 0.09 to 0.06 percent during a nine-month field test experiment. In the second experiment a two-layer committee-machine structure achieved an 85.7 percent detection rate on 28 samples of historical incident data.

This paper presents the findings of two incident-detection experiments that were based on pattern-recognition concepts and carried out on the Queen Elizabeth Way (QEW) Freeway Surveillance and Control System (FSCS) (1).

This system includes an electronic incident-detection system that employs a modified California algorithm (2). It has achieved an 85 percent detection rate with a 0.09 percent false-alarm rate. To further enhance the effectiveness of this system, two incident-detection improvement experiments were conducted with historical data from QEW. In the first experiment, a pattern-recognition process was used to improve the incident-detection false-alarm rate. In the second experiment, a two-layered committee-machine concept was developed to implement a freeway-lane incident-detection algorithm.

INCIDENT-DETECTION PERSISTENCE-TEST ALGORITHM

The performance of an incident-detection algorithm is usually evaluated in terms of three measures of performance: detection rate, false-alarm rate, and detection time (3). This section examines the feasibility of improving the false-alarm rate by a pattern-recognition approach (4).

Pattern-Recognition Approach

Essentially, the problem is to discriminate between true and false alarms on the basis of their different duration characteristics. One can consider this as a pattern-recognition process whose alarms fall into either of two different pattern categories; true alarms (category 1) or false alarms (category 2).

To illustrate, consider the typical true- and false-alarm duration probability distributions shown in Figure 1. The large overlap of the two distributions indicates that there is poor pattern separability if one relies solely on alarm duration to distinguish between true and false alarms.

If, however, one considers the alarm duration pattern feature only up to a certain value, X′ say, then one can use Bayes' optimal decision rule to determine an X′ that will maximize the likelihood that an alarm with a duration less than X′ is a false alarm. The value of X′ so determined can then be incorporated into an incident-detection algorithm in the form of a persistence test to reduce the false-alarm rate. The penalty for the improvement will be an increase in the detection time of X′ minutes. Bayes' optimum decision rule can be stated as follows:

$$P(I|X) = [P(X|I)P(I)]/P(X) \tag{1}$$

where

- I = the pattern category ($I = 1$ for a true-alarm pattern and $I = 2$ for a false-alarm pattern);
- X = the pattern feature, defined only in $0 \leq X \leq X'$;
- $P(X|I)$ = the probability of occurrence of pattern X given that it belongs to category I;
- $P(I)$ = the a priori probability of occurrence of category I;
- $P(X)$ = the a priori probability of occurrence of pattern X; and
- $P(I|X)$ = the probability of occurrence of category I given that it belongs to pattern X.

The likelihood ratio, which must be maximized with respect to X′, is given by

$$LR = P(2|X)/P(1|X) \tag{2}$$

If this is greater than unity then pattern X can be categorized as belonging to a false-alarm pattern category according to the maximum-likelihood decision principle.

Duration of Persistence Test Interval

To illustrate the application of the above approach for improving incident-detection performance, we shall consider the historical incident-detection data collected on the QEW over a 14-month period from January 1977 to February 1978. The data are shown plotted as histograms in Figures 2 and 3 for true- and false-incident-alarm conditions, respectively. In each figure, the frequency of occurrence of the alarm condition within prescribed alarm duration intervals is indicated. If alarm duration can be considered as a random variable, then the probability of a sample alarm condition occurring within a given alarm duration interval is approximately equal to the number of samples in that interval divided by the total number of samples.

The data in Figures 2 and 3 can be used directly to calculate the likelihood ratio. For example, if we assume a value of $X' = 1$ min, then we have

$$P(X|I = 1) = (1 + 2)/89 = 0.0337$$
$$P(X|I = 2) = (103 + 78)/485 = 0.373$$
$$P(1|X) = 0.0163$$
$$P(2|X) = 0.984$$

which indicates that only 1.6 percent of the alarm patterns occurring within an alarm duration interval of 1 min are true alarms.

Then the likelihood ratio is given by

Figure 1. Typical alarm duration probability distributions.

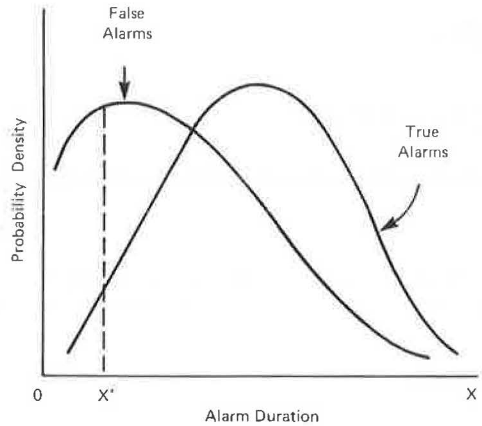


Figure 2. True-alarm duration histogram.

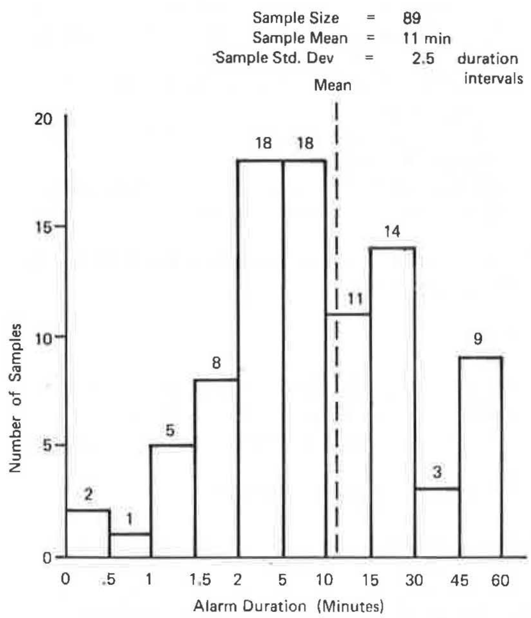
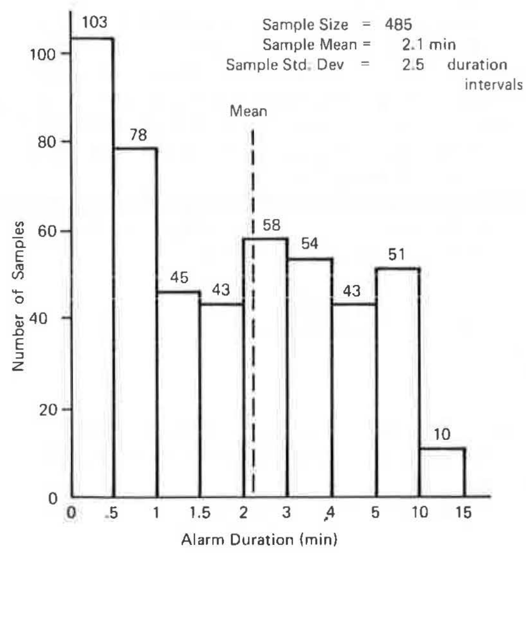


Figure 3. False-alarm duration histogram.



$$LR(X' = 1) = P(2 \mid X)/(1 \mid X) = 60.4 >> 1 \quad (3)$$

The likelihood ratios were calculated for two other values of X′ and are shown plotted in Figure 4. Clearly, X′ = 1 min is the best choice.

Experimental Results

According to the preceding analysis of the QEW historical data, it appears that a computer algorithm with a 1-min incident-duration persistence test can effectively remove 37.3 percent of the false alarms without excessively delaying the incident-detection response time. This can be accomplished by simply delaying the incident alarm output for a 1-min period. At the end of the minute, if the incident alarm still persists, then the pending-incident alarm can be issued by the incident-detection program. Otherwise, the pending-incident alarm will be cancelled.

This incident-duration persistence check algorithm was implemented on the QEW FSCS in March 1978, and the algorithm performance data were collected from March to June 1978. During this period, the false-alarm rate was reduced by 33 percent from the previous value of 0.09 to 0.06 percent. This was achieved at the expense of a reduction in detection rate of from 85 to 74 percent.

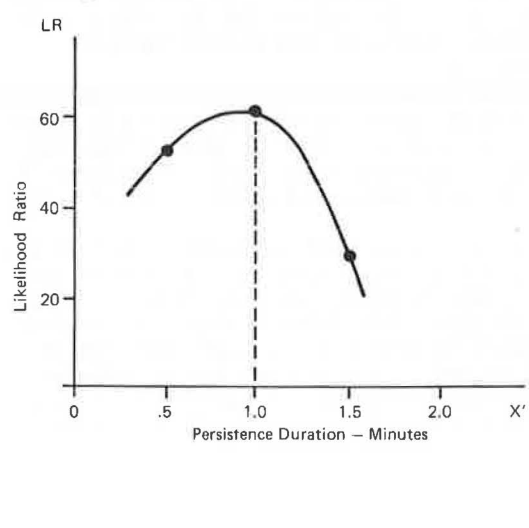
To put the significance of this improvement in better perspective, one might translate this 33 percent reduction in false-alarm rate into the elimination of 160 false alarms if this algorithm had been applied from January 1977 to February 1978. The reduction in detection rate can only mean that those incidents that have an alarm duration less than or equal to 1 min are not being detected.

Normally these short-duration incidents appear to have only minor, transient effects on the traffic flow. Their not being detected presents no operational problem. Also, the accompanying increase in the detection time of 1 min has negligible effect on the incident-management operation. These are confirmed by a lack of complaints from QEW FSCS operators.

LANE INCIDENT DETECTION ON A MULTILANE FREEWAY

The development and experimental verification of the lane incident-detection system described here was based on

Figure 4. Variation of likelihood ratio with persistence duration.



1. Consideration of only a three-lane freeway,
2. Investigation of only single-lane freeway incidents,
3. Detectorization of all three lanes at each incident detector station, and
4. Identification of the lane incident location after identification of the station incident location.

In this section, the committee-machine concept (4) is first applied to the general problem of multilane incident detection. This is followed by the description of a realistic (though simplified) practical application and some experimental results.

Committee-Machine Approach

Freeway-lane incident detection can be considered a pattern-recognition process with three pattern categories, each corresponding to the occurrence of an incident in one of the three freeway lanes (see Figure 5). Figure 5 also shows QEW FSCS detector system configuration. With this type of configuration, a 30-s lane occupancy, lane speed, and lane volume data set can be obtained. The data set containing the patterns to be so classified is the selected lane-surveillance data from the various freeway detector stations. These patterns are processed by the various lane incident-detection algorithms to produce an incident-lane number decision. The lane with the highest number of decisions in its favor is then selected as the most probable incident-lane location based on the majority decision principle.

To illustrate how the above concepts can be formulated into a committee-machine structure, consider first the basic committee logic decision unit (CLDU) shown in Figure 6. This unit is provided with surveillance data from both upstream and downstream detector stations for all three lanes as its input and contains an algorithm that generates a decision about which of the three lanes has experienced the incident. These units are arranged in banks to form the first layer of a committee-machine structure, as illustrated in Figure 7. The second layer of the committee machine is a vote-taking logic unit (VTLU) that accepts the decision outputs from the first-layer CLDUs and selects the lane where the incident occurred according to the majority decision principle. The three such two-layered committee machines shown in Figure 7 correspond to the case where occupancy, volume, and speed surveillance data are all available. The outputs of these three two-layered committee machines are fed to the third-layer VTLU, possibly with different weights, which will then select an incident lane according to the majority decision principle.

The VTLU polls the decision outputs from each of the CLDUs in the first layer (or the weighted counts from the three VTLUs in the second layer), summarizes the total number of decision counts for each type of decision output, and selects a desired decision output according to the consensus function $\max [n_i/N]$, where n_i is a number of decision counts for decision category i for i = 1, 2, 3, and N is the total number of algorithms (and therefore, CLDUs) dedicated to generating decisions for any given VTLU. In other words, the decision type i that has the maximum number of decision counts is designated as the lane where the incident occurred.

Practical Application and Experimental Results

To illustrate the practical application of the committee-machine approach, the two-layer committee-machine structure shown in Figure 8 was employed. All of the CLDUs were identical in function but, in effect, used a different algorithm because each was provided with a different time slice (30-s sample time) of lane-occupancy data. Each CLDU computed the differential occupancy,

Figure 5. Lane incident detection on a three-lane freeway.

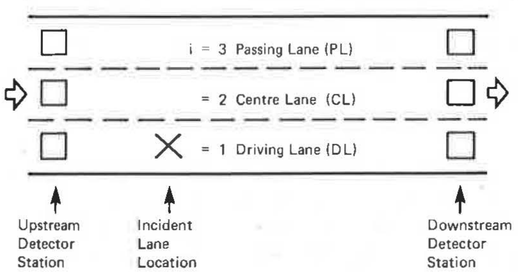


Figure 6. Committee logic decision unit.

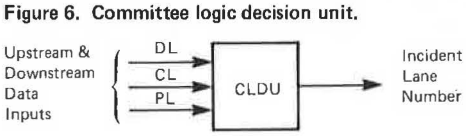


Figure 7. Three-layer committee machine for lane incident detection.

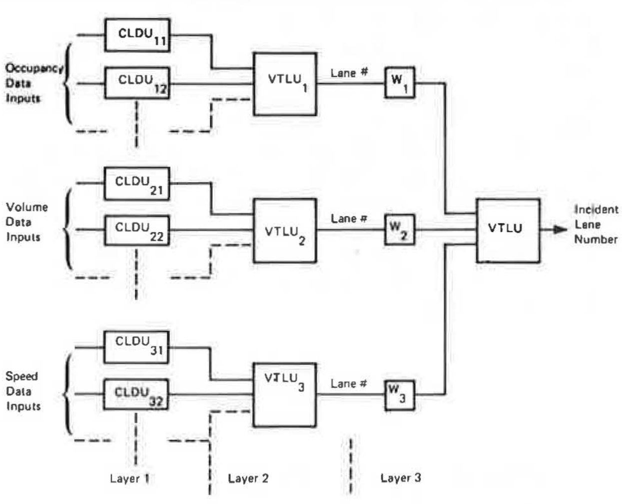


Figure 8. Two-layer committee-machine structure.

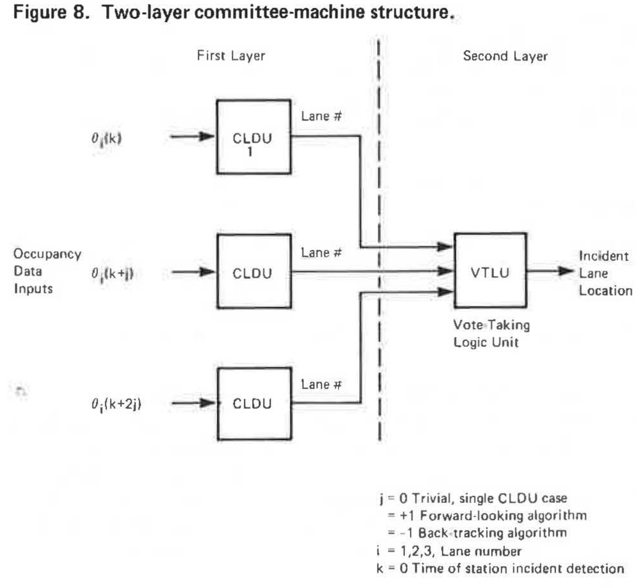


Table 1. Percentages of lane occupancy for center-lane incident.

Time	Upstream-Station Data by Lane			Downstream-Station Data by Lane		
	Driving	Center	Passing	Driving	Center	Passing
7:52:30	30	36	53	61	53	62
7:53:00	62	53	53	55	44	44
7:53:30	51	40	36	37	41	36
7:54:00	38	72	45	42	29	34
7:54:30	25	100	47	16	14	19
7:55:00	35	49	46	8	13	12
7:55:30	42	80	71	17	14	12
7:56:00	36	40	50	10	17	12
7:56:30	27	43	61	6	11	15

$\sigma_i(k)$, from downstream-station lane-occupancy data according to the following equation:

$$\sigma_i(k) = \{[\theta_i(k)]_i - \theta_i(k)\}/[\theta_i(k)]_i \quad (4)$$

where

$[\theta_i(k)]_i$ = downstream-station occupancy at time slice k averaged over all three lanes,
$\theta_i(k)$ = downstream-station lane occupancy at time slice k for i = 1, 2, 3, and
k = 30-s time slice.

The minimum of $\sigma_i(k)$ was then selected and compared to an empirically determined constant k. If min $[\sigma_i(k)] \geq$ K (K = 0.2 for the QEW freeway section being considered), then the CLDU indicated lane i as the incident location. Otherwise, the CLDU sought the maximum upstream-station lane occupancy and indicated lane i as the incident location.

As indicated in Figure 8, three different types of algorithms were tested. The first (j = 0) is a trivial case where only data at the time of station incident detection (k = 0) were used; in this case two of the three CLDUs are redundant. In the second case (j = +1) forward-looking algorithms were used in which data at the time of station incident detection and those from the two succeeding time slices were used. The third case (j = -1) employed back-tracking algorithms in which data at the time of station incident detection and those from the preceding two time slices were used.

The rationale for testing the forward-looking and back-tracking types of algorithms is based on the observed highly stochastic nature of the lane incident data. This is clearly illustrated by Table 1, which shows typical upstream and downstream lane-occupancy data for several time slices both before and after the time of station incident detection.

The experimental results obtained by testing the above-defined algorithms in the two-layer committee-machine configuration shown in Figure 8 are summarized below

Algorithm Type	Detection Rate (%)	Detection Time
Trivial single-CLDU case	67.8	Same as for station incident detection
Forward-looking	67.8	Two time slices (1 min) after station incident detection
Back-tracking	85.7	Same as for station incident detection

They are based on the same 28 samples of lane incident data from the QEW FSCS. The back-tracking algorithms achieved an 85.7 percent lane incident-detection rate, which is clearly superior to the other two algorithms, which achieved a rate of only 67.8 percent. The back-tracking algorithms also have the obvious advantage of shorter lane incident-detection times compared to the other two.

SUMMARY AND CONCLUSIONS

1. A pattern-recognition approach was successfully applied to the development of improved incident-detection algorithms. The results indicate that this approach provides a useful conceptual framework and is a practical tool for examining such problems as well.
2. The true- and false-incident-alarm identification process was considered as a pattern-recognition process and the maximum-likelihood decision principle was applied to develop an optimum incident-duration persistence test. This was tested experimentally and was found to reduce the false-alarm rate from 0.09 to 0.06 percent during a three-month field test.
3. A multilayered committee-machine structure was developed to implement a set of freeway-lane incident-detection algorithms. This concept was tested experimentally by using a two-layer committee-machine structure that achieved a lane-detection rate of 85.7 percent based on 28 samples of historical freeway-lane incident data.

ACKNOWLEDGMENT

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