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Effectiveness of Traffic Operational Measures

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51	Highway Safety
53	Traffic Control and Operations
55	Traffic Measurements

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CONTENTS

FOREWORD

The five papers and three discussions in this RECORD describe ways to improve a wide range of important traffic features such as travel time on freeways during incidents, safety, and traffic engineering decision-making. The material will be useful to traffic operations engineers, designers, and maintenance planners and can provide support for more enlightened decisions by transportation administrators.

In the opening paper, Messer, Dudek, and Friebele describe their development of a model for predicting travel time on a freeway during incident conditions. The model can also be used to predict speeds, volumes, and other operational measures and was developed for possible use in an operational control strategy using variable-message signs to divert motorists to alternate routes when justified. They say that it can also be used to predict queue backups and delays due to lane closures for scheduled maintenance operations. Three discussions by Gordon, Wattleworth, and Reilly extend the usefulness of the work and suggest that the future application of the model can be enhanced.

West and Heimbach present an analysis procedure for determining the significant changes in instrumented car responses on adjacent short highway sections in order to correlate the sections with highway geometry elements, intersections and grades, and number of accidents. Using data from US-70 in North Carolina, the authors show highest correlations between accidents and those significant change sections whose car responses were speed change, running time, and total time. They also found that significant driver responses did not correlate highly with intersections or grades.

Looking at both automobile and mass transportation in an area of West Philadelphia, Vuchic and Weston report on their study of possible short-range improvements in traffic conditions. They show the development of alternative plans that are limited to improvements possible under TOPICS and suggest the proper alternative to select to maximize benefits to all modes.

In a greatly shortened version of a final project report, Minister, Lew, Ovaici, and May describe their computer simulation model for evaluating priority operations on freeways. Several types of reserved-lane configurations for differing types of traffic mixes, e.g., buses and car pools, can be considered. Based on their experience with the model, the authors suggest a number of ways to add further realism and operational ease to it.

Lipinski examined the decision-making process by which traffic engineering decisions are made in urban areas and proposes a methodology for improving these procedures. Existing processes were studied by a review of existing conditions in 17 cities, and several examples of decision-making behavior are presented to illustrate the application of the author's theoretical model.

METHOD FOR PREDICTING TRAVEL TIME AND OTHER OPERATIONAL MEASURES IN REAL-TIME DURING FREEWAY INCIDENT CONDITIONS

Carroll J. Messer and Conrad L. Dudek, Texas Transportation Institute, Texas A&M University; and
John D. Friebele*, City of Austin, Texas

This paper presents the development of a method for predicting the travel time required by a motorist to travel from any selected freeway location to the end of the freeway system during incident conditions. It is predictive in that it computes an estimate of a motorist's travel time if he were to enter the freeway several minutes in the future. Speeds, volumes, and other operational measures can be predicted also. These calculations are made immediately after the incident is detected and the necessary operational measures have been evaluated. Speeds of the various shock waves and travel-time results are also presented. The model was developed following the kinematic wave theory of Lighthill and Whitham for possible use in an operational control strategy of variable-message signs whereby motorists would be diverted to alternate routes if conditions on the freeway relative to selected alternate routes justified the diversion. The model could also be used to predict queue backups and delays due to lane closures caused by scheduled maintenance operations.

•FREEWAY ramp control systems have proved their effectiveness in relieving freeway congestion when operations are free of incidents. Incident conditions, however, are a frequently occurring phenomenon on urban freeways. Goolsby found that, within a 6-mile section on the Gulf Freeway in Houston (1), more than 13 lane-blocking incidents occur on the average during the time period of 6 a.m. to 7 p.m. from Monday through Friday. Stalled vehicles and accidents were the contributing causes of 97 percent of the incidents observed. Approximately 80 percent of the incidents reduced the capacity of the freeway by about one-half or more.

Freeway operational improvements have been implemented or proposed to improve the level of service provided during incidents. Several of these systems have consisted of some form of variable-message signs (2-6). One of the chief operational objectives of these signs is to increase the effective capacity of the freeway corridor during incidents on the freeway by achieving a higher utilization of the adjacent frontage road and surface street system. Driver preference questionnaire studies indicate that drivers will divert around congestion if accurate, reliable, and timely traffic information is provided to them. This diversion could occur from the freeway, at the frontage roads, or at major intersections located within the freeway corridor (7). One measure of the likelihood and desirability of diversion is the travel-time saving that may occur to motorists if they are diverted (7, 8). This evaluation requires an estimate of the travel times along the alternate route and along the freeway during the incident conditions.

This paper presents the development of a method for predicting the time a motorist will need to travel from selected freeway locations to the end of the freeway system during incident conditions on the freeway. It is predictive in that it computes an esti-

*Mr. Friebele was with the Texas Transportation Institute when this paper was written.

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mate of what a motorist's travel time would be if he were to enter the freeway at a selected location at a given time. Speeds, volumes, and other operational measures together with the speed and location of shock waves can also be predicted. Previous methods for calculating travel times have been based on measured or average speeds in fixed subsections (9, 10) rather than on predictions of changing traffic flow conditions.

DEVELOPMENT OF METHOD

Traffic Flow Theory

The deterministic theory of traffic flow has been shown to be very useful in describing freeway traffic conditions and in providing a basis for a rational explanation of certain observed traffic phenomena (11, 12, 13). The results of several approaches to the deterministic theory of traffic flow have been summarized by Drew (14) in his textbook on traffic flow theory and control. In general, the traffic flow theory has presented several mathematical models that interrelate the traffic flow variables of volume, speed, and density.

One of the more used deterministic theories of traffic flow is Greenshields' well-known linear speed-density model (11):

$$u = u_f - \frac{u_f}{k_j} k \tag{1}$$

or

$$k = k_j - \frac{k_j}{u_f} u \tag{2}$$

where

u = speed of the traffic stream,
u_f = free speed,
k = density of the traffic stream, and
k_j = jam density.

Using the general equation of the traffic stream, $q = ku$, where q is the mean rate of traffic flow, we can formulate the parabolic relations between traffic speed u and volume. Substituting from Eq. 2 for density k into $q = ku$ yields

$$q = k_j u - \frac{k_j}{u_f} u^2 \tag{3}$$

A similar relation exists between volume q and density k. Substituting from Eq. 1 for speed u in $q = ku$ yields

$$q = u_f k - \frac{u_f}{k_j} k^2 \tag{4}$$

Equations 1, 3, and 4 are shown in generalized form in Figures 1a, 1b, and 1c respectively. Also shown is the point on each of the respective curves that represents an assumed traffic flow condition existing on a section of freeway during normal operating conditions. Normal operating conditions are assumed to be free of traffic congestion or incidents that might cause congestion to develop.

Initial Effects of Incident

When an accident occurs on a high-volume freeway, it has been widely observed that a queue forms at the location of the accident. The queue and its resulting congestion then begin backing upstream from the scene of the bottleneck, often for several miles during peak-hour operations. Whitson (15) has presented volume-density plots of freeway operations in Houston during an incident, which clearly illustrate this upstream progression of the queuing area and its corresponding congestion. The frontal boundary of this queue, as it moves upstream, is commonly called the shock wave. Freeway surveillance of traffic operations during incidents has indicated that the shock wave commonly travels from 10 to 20 mph during moderate to heavy traffic conditions.

Whitson (15) also noted that a wave moves downstream from the incident location. This wave denotes the change that occurs downstream of the incident, from normal traffic flow to a much lighter flow. The reduction in the capacity of the freeway caused by an accident, or other lane-blocking incident, thus meters the freeway flow downstream from the site of the incident but causes a queue and congestion to form upstream of it.

Figure 2 shows a graphic summary of freeway traffic conditions upstream and downstream of the incident location while the incident blocks the freeway. The congested queue is bounded by the shock wave and the incident location with the queue having a nearly saturated density k_q that is much higher than the normal density k_n (Fig. 3). Downstream of the incident in the metered flow region, the density is reduced from the normal density k_n existing before the incident to a much lighter metered density k_m, reflecting a higher mean traffic speed. The location of the clearing wave defines the boundary between the metered flow and the as yet undisturbed normal flow region.

Wave Theory

Lighthill and Whitham have presented a theoretical model for computing the speed of a shock wave based on changes in volume and density. The speed of the shock wave is given (16) by

$$W_{u1} = \frac{q_q - q_n}{k_q - k_n} \tag{5}$$

where

W_{u1} = the speed of the shock wave,
k_q = traffic density in the congested queue,
k_n = traffic density during normal operations,
q_q = stream flow rate in the congested queue, and
q_n = stream flow rate during normal operations.

The wave subscript notation refers to the direction of travel of the wave and the position number. That is, W_{u1}, the shock wave, is the speed of the first wave that travels upstream during incident conditions. W_{d1} would be the first wave traveling downstream. As shown in Figure 2, the density k_q in the congested queue is greater than the normal density k_n. The incident is assumed to reduce the capacity of the freeway to q_q, that is, less than the normal flow q_n, which is a requirement if congestion is to form. Thus, the speed of the shock wave W_{u1} will be negative, indicating the wave is moving upstream.

As shown in the volume-density curve in Figure 3, the speed of W_{u1}, the shock wave moving upstream from the location of the incident, is the slope of the chord that connects the point characterizing the traffic condition within the congested queue with the point characterizing normal traffic conditions. The negative speed of W_{u1} is also shown in Figure 3 because the slope of the chord that defines W_{u1} from Eq. 5 is negative.

As shown in Figure 3, the traffic flow rate q_m in the clearing metered section downstream of the bottleneck incident is the same as the bottleneck flow rate q_q, but the density k_m within the metered area is much lower than the density k_q in the congested queueing section. The speed of the metered wave, which is the boundary between the metered and normal traffic operation, is

$$W_{d1} = \frac{q_m - q_n}{k_m - k_n} = \frac{q_q - q_n}{k_m - k_n} \tag{6}$$

where

W_{d1} = speed of the clearing metered wave being the first wave moving downstream from the incident,

q_m = flow rate in the metered section,
q_q = flow rate in the queue and equals q_m,
q_n = normal flow rate,
k_m = density in metered section, and
k_n = normal density.

Because both the numerator and the denominator of Eq. 6 are negative, W_{d1} is positive, indicating that the clearing metered wave is traveling downstream from the site of the incident bottleneck.

After a time T has elapsed since the incident occurred, the incident is assumed to be completely removed from the freeway (Fig. 4). When the incident is removed, the capacity of the freeway is increased, and the vehicles stored upstream of the site of the incident then begin to travel downstream. The flow of these vehicles out of the downstream end of the congested queue also begins to shorten or clear up the queue upstream of the site of the incident. Figure 4 shows a summary of the traffic operating conditions along the affected sections of freeway from the time the incident begins until the freeway traffic operations return to normal sometime after the incident is removed. The shock wave W_{u1} and the clearing metered wave W_{d1} are depicted as the boundary vectors emanating upstream and downstream respectively from point A in Figure 4, which defines the beginning of the incident. The equations shown in Figure 4 for the wave speeds will be developed in a later section.

The freeway traffic flow in the high-density, high-flow region, denoted as region c (capacity) in Figure 4, may be described as generally being unstable flow at or slightly under the maximum flow at normal capacity. For the purposes of this analysis, the average flow and density within this high-density, high-volume section are assumed to be at capacity, noted as the capacity point in Figure 3. As soon as the incident bottleneck is removed from the freeway, this unstable, near-capacity region of flow begins to travel upstream from the incident location (point B, Fig. 4), reducing the queue length, and downstream from the incident, increasing the flow and density downstream.

Associated with the movement upstream of the capacity flow region is the wave W_{u2} shown in Figure 4. Likewise, the wave W_{d2} moves downstream from the site of the incident (when it is removed) that defines the boundary between the capacity flow and the metered regions. Using Figure 3, it follows that

$$W_{u2} = \frac{q_c - q_q}{k_c - k_q} \tag{7}$$

where W_{u2} is the speed of the capacity boundary wave moving upstream, and (q_c, k_c) and (q_q, k_q) define the volume-density operating conditions in the capacity flow region c and congested queue region q respectively shown in Figure 3. Note in Figure 4 that W_{u2} is the second wave that travels upstream.

The boundary of the high-density, capacity-flow region travels downstream at a speed of

$$W_{d2} = \frac{q_c - q_m}{k_c - k_m} = \frac{q_c - q_q}{k_c - k_m} \tag{8}$$

where W_{d2} is the speed of the boundary wave, and (q_c, k_c) and (q_m, k_m) define the volume-density operating conditions in the capacity flow region c and the clear metered region m respectively noted in Figures 3 and 4. Note again that $q_m = q_q$.

As shown in Figure 4, one remaining wave occurs before the freeway traffic conditions return to normal. Sometime after the incident is removed, the capacity flow wave W_{u2} will catch the shock wave W_{u1}, and the congested queue will have been dissipated. At this point, the final clearing wave W_{d3} forms and begins to move downstream. This wave defines the boundary between the high-density capacity flow region and normal traffic flow. The speed of the wave is

$$W_{d3} = \frac{q_c - q_n}{k_c - k_n} \tag{9}$$

where W_{d3} is the speed of the last clearing wave, and (q_o, k_c) and (q_n, k_n) define the volume and density in the capacity flow and normal regions respectively shown in Figures 3 and 4.

Computing Shock Waves From Speed

Freeway surveillance of incidents in Houston has indicated that a very useful and reliable method for readily detecting the occurrence of an incident on the freeway is to measure the change that occurs in the stream speed (or occupancy) in the queuing area immediately upstream of the scene of the incident. This suggests that it would be desirable if the entire freeway traffic flow existing during incident conditions (in essence a mathematical description of Fig. 4) could be related to the normal speed u_n existing before the incident occurred and the average speed within the congested queue u_q. The average speed in the queue could be determined from the incident bottleneck capacity q_q using Eq. 3.

Figure 4 shows that a description of freeway traffic conditions during an incident depends heavily on knowing the speeds and locations of the various waves in time and space and on knowing the duration of the incident. The following development is directed toward relating the previously discussed wave speeds to the normal traffic speed u_n and the queue speed u_q.

The two wave speeds W_{u1} and W_{d1} are of primary interest while the incident forms a bottleneck on the freeway. Note that the shock wave W_{u1} in Eq. 5 can be written as a function of only the normal traffic speed u_n and the speed u_q in the congested queue because $q = f(u)$ from Eq. 3 and $k = f(u)$ from Eq. 2. Because the speed of the shock wave is

$$W_{u1} = \frac{q_q - q_n}{k_q - k_n}$$

based on Eq. 5, substituting for $k = f(u)$ and $q = f(u)$ from Eqs. 2 and 3 yields

$$W_{u1} = \frac{k_j u_q - \frac{k_j}{u_f} u_q^2 - k_j u_n + \frac{k_j}{u_f} u_n^2}{k_j = \frac{k_j}{u_f} u_q - k_j + \frac{k_j}{u_f} u_n} \tag{10}$$

and substracting the k_j's and rearranging yield

$$W_{u1} = \frac{k_j (u_q - u_n) - \frac{k_j}{u_f} (u_q^2 - u_n^2)}{-\frac{k_j}{u_f} (u_q - u_n)} \tag{11}$$

Dividing by $-k_j/u_f$ and by $(u_q - u_n)$ leaves

$$W_{u1} = -u_f + u_n + u_q \tag{12}$$

where W_{u1} is the speed of the shock wave, u_f is the free speed, and u_n and u_q are the normal and queue speeds respectively. For the Greenshields linear speed-density model being used, the speed-volume curve of Figure 1b is symmetrical about the speed at capacity. Thus, the sum of $u_n + u_q$ will be less than u_f so long as the bottleneck capacity flow q_q is less than the normal flow q_n that existed before the incident occurred.

The speed of the clearing metered wave W_{d1}, progressing downstream from the scene of the incident, can be developed in a similar manner because

$$W_{d1} = \frac{q_q - q_n}{k_m - k_n}$$

from Eq. 6. However, k_m must first be related to traffic conditions existing within the queuing section. By referring to Figure 3 and using Eq. 4, which relates $q = f(k)$, we can show that $k_m = f(q)$ is

$$k_m = \frac{k_j}{2} - \sqrt{\frac{k_j^2}{4} - \frac{k_j}{u_f} q_q} \tag{13}$$

Substituting $q_q = f(u_q)$ from Eq. 3 into Eq. 13 yields

$$k_m = \frac{k_j}{2} - \sqrt{\frac{k_j}{4} - \frac{k_j}{u_f} (k_j u_q - \frac{k_j}{u_f} u_q^2)} \tag{14}$$

which reduces to

$$k_m = \frac{k_j}{u_f} u_q \tag{15}$$

Returning to the equation for the metered wave speed of Eq. 6,

$$W_{d1} = \frac{q_q - q_n}{k_m - k_n}$$

the results of Eq. 15 are then substituted for k_m, which yields

$$W_{d1} = \frac{q_q - q_n}{\frac{k_j}{u_f} u_q - k_n} \tag{16}$$

Next, the volume and density relations as a function of speed, Eqs. 2 and 3, are then substituted into Eq. 16, yielding

$$W_{d1} = \frac{k_j u_q - \frac{k_j}{u_f} u_q^2 - k_j u_n + \frac{k_j}{u_f} u_n^2}{\frac{k_j}{u_f} u_q - k_j + \frac{k_j}{u_f} u_n} \tag{17}$$

$$W_{d1} = \frac{k_j (u_q - u_n) - \frac{k_j}{u_f} (u_q^2 - u_n^2)}{\frac{k_j}{u_f} (u_q + u_n) - k_j} \tag{18}$$

Dividing by u_f/k_j yields

$$W_{d1} = \frac{u_f (u_q - u_n) - (u_q - u_n)(u_q + u_n)}{u_q + u_n - u_f} \tag{19}$$

Factoring $-(u_q - u_n)$ results in

$$W_{d1} = \frac{-(u_q - u_n)(u_q + u_n - u_f)}{u_q + u_n - u_f} \tag{20}$$

and dividing out $(u_q + u_n - u_f)$ yields

$$W_{d1} = u_n - u_q \tag{21}$$

where W_{d1} is the clearing metered wave speed, u_n is the normal speed on the freeway before the incident, and u_q is the speed in the congested queue.

As has been shown in Figure 4, when the bottleneck incident is removed, three additional waves are generated. The equations for computing these waves have also been presented. The procedures used to relate the wave speeds to the normal speed u_n and the speed in the congested queue u_q follow the two previous examples. Hence, only the results of these three analyses will be presented:

$$W_{u2} = -\frac{u_f}{2} + u_q \tag{22}$$

$$W_{d2} = \frac{u_f}{2} - u_q \tag{23}$$

$$W_{d3} = -\frac{u_f}{2} + u_n \tag{24}$$

Discussion of Model

The results of the previous equations are summarized in Figure 4. The bottleneck incident occurs at point A in time and space, and it lasts until the time of point B is reached. The maximum queue backup along the freeway from the location of the incident is shown as point C in Figure 4.

Comparisons of different wave speeds are made in the interest of providing additional insight and information on the model's description of freeway operation during the incident. Because it is proposed that W_{u2} must catch the initial shock wave W_{u1}, the difference between them yields the rate of queue dissipation, or

$$W_{u2} - W_{u1} = \frac{u_f}{2} - u_n \tag{25}$$

This difference will be negative as expected because the normal speed u_n is greater than the speed at normal capacity flow $u_f/2$ using Greenshields' linear model of traffic flow. The expected negative difference also follows from the initial assumption that the normal flow was stable before the incident occurred with operating speeds above the speed at capacity (Fig. 1). Eq. 25 confirms the expectation that, the lighter the normal traffic flow is before the incident (a larger u_n), the quicker the queue is dissipated.

For the three waves traveling downstream, the differences indicate that each subsequent wave is slower than the previous one. This suggests that these waves never intersect downstream of the incident—as if all three waves were rays emanating from a common point source. These results are based on the differences between

$$W_{d1} - W_{d2} = u_n - \frac{u_f}{2} \tag{26}$$

and

$$W_{d2} - W_{d3} = u_f - u_n - u_q \tag{27}$$

Both of these differences are positive, indicating that W_{d1} is faster than W_{d2}, which in turn is faster than W_{d3}, the third and final wave traveling downstream. These results are reflected in the respective slopes of the waves shown in Figure 3.

PREDICTION OF FREEWAY TRAVEL TIMES

The procedure for computing the travel times of vehicles on the freeway during incident conditions requires a knowledge of freeway traffic speeds as a function of time and distance. Figure 4 has been shown to define the time and space locations of the four different freeway traffic flow conditions that exist during incident conditions. The average volumes and densities existing within each of these flow regions have been shown in Figure 3. Thus, the average traffic speed within each of the flow regions can be determined using Eq. 1. The traffic speed within each region and two examples of vehicles traveling through a congested freeway section during an incident are shown in Figure 5. Again all speeds are being computed from only two traffic variables, the normal speed u_n and the speed within the congested queue u_q. Recall that u_f is the free-speed parameter in Greenshields' linear speed-density model.

The procedure for computing the travel times of two vehicles will be illustrated. Shown as point A in Figure 5, one vehicle is assumed to be at an entrance ramp at t_0, the time the incident occurs. This vehicle would travel at a speed u_n until it intercepts the shock wave backing up the freeway at B. The speed of the vehicle would then drop considerably to u_q while the vehicle travels through the congested queue. When it passes the incident location at C, the vehicle then enters the high-speed metered region at a speed $u_a = u_f - u_q$. The vehicle is assumed to leave the freeway system at D. The travel time for this vehicle would be $t_D - t_0$.

One feature of the travel-time model is that it permits an immediate prediction, as soon as the incident is detected, of the travel times of vehicles that may enter the freeway some time after the incident occurs. Assume that a vehicle enters the freeway at the on-ramp, point I in Figure 5, 10 min after an incident occurred. Entering the freeway, the vehicle then intercepts the shock wave at J and remains in the queue until the capacity flow wave at K is reached. The vehicle then remains in the capacity flow region, leaving the system at L. The travel time on the freeway from point I to L would be $t_L - t_{10}$.

The time-distance path that a vehicle would trace, e.g., path IJKL in Figure 5, is not known initially for an incident and must be computed in a trial-and-error manner. A computer program, which requires only a few seconds to execute, was written to compute these travel times.

A travel-time solution will be presented for a typical lane blockage incident that occurred on the inbound Gulf Freeway in Houston. A vehicle stalled in the median lane at 8:16 a.m., reducing the capacity by about one-half, and was removed 6 min later at 8:22, as shown in Figure 6. This figure also shows the predicted operating speeds, wave speeds, and average traffic conditions during incident conditions. The incident generated a shock wave having a speed of 11 mph. It moved upstream for 13 min, until 8:29, resulting in a queue backup of about 2½ miles. The shock wave was predicted to arrive at the Griggs ramp at 8:24 a.m. and was observed to arrive at 8:25.

Figure 7 shows the predicted travel time from any freeway location shown to the end of the system if the vehicle were to begin its trip at the time shown. The predicted travel times at 8:16, the time the incident occurred, are higher than the travel times expected just before the incident occurred. Note that the predicted travel times at the Griggs and Lombardy ramps located upstream of the incident increase for about 10 min, 4 min after the blockage was removed.

FEASIBILITY STUDY

The method presented for predicting travel times requires estimates for several variables and parameters. The location, duration, and severity of the incident must be established in addition to the normal average operating speed, speed in queue, and free speed. During real-time operations, all of these would have to be estimated within a short period of time based on real-time traffic data. The accuracy of these estimates would directly affect the accuracy of the travel-time prediction model. Based on the literature available and freeway operations experience, it would appear that an accurate prediction of incident duration would be the most difficult variable to determine (1). Research is currently being conducted in this area to develop the necessary detection and estimation techniques.

Figure 1. Speed, volume, and density relations using Greenshields' model.

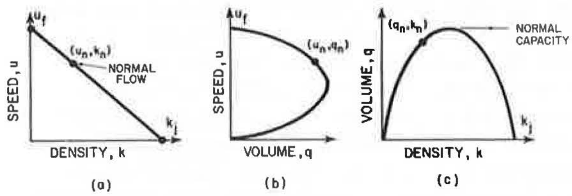


Figure 2. Existing freeway traffic conditions until incident is removed from freeway.

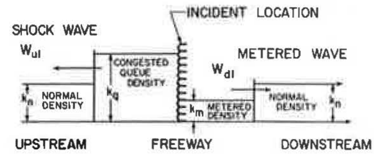


Figure 3. Deterministic relations of five waves caused by incident.

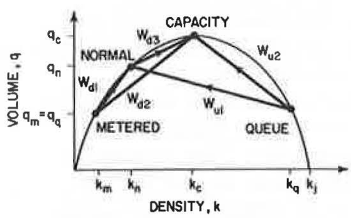


Figure 4. Time-space model of freeway traffic flow conditions and wave speeds.

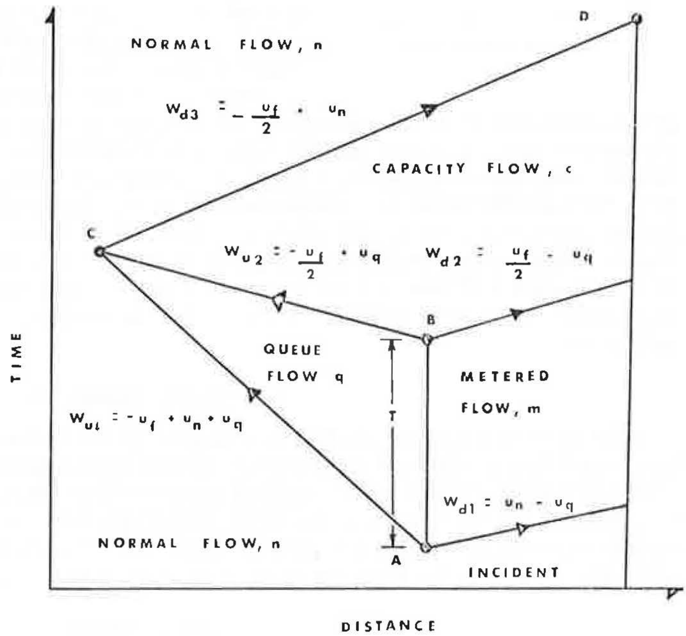


Figure 5. Method of predicting travel times of vehicles traveling through incident conditions.

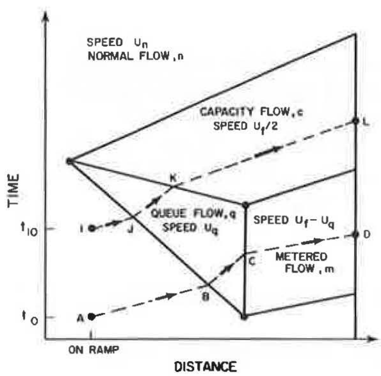


Figure 6. Time-space diagram of traffic conditions for incident on Gulf Freeway.

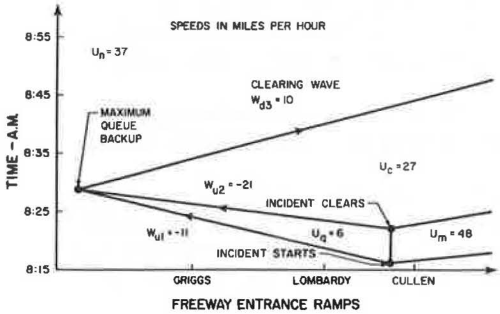


Figure 7. Travel times predicted by model for incident on Gulf Freeway.

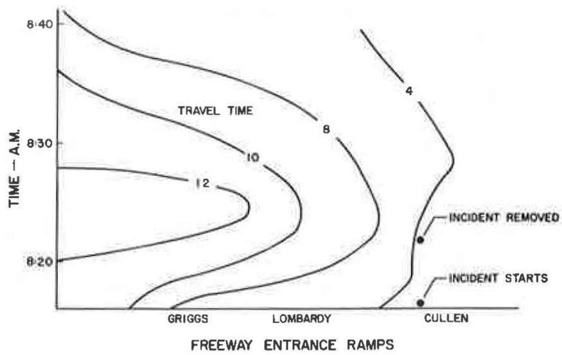
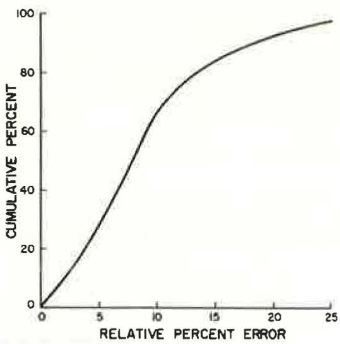


Figure 8. Accuracy of calibrated model for Gulf Freeway, all conditions known.



An initial feasibility study was conducted, however, to determine the accuracy of the method in predicting travel times if all the necessary variables and parameters were accurately determined. One off-peak- and three peak-period incidents that occurred on the Gulf Freeway in Houston were evaluated. Freeway traffic flow was normal and not congested before the lane blockages occurred. The incident data were accurately recorded from television surveillance available in the freeway surveillance center, and resulting freeway traffic flow data were available from computer printout. Ten automobile travel times were manually recorded from the television surveillance for each incident. All travel-time computations were made at a later date. Because each incident occurred at a different location on the freeway, the free speed u_f used in the method was adjusted slightly to provide the best possible fit of the recorded data.

Figure 8 shows the cumulative percentage of the relative percentage of error among the 40 samples of the automobile travel times taken during the four incidents and the computer travel times. Two-thirds of the observed travel times were within 10 percent of the computer travel times, a level felt satisfactory for consideration as reliable information. Most of the larger errors arose when travel times were being predicted for times 10 to 20 min after the incidents occurred. Again, based on the available data, this is the highest accuracy that could be expected to be obtained with accurate estimates of the incident variables under ideal conditions. It remains to be determined how accurately the incident variables can be estimated in real time.

ACKNOWLEDGMENT

This research was conducted in Houston by the Texas Transportation Institute and sponsored by the Texas Highway Department in cooperation with the Federal Highway Administration. The contents of this paper reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This paper does not constitute a standard, specification, or regulation.

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DISCUSSION

Robert L. Gordon, Sperry Systems Management Division, Sperry Rand Corp.

The authors have developed an application of the theory of kinematic waves for freeway control. The techniques described in the paper permit the calculation of wave propagation rates under incident conditions and allow travel times to be computed under these conditions. The authors, however, use the linear Greenshields' speed-density model that results in parabolic relations for volume-speed and volume-density plots. A considerable amount of empirical data exists (e.g., 15) that indicate that speed-volume and speed-density curves that are skewed are generally more representative of freeway traffic flow, particularly where speed limits influence flow. This discussion is intended to describe the general magnitudes of the possible errors that can result in the wave propagation rates computed by the algorithms suggested in the paper as a result of differences among the parabolic model for the flow-density curve and certain empirical data.

Figure 9 shows a volume-speed curve obtained as a result of measurements over three lanes of roadway made on the Van Wyck Expressway in New York City. The data are based on 5-min volume and speed samples, most of which were obtained between speeds of 15 and 50 mph. The figure also shows a hypothetical plot based on the parabolic relation, i.e., Eq. 3 of the paper.

It was necessary to make certain assumptions concerning the parameters of the parabola to develop this plot. Because the maximum flow is a measurable quantity, this quantity was selected to be one of the specified values. Either jam density or free speed may be selected as another value. Jam density was obtained by extrapolating measured data, and the resultant free speed is seen (Fig. 9) to be close to the measured value for this quantity.

Figure 10 shows the plots of Figure 9 converted to volume-density plots by the use of the relation

$$k = q/u$$

In this conversion, differences between mean space speeds and mean time speeds (17) have been ignored, and this may lead to some error in the results of the computation in this discussion.

A normal (upstream) flow of 5,200 vehicles per hour and an incident providing a capacity flow of 2,800 vehicles per hour were assumed in order to compare wave velocities calculated from both the measured data and the parabolic representation. The shock wave speeds, W_{u1}, W_{d1}, W_{u2}, W_{d2}, and W_{d3}, were calculated for the following situations:

Figure 9. Volume-speed curves for vehicle flows.

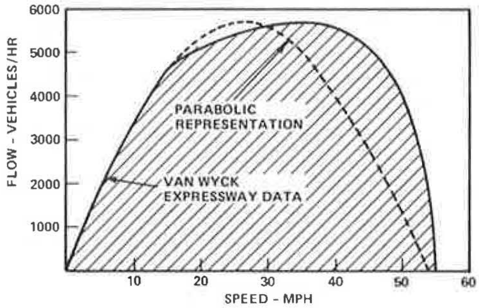
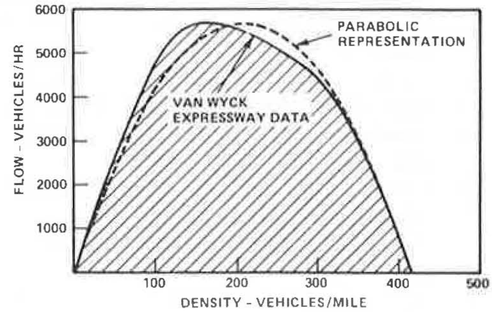


Figure 10. Volume-density curves for vehicle flows.



1. Case 1—Wave velocities were calculated using the measured volume-density data shown in Figure 10 and Eqs. 5, 6, 7, 8, and 9 of the paper. These calculations are assumed to provide the correct wave velocities.
2. Case 2—Wave velocities were calculated using the parabolic representation of the volume-density data shown in Figure 10 in conjunction with Eqs. 5, 6, 7, 8, and 9. These calculations describe the basic fundamental wave speeds if the parabolic representation for the flow conditions described was in fact valid.
3. Case 3—Wave velocities were calculated using the speed measurements for the actual data curve in Figure 9 applied to the computational algorithms (based on the parabolic assumption) described in Eqs. 12, 26, 27, 28, and 29 of the paper. These computations describe the wave velocities that would be calculated based on vehicle-speed measurements that would actually be made on the roadway under the flow conditions described and then processed by the algorithms described in the paper.

Comparison of the results for the three sets of calculations is as follows:

Variable	Case 1, Correct Wave Speed	Case 2, Computed Wave Speed	Case 3, Computed Wave Speed
W_{u1}	-10.1	-11.5	-1.8
W_{d1}	37.0	26.6	36.7
W_{u2}	-14.4	-19.3	-19.2
W_{d2}	27.2	19.0	19.2
W_{d3}	11.2	7.5	17.5

The results show that, when upstream flow conditions are in regions where actual and parabolic curves do not coincide, significant errors can be made in calculating the wave velocities based on the parabolic assumption. It is suggested that the accuracy of the wave velocity computation could be improved by the use of an empirical volume-speed curve based on measured relations, conversion to density by using $k = q/u$, and use of Eqs. 5, 6, 7, 8, and 9 to calculate wave velocity.

Another alternative is to use Eqs. 5, 6, 7, 8 and 9 in conjunction with a more representative analytic model for the moderate- and high-volume cases. Such models that might be considered are the Greenberg model (13) or alternatively a higher order polynomial than that provided by Eq. 4.

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Joseph A. Wattleworth, University of Florida

The authors are to be commended for the development of an extremely interesting mathematical model of conditions near a freeway incident and the application of this model to the very practical purpose of real-time estimation of the measures of freeway operations. This model has the potential of increasing the accuracy of measures intended for the estimation of travel time for the individual vehicles on freeway sections.

Past systems for estimating travel time were based on the measurement of speed at several points along a route and the assumption that this speed was constant over a length of roadway. Normally, the speed at a detector is assumed constant from a point halfway to an upstream detector station to a point halfway to a downstream detector station. This assumption may be reasonably valid in cases in which no shock wave formation takes place. Most practical applications, however, are those in which shock waves do form a congested freeway and an arterial sheet, to name but two.

The model presented by the authors uses simple measurements to infer the location of the shock wave. A vehicle is assumed to travel at one speed until it reaches the shock wave and to travel at another speed thereafter. This model is more refined than the earlier models in that it calculates the (changing) position of the speed change point and does not simply assume that it is halfway between detector stations.

One can examine the maximum speed error between a pair of the detector stations if the present logic is employed, namely, assuming a constant speed from a detector to a point halfway to the next detector. Figure 11 shows a schematic of this situation. The upstream detector station measures a speed V_1, while the downstream station measures a speed V_2. Under the conventional systems, the speed would be assumed to be V_1 from the upstream station to the midpoint and would be assumed to be V_2 from the midpoint to the downstream station.

Under these assumptions the speed of a vehicle in traversing the distance between the detectors can be calculated to be

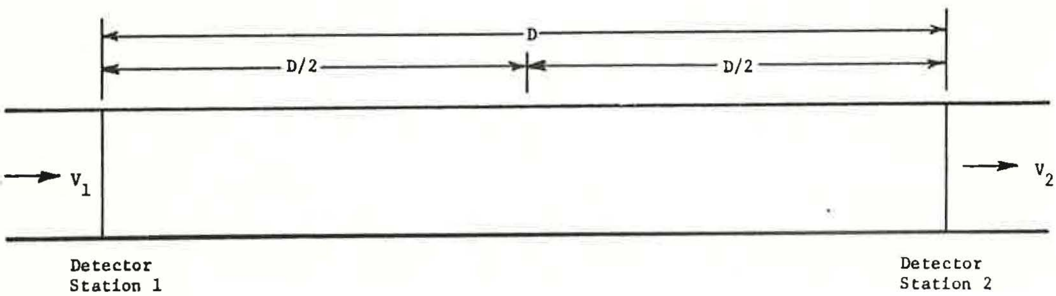
$$V = \frac{2V_1V_2}{V_1 + V_2}$$

If the actual speed over the section is actually V_1 from the upstream detector to a point very close to the downstream detector station, the actual average speed over the distance is V_1. Thus the error in speed is

$$V_1 - V = V_1 - \frac{2V_1V_2}{V_1 + V_2} = \frac{V_1^2 - V_1V_2}{V_1 + V_2} = \frac{V_1(V_1 - V_2)}{V_1 + V_2}$$

In the case of a shock wave due to congestion, V_1 might be about 40 mph and V_2 about 20 mph. The error in this case would be 13 mph. Thus, the method suggested by the authors will potentially improve the accuracy of speed and travel-time estimates over a length of a facility.

Figure 11. Schematic of detector stations.



One must ask several questions regarding the measurement technique and model presented by the authors. First, the shock wave speeds are easily calculated from detector speeds or speed parameters if the linear speed-density relation is assumed (Greenshields' model). Have investigations been made into the relations for calculating shock wave speeds if other traffic flow models are assumed? Are the relations workable?

Second, has an investigation been undertaken into the detector station frequency and travel-time accuracy? The accuracy should be greater than for existing techniques for a given detector spacing, and this analysis would be interesting.

Third, the authors have assumed a straight-pipe section for their analysis. What changes must be made in order to analyze a realistic system such as an urban freeway with exit and entrance ramps? What detector configuration would be required in order to implement the model? In this regard, has consideration been given to the implementation of this model on an arterial street where the incident is a traffic signal and there is an incident at each signalized intersection once each cycle? The model would appear to have the greatest potential for improvement in travel-time estimations on streets (because of the number of incidents) if the model can be adapted to such an application.

In summary, the authors have presented a very interesting and potentially very useful model. The discussion has raised some questions regarding the implementation of the model.

Eugene F. Reilly, New Jersey Department of Transportation

The authors have done a good job in applying the theoretical flow-concentration curve as expressed by Greenshields to the determination of travel time on the freeway. Freeway incident detection and the associated handling of traffic present the engineer with many problems when he attempts to give accurate delay information to the motorist.

Without the aid of a television camera, we must use remote-sensing devices and historical traffic data. The sensors will detect the reduced speeds and flow, or the increased occupancy and density of traffic, which will then be used to estimate the time of occurrence and location of a possible incident. With the passage of sufficient time to verify an incident, action is immediately taken to meter upstream entry ramp traffic or to inform entry traffic of the advisability of taking alternate routes or both.

The task of diverting traffic is the final objective of the authors in the paper under discussion. For the engineer to achieve that same objective, he must have defined his usable network within the corridor; he must have chosen the optimum locations for his changeable-message signs that will be used to give the delay information to the motorist; he must have detailed the messages that will be displayed; and he may also have to develop a method of estimating or measuring the diverted or alternate route traffic flow so that he can determine the travel time of motorists using the alternate route. The difference in travel time between the alternate route and the freeway is one inducement to the motorist to use the alternate.

If the freeway corridor conditions are known, we can concentrate on the items that directly affect the freeway corridor travel time during the time of an incident and consider those motorists who will be given the information about the freeway delay.

The data received at a central control station from the remote sensor will be received up to several minutes after the incident has occurred. If the flow conditions are severe enough, the programmed logic will assume the existence of an incident. Up to this point in time, all traffic that has passed decision points will become part of the backed-up queue. But perhaps the most severe problem now exists for the engineer: How long will the incident last? If the engineer assumes a short time interval for the incident, there may be no reason to divert traffic because the savings in travel time may be but a few minutes for the diverted traffic. Historically, the distribution of the length of time of previous incidents can give the engineer a few alternatives in this regard. It may be reasoned that the engineer should assume short time interval incidents as a matter of policy and then relate the resulting short delays to the motorists if they chose the freeway as their route. What might be expected to result is a growing acceptance of the displayed delay information by the motorists who use the freeway. These motorists will always experience some delay, but rarely less than they are told to expect; and, because the time length of incident will be longer than the engineer assumes, the motorist choosing the freeway will learn to expect that the displayed delay information is usually a minimum amount.

The other alternatives left to the engineer would be the choice of either an average length incident or an incident of long duration. In either of these latter cases, the motorist can never be sure when he would save time by diverting. In at least half the cases when he chooses the freeway, he will experience less delay than he was told. The results of a system that would function on such a policy would be one of total unreliability. The motorist could not accept the displayed delay or time savings information based on his previous experiences, and the engineer would have no reliable method of estimating traffic flows on any of the routes in the corridor.

The second most important aspect of this entire process is determining the location of the incident and verifying the extent of capacity restriction. The information that is displayed to the motorist is again vitally dependent on these factors. Regardless of whether speed, volume, or occupancy, as measured by the remote sensors, is the important parameter, a certain amount of inaccuracy will exist in determining the reduced capacity of the roadway. With the arrival of police on the scene and subsequent traffic handling, the roadway capacity could be further affected. The spacing of the detectors allows additional inaccuracy because the location of the incident has to be estimated downstream of the sensor. Assuming that the incident is half the distance to the next downstream detector, the engineer can estimate the time of its occurrence using the measured flow data to give him the speed of the initial shock wave. By using estimates of location and by making allowances for time to verify the existence of an incident, the engineer maintains as high a degree of accuracy as methods will currently allow.

The first-hand information that television surveillance gives does not overcome the problem of estimating the length of time of the incident. This aspect of the problem will be the major barrier to reliable and timely display data until ongoing research programs can satisfactorily estimate clearance intervals.

With the continued success of the authors and other researchers in this field, the solution of the other problems related to incident detection will make a significant contribution to the engineer in his handling of freeway operations.

AUTHORS' CLOSURE

The authors would like to express their appreciation to Gordon, Wattleworth, and Reilly for their stimulating reviews of the paper. These discussions, in themselves, will provide considerable guidance for future research and development.

The question was raised as to the need for using a traffic flow model other than the Greenshields model used in the paper. Traffic flow data were presented that do not totally follow Greenshields' model. If linear regression had been used to fit the model, it is suggested that a better fit would have resulted. In order to provide the flexibility of describing various traffic streams, the authors are considering the use of the generalized traffic flow model rather than the Greenshields model at the cost of increasing the complexity of the travel-time model.

It was also noted that the input-output flows to the freeway should also be considered. Perhaps the generalized traffic flow model, if used, should be calibrated to closed system data of total travel and total time rather than point location data.

In the proposed application of the travel-time model to driver information systems, the consequences of overestimating or underestimating the duration of incidents were noted. Because it may be difficult to accurately estimate the duration of an incident, the estimate should tend to underestimate the average duration so that a higher respect of the driver information system can be maintained. Research is currently being conducted to develop improved incident duration estimation techniques based on measured operational and environmental data.

CORRELATING INSTRUMENTED CAR RESPONSES WITH CERTAIN GEOMETRIC ELEMENTS OF HIGHWAYS AND ACCIDENTS UTILIZING SHORT SECTION ANALYSIS

Leonard B. West, Jr., Research Triangle Institute, North Carolina; and
Clinton L. Heimbach, North Carolina State University at Raleigh

An analysis procedure is presented for determining the significant changes in instrumented car responses on adjacent short sections of a highway. The sections on which significant responses occur are then correlated by and with highway geometry elements, intersections and grades, and the number of accidents per short section. The procedure is demonstrated by the use of instrumented car data taken on US-70, a four-lane, divided, non-access-controlled highway in Wake County, North Carolina. The first step in the procedure consisted of removing systematic errors in location. Values were calculated by a least squares process for each driver response as measured by the instruments in the car for a specified short section of roadway. These values of the driver's response were compared sequentially for significant change by using an F-test. Those sections in which significant change was found were correlated on a binary basis (i.e., a zero given for the section without significance and a one given for sections with a significant response) with the intersections, grades of more than 4 percent, and accidents. Correlation coefficients were calculated for six different lengths of section. The highest correlations were found between accidents and those significant change sections whose instrumented car responses were speed change, running time, and total time. Significant driver responses did not correlate highly with intersections or grades.

•PRIOR to 1950, investigators in the field of highway accident research in the United States were, for the most part, concerned with the role of the driver in accident causation. This research focused on the inattentive, careless, or dangerous acts of the driver in relation to accident involvement. The roles of the vehicle and the roadway environment in highway accidents received scant attention.

In June 1958, the President's Committee for Traffic Safety held a conference in Williamsburg, Virginia, to discuss future research strategies in highway accident research (19). From the various proposals set forth, it was clear that the conference participants were in agreement that they were dealing with a dynamic system. Moreover, the three interacting elements of this system were considered to be the driver, the vehicle, and the highway environment. This system concept for future highway accident research strategy had the effect of focusing attention on the vehicle and the roadway as well as the driver in the accident investigation process. The singular concern with the driver in accident research gave way to a consideration of the driver as one of the elements in an interacting system. The final development in this research strategy has been the assumption that driver failures are inevitable. Therefore, the roadway environment and vehicle should be made as safe as possible so that driver injuries are minimized.

The precise manner in which the driver performs the vehicle tracking task as he moves through the highway system is not well defined. However, it is assumed, and

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has generally been confirmed, that the majority of drivers have a stable movement pattern in which they attempt to travel at a uniform speed consistent with the nature of the highway and their trip purpose. It has further been postulated and confirmed that most drivers, consciously or unconsciously, try to minimize deviations from the speed at which they desire to travel. The system concept would suggest that the tracking task of the driver, which consists of controlling changes of direction and speed for the vehicle, can be influenced by the characteristics of each driver's vehicle, the geometry of the roadway, and the density and pattern of other traffic moving on the roadway.

The assumption of stable operating characteristics for the majority of all drivers, subject only to the driver's interaction with his vehicle and the roadway environment, leads to a fundamental assumption on which this thesis is based: The manner in which the driver performs the tracking task in response to a given roadway environment is a direct measure of the quality of movement in that environment. Poor quality of movement would be characterized by large deviations from a stable pattern. For example, if a car is operated on an open road in the absence of all other traffic, the driver attempts to maintain a uniform speed, subject only to the geometry of the roadway. If that roadway consists of long tangents and flat curves, the driver can come very close to achieving a uniform speed over the entire length of roadway. But if that same roadway becomes cluttered with traffic, or if the roadway winds over hills and around curves, the driver will be frustrated in his attempt to drive at a constant speed.

The concept of stable movement characteristics for each driver can be extended to its collective effect when integrated over all vehicles in the traffic stream. The collective judgment of all drivers relative to the proper speed for any given roadway environment results in the average speed for the entire traffic stream. Again, as in the case of the individual driver, the more stable the traffic stream flow is, the less the speed variability of individual vehicle speeds about the overall mean speed is and the lower the likelihood of accidents on that highway is.

In a systems concept, highway accidents are treated as systems failures that are caused by the driver, the vehicle, the roadway, or some combination thereof. Those defects that are an inherent part of the system can be studied with a view toward eliminating or reducing their effect. It has been widely postulated by other investigators that the location of inherent highway system defects can be determined by investigating the driver-vehicle response to the system and correlating these responses with the roadway elements at these same locations. This is in essence the subject matter of this investigation. Driver-vehicle responses for five different drivers were studied for one functionally homogeneous highway, using a single instrumented vehicle for collection of the data. Earlier investigations have postulated and demonstrated that driver-vehicle responses tend to be a measure of the accident potential for extended sections of highway. However, little or no work has been done relating driver-vehicle responses for short sections of highway to the geometric design characteristics of these short sections and to the relative hazard of the section as measured by the accident history occurring within the possible field of view of the driver. The major thrust of the investigation that follows is to develop a technique using least squares principles to provide an estimate of the collective driver-vehicle actions on juxtaposed sections of highways and to test the hypothesis that the accident potential of discrete highway locations can be identified from driver-vehicle responses to those locations.

METHOD OF ANALYSIS

Most of the prior work utilizing the instrumented vehicle parameters has utilized data collected over extended homogeneous sections of highway (15, 20, 22, 23). The extended section approach assumes that there is no difference in the road linearly. That is to say, a sample taken at any point along the road section is just as good as any other, and variations that may exist are independent of each other. If the slight correlations that were found by Zimmerman (24), Beeson (2), and Hooper (14) were in fact true, then the Drivometer events by section are not independent but are directly related to the section over which the values were obtained. It then follows that, for each segment of road, there exists a representative value of driver-vehicle-road interaction that may be measured by the driver responses.

This representative value should underlie each run made on the section, but the true value may be masked by random events that can occur on the section while the run is being recorded. The true minimum value of driver-vehicle-road interaction could be obtained by driving the vehicle through the section under consideration in the absence of transient vehicle interference or when the probability of this interference is quite low. These values could be related more directly to the geometric design elements of the road, the terrain, and the other cultural elements of the highway that are of a permanent nature. The value of this measurement must be regarded as quite low in utility because it would not present the driver-vehicle-road interaction as measured by the driver responses in a normal roadway environment. A useful measurement would be the value that reflects the normal environment. This measurement will have associated with it a higher likelihood of an uncertain event occurring. Several examples of the transient highway events that could occur are as follows:

1. A vehicle pulls onto the road forcing the vehicles following to change position or speed,
2. A vehicle queue collects behind a slow-moving vehicle,
3. A passing maneuver is made, or
4. Inattention to the driving task requires compensations to keep the vehicle in correct attitude.

If there is a set pattern of response to the driver-vehicle-road interaction that will not be masked by the transient events, then these values should be useful to the highway engineers and administrators. Research reviewed indicated little success by prior investigators in relating driver responses to specific highway elements by regression. Hence, it was felt that an analysis should be made by the response variables without requiring them to be related except to sections of highway on which they were recorded. The remainder of this section gives the details of the methodology used to determine the driver response per section and the subsequent analysis that was performed.

Distance Adjustment

A study of the odometer data at the beginning and end of each run indicated that there were variations present in the distance measurement. These variations could have been caused by many factors including tire wear, air pressure, differences in the manner in which each driver performed the tracking task, passing maneuvers, other lane changes, and transient lapses of driver attention to the driving task, but the distance from the start point to the end point must be constant for any definite path. Because a set of tires will provide adequate service for many runs, this source of variation was felt to be systematic and easily corrected by simple proportioning. An automobile tire will likewise maintain a fairly constant pressure throughout a 6-mile drive. The variation in driver behavior was not found to be significant in prior studies.

The passing maneuver or lane change may also be a contributor to the variation in distance between the start and end points. However, by instruction the instrumented car floats with the traffic stream. The driver of the car does not pass another vehicle unless the floating car is first passed. Moreover, if two 12-ft traffic lanes and a 60-mph travel speed are given, the additional distance that is traveled is on the order of 0.2 ft per lane change. Therefore, it was assumed that the passing maneuver and lane changes were of little consequence.

Each odometer reading was proportioned to a distance taken from the North Carolina Department of Transportation straight-line diagram by use of the equation

$$ND_i = FD_i \times \frac{TD}{\sum_{j=1}^{n} FD} \qquad (1)$$

where ND_i is the normalized distance for a section i of the run, FD_i is the odometer distance for section i, TD is the base distance, and n is the number of sections whose distance was measured in a run j. This proportioning placed all odometer readings to the same base, but it also distributed any localized errors throughout the entire run.

Formulation of the Matrix

There is now associated with each drivometer reading a standardized distance. The reading y_{ij} associated with the standardized distance reflects both the true value, μ_n, and the random component, ϵ. This may be written in mathematical terms as the following:

$$y_{ij} = \sum_{n=1} \mu_n + \epsilon \tag{2}$$

where i is the distance point at the beginning of a subsection, and j is the distance point at the end of the subsection.

If the road were divided into sections of fixed length, for example, 0.05 mile, a series of equations could be written that would relate the y_{ij} to a particular subsection of the highway. For example, let n be the number of sections of length 0.05 mile; a section of road 2 miles long would have 40 such sections. A y_{ij} reading taken between 0.16 and 0.25 mile (y_{16-25}) can be represented in μ_n form as

$$y_{16-25} = 1.0\mu_4 + 1.0\mu_5 + \epsilon \tag{2a}$$

and

$$y_{17-26} = 0.8\mu_4 + 1.0\mu_5 + 0.2\mu_6 + \epsilon \tag{2b}$$

This process is easily programmed for the computer and can use each of the Drivometer readings to form a series of equations in μ and y. These equations can be written in matrix notation as

$$DU = Y \tag{3}$$

where D is the distance matrix and is m rows by n columns, and U is a matrix $n \times 8$ and Y is a matrix $m \times 8$. This can be solved by the least squares technique for μ, a representative value of the driver-vehicle interaction:

$$\mu = (D^T D)^{-1} D^T Y \tag{4}$$

The variance of the μ is

$$\sigma_\mu^2 = \epsilon(\mu\mu^T) - \epsilon(\mu)\,\epsilon(\mu^T) \tag{5}$$

When the data are ordered by initial coefficients, the $D^T D$ matrix is a j width band matrix. The value of j depends only on the maximum distance over which the particular observations were taken.

Because of symmetry, the formation of the $D^T D$ matrix on the computer becomes one of bookkeeping the data by correlating their computer location and their "true" location. The bookkeeping problem was solved, and the matrix was formed.

Solving the Formulation

Investigation into available programming aids revealed that the Cholesky method of solution of symmetric positive, definite, banded matrices was programmed as subroutine MCHB and was available in the IBM System 360 Scientific Subroutine Package at the Triangle Universities Computation Center. The Cholesky method is a triangulation process. The coefficient matrix $(D^T D)^{-1}$, which we now denote as A, can be transformed into a matrix form $(Tu)^T Tu$ because it is a symmetric, positive, definite matrix.

The Cholesky method determines the values of the t_{kj} by

$$t_{kj} = (1/t_{kk}) \left(a_{kj} - \sum_{i=1}^{K-1} t_{ij}\, t_{ij} \right) \tag{6}$$

$$K = 1, \ldots, n$$

$$j = k,\ k+1, \ldots, n$$

where a_{kj} and t_{kj} are the elements of the matrices A and Tu respectively.

The inverse of the traingular matrix form $(Tu)^T Tu$ is found by

$$S = (Tu^T)^{-1} (D^T Y) \tag{7}$$

and then by forming $(Tu)^{-1}S$. This is seen to be $(Tu)^{-1}(Tu^T)^{-1}D^TY$ or $(D^TD)^{-1}D^TY$, the solution sought in Eq. 4. The inverse of $(D^TD)^{-1}$ is found by setting the right-hand side of the equation equal to I, the identity matrix. The utilization of this method allows the solution of all matrices, μ or X, to be computed quickly and efficiently.

Testing of Responses

The results of the data analysis were then tested to locate those contiguous distance increments in which a significantly different response was found. The test used was a standard F-test:

$$F = \frac{(x_{i,j} - x_{i,j+1})^2}{\sigma_{i,j} + \sigma_{i,j+1} - (2\ \mathrm{cov}_{j,j+1})} \tag{8}$$

where F is the standard F statistic, $x_{i,j}$ is the i th variable in the j th position, $\sigma_{i,j}$ is the variance of $x_{i,j}$, and $\mathrm{cov}_{i,j}$ is the covariance of the x's. The test was performed sequentially for all neighboring distance increments.

The sections that had significantly different responses were indicated in binary form, and a correlation coefficient r was found among the response sections, accidents, intersections, and hills.

Computer Programming

The methodology described in this section was programmed in FORTRAN IV on the IBM 360/Model 75. Because of the size of the data arrays, the programming was done in five segments. The sequential running of all five programs was required for a complete evaluation of the data. Program 1 read the raw data and checked for coding errors. It converted all distances to the same base and sorted the adjusted data into ascending distance increments. The sorted data were stored on disk for utilization by program 2. Program 2 read the data generated by program 1 and computed the two matrices, D^TD and D^TY, which were then stored on disk. Program 3 read the program 2 data and computed $(D^TD)^{-1}DTY$ for the distance interval involved and stored this result on disk. Program 4 read the data files from programs 1 and 3 and computed the variance of each event and an F-value. Sections with an F-value greater than 4 were selected as significant and were input to program 5. Program 5 computed the correlation of the distance that had produced a significant change in response with the accident, hills, and intersections. It also plotted the significant variables by location.

FINDINGS

This section reports the findings from the analysis of the instrumented-vehicle data collected for both directions of travel on US-70 in Wake County, North Carolina, by the methodology described in the previous section.

The computer programs described in the previous section required two items of data in addition to those provided by the transcribed film of the instrumentation readings. These were the correct length of the road section and the distance increment to be used for the analysis subsection length. The correct length of the road was obtained from the analysis performed by Brothers (3) because his distances agreed with the North Carolina Department of Transportation straight-line diagram. The adoption of a distance increment was a greater problem because of computer programming requirements. The initial attempts to solve this problem utilized a distance increment of 0.01 mile, the smallest reading of the odometer. This 0.01-mile value produced a matrix that was 577 × 577 and could not be directly handled in computer core. A method for computing and storing the matrices was formulated that compressed the storage required and used logical variables to reduce computer execution time by a factor of 10. This program was completed, debugged, and executed. The results were not as expected. The 30 random starts and subsequent observations (more than 1,800) of the data acquisition process did not provide sufficient independent equations to allow solution. The method utilized in compressing the matrix was based, in part, on a distance of 0.01 mile. The change to other distances required the rewriting of programs 2 and 3 to allow for the use of any distance interval, with the only restriction being the number of sections. In due course, programs 2 and 3 were rewritten, debugged, and executed.

A least squares solution can provide unreasonable answers if the data do not allow elements of the solution to be negative. Because of the manner in which the data in this study were taken, all values should be either positive or zero; but relatively rare responses, such as brake applications, require a very long distance for a nonnegative response. Because of this, the commonly occurring events, running time and total time, were selected as control criteria. The data were then subdivided in several ways in an attempt to determine the maximum number of instrumented car runs that would meet the desired criteria. The final data sets were formed by subdividing the data into halves based on the half-hour periods during which observations were made; i.e., one group included the data from all runs whose start time was on the hour, and the other group was formed from those runs whose start time was on the half hour.

Determination of Significantly Different Sections

Table 1 gives the predicted values of the eight measurements recorded by the instrumented car for three distance increments, 0.05, 0.10, and 0.15 mile. Table 2 gives a sample of the summary form denoting those locations in which there was a significant change in the recorded driver responses from the previous section as determined by an F-Test. The road sections that were significantly different are designated by a 1 or a 2 beneath the type of driver response.

As an example of the findings given in Table 2, consider distance unit (line) 12. The unit is 0.6 (0.05 × 12) mile from the start of the instrumented car run. It is located near the top of a hill and is not at an intersection. A 2 in the brake application column indicates a significant variation in total time from this 0.05-mile section to the next one, and a 1 for the 0.10- and 0.15-mile sections indicates a significant variation.

A 2 in Table 2 indicates those sections in which significant variations were found in both the hour and half-hour divisions of the data. The absence of a 2 in the table is an indication that there is a lack of correlation between the driver responses and the elements of the roadway, for, if such a correlation were to exist, the significant changes in driver responses on each run would occur at the same location. A primary finding of this research is that the intersection measured by this set of driver-vehicle-roadway instruments is not at a level that allows repeatability.

For US-70, the greatest number of significant changes occurred in the travel time-running time responses. The smallest number of significant deviations was found in direction change. (US-70 is almost straight.) There are several points of interest apparent on examination of all the tables:

1. For total time, direction 1 shows more deviations than does direction 2. Direction 1 is outbound from Raleigh, westward toward NC-1002. The greatest number of

Table 1. Predicted values.

Distance Unit	Event: Total Time	Small Steering Reversal	Large Steering Reversal	Brake Application	Direction Change	Accelerator Movement	2½-mph Speed Change	Running Time
1	9.86	7.32	2.39	-0.00	0.80	0.54	6.75	9.91
2	9.41	7.70	3.29	0.00	0.85	0.20	7.00	9.07
3	9.46	6.62	3.03	-0.00	1.27	0.54	6.32	8.74
4	10.33	5.36	2.46	0.13	0.73	0.30	12.30	10.35
5	10.19	6.37	2.43	0.15	1.02	0.41	15.46	10.16
6	9.61	5.51	2.12	-0.03	0.18	0.65	7.93	9.17
7	9.88	4.54	2.74	0.00	1.55	0.17	5.35	9.62
8	9.08	5.69	2.97	-0.00	1.48	0.07	8.74	9.00
9	8.90	4.06	1.79	0.00	0.88	-0.02	4.64	8.95
10	9.42	5.21	2.76	0.00	0.03	0.11	4.46	9.22
11	9.22	6.32	2.85	-0.00	0.36	0.21	3.36	9.25
12	9.38	5.29	2.74	0.09	0.49	0.46	7.11	9.37
13	9.01	4.44	1.42	-0.01	0.86	0.54	9.12	9.01
14	8.83	6.32	3.11	0.00	1.00	0.02	5.30	8.83
15	9.40	5.31	1.45	-0.00	0.65	0.33	5.95	9.44
16	8.86	5.33	2.31	0.00	0.42	0.13	6.41	8.54
17	9.67	6.60	2.92	-0.00	0.87	0.18	6.49	9.52
18	8.80	6.51	3.16	0.00	0.92	0.01	7.32	8.73
19	9.16	5.63	2.17	-0.00	0.75	0.06	13.48	9.17
20	8.82	6.04	2.28	-0.00	0.24	0.29	5.52	8.82
21	9.18	6.89	2.95	0.00	1.95	0.35	6.07	9.19
22	9.06	5.60	2.19	-0.00	0.56	0.26	5.27	8.96
23	9.34	5.09	3.20	0.00	1.03	0.22	8.86	9.27
24	9.20	5.10	2.22	-0.00	0.70	0.07	6.38	8.87
25	8.95	6.32	2.40	0.00	0.49	0.01	5.80	8.97
26	9.22	5.21	2.94	-0.01	0.37	-0.03	6.51	9.27
27	9.19	4.86	2.50	0.06	0.35	0.52	9.42	8.79
28	10.16	5.36	2.52	0.05	0.51	0.75	9.36	10.01
29	8.91	6.75	2.24	0.04	0.41	0.09	7.85	8.93
30	9.44	5.68	2.82	-0.01	0.22	0.16	5.68	9.44
31	9.03	5.23	2.68	0.00	0.17	0.10	5.20	9.03
32	8.98	4.73	1.62	-0.00	0.39	0.25	7.67	8.98
33	9.45	4.15	2.58	0.00	0.22	0.16	5.10	9.45
34	8.84	4.66	1.56	-0.00	0.32	0.30	8.40	8.83
35	9.22	5.53	2.79	0.00	0.71	0.71	6.80	9.30
36	9.51	4.19	1.84	-0.00	1.60	0.32	6.36	9.01
37	8.83	5.05	1.28	0.00	0.40	0.23	5.47	8.90
38	9.07	4.57	2.85	-0.02	0.92	0.11	7.84	8.99
39	7.20	5.12	1.90	0.46	2.10	0.70	23.72	7.22

Table 2. Location of significantly different driver response, direction 1.

Distance Unit	Event: Total Time 0.05 Mile	Large Steering Reversal 0.05 Mile	Large Steering Reversal 0.10 Mile	Brake Application 0.05 Mile	Brake Application 0.10 Mile	Brake Application 0.15 Mile	Direction Change 0.05 Mile	Direction Change 0.10 Mile	Direction Change 0.15 Mile	Running Time 0.05 Mile	Accidents per 0.10 Mile
1											
2											2
3	1									1	
4	1		1				1			1	0
5	1	1	1				1			1	
6		1					1			1	7
7		1		1			1				
8		1		1			1				5
9				1			1		1		
10				1		1	1	1	1		0
11				1		1	1	1	1		
12				2	1	1	1	1	1		1
13				1	1	1		1	1		
14				1		1			1		2
15				1							
16											8
17											
18											2
19											
20											8

deviations occurs in the segment between NC-1727 and NC-1664. This area has numerous side roads.

2. Small steering movements were significant between NC-1666 and just past NC-1664 on the 5 percent downgrade.

3. Large steering movements preceded and followed the total time deviations noted previously.

4. Brake applications occurred after or during large steering movements.

5. Accelerator application occurred on hills and was occasionally intermixed with brake motions.

6. Speed changes preceded or followed the accelerator applications.

7. Running time almost mirrored the total time.

A comparison of all the tables indicates the effect of the more urbanized area closer to Raleigh.

Correlation of Significantly Different Sections With Accidents, Intersections, and Grades

The sections that were found to have significantly different responses from the sections preceding were coded in binary form; i.e., a 1 is significant, a 0 is not. The coding was on a 0.05-mile basis. The 0.10- and 0.15-mile sections were coded as two or three consecutive 0.05-mile sections. Three other elements of highway environment were also coded. These elements (the stimuli) are the number of accidents, the number of intersections, and the number of hills with grades greater than 4 percent. Accidents and intersections were coded per 0.05 mile, and hills were coded in binary form using 1 if present and 0 if not.

It can be postulated that any individual driver's response might be due to a stimulus that was not physically a part of the section on which the response occurred. For example, a driver might see a vehicle start to cross the road some distance ahead and ease the accelerator back; or he might detect a slow-moving vehicle and change lanes so that his own speed would not have to be changed. To provide for this possibility, a correlation coefficient was found between the section on which driver-vehicle responses were significantly different from the preceding section and the three stimuli over changing distance intervals. The set of significant change responses was held constant. The set of stimuli was formed for six different lengths. First, correlations were found assuming the stimulus was given only by the presence or absence of a response on the 0.05-mile section. The section length was then incremented by 0.05 mile until it was 0.30 mile long. Correlations were found for each increment. If the significantly different responses are a true indicator of the fixed stimuli, the correlations should be similar for each direction even though the response from the Heimbach et al. (11) study showed the directions to be significantly different. Each direction of travel on US-70 can be considered as a separate road.

A curve of the simple correlation between the responses and their stimuli was plotted for each direction of travel. These curves were then examined for similarity of pattern and range. Pattern of the curves was observed through a coarse examination of the slope and the range from a comparison of the low values and high values and the amount of their difference. Table 3 gives a summary of these results. Table 3 has two points of interest through all stimuli. First, direction change is the only response that is constant in both pattern and range for all three stimuli, but it is also the least frequent event. This response is not under driver control and reflects only the 5-deg changes in the vehicle direction. The second point of interest concerns accelerator applications. Correlations between the three stimuli and accelerator applications were reversed when the vehicle direction changed. A possible explanation of this reversal is that the positive grades in one direction become negative grades in the other direction. The pattern of correlations for one direction is reversed when examined from the other direction. The pattern for direction 1 has positive correlations and decreases with increasing distance units; the pattern for direction 2 has negative correlations and decreases with increasing distance units. The 4 percent restriction excludes minor grades and leaves only those grades that are significant. The constraint of the floating car technique to maintain speed would cause the accelerator movement to occur at different points on the road.

Table 3. Correlation coefficients and distance curves, directions 1 and 2.

	Intersection		Hill		Accident	
Event	Pattern	Range	Pattern	Range	Pattern	Range
Total time	1	1			1	1
Small steering reversal		1				1
Large steering reversal						
Brake application	1	1				
Direction change	1	1	1	1	1	1
Accelerator application	2		2		2	
Speed change	1				1	1
Running time		1			1	1

Note: 1 = pattern and range are similar for directions 1 and 2; 2 = pattern and range are dissimilar for directions 1 and 2.

Table 4. Drivometer events and values.

Event	Value Recorded
Trip time	Elapsed time since the start of a trip (measured in seconds)
Small steering reversal	Count of the number of $^3/_8$ in. or greater movements of the rim of the steering wheel (measured by small wheel in contact with steering column)
Large steering reversal	Count of the $1^1/_2$-in. or greater movements of the rim of the steering wheel (measured by small wheel in contact with steering column)
Brake application	Number of times brake was pressed (measured by electronic switch)
Accelerator application	Number of times accelerator was depressed or released $^1/_4$ in. (measured by linkage attached to the carburetor)
Speed change	Number of times speed varied 2 mph (measured by photoelectric device on speedometer)
Running time	Time vehicle is in motion (measured in seconds)
Direction change	Count of 2.5-deg increments of change in direction (measured by gyrocompass)
Odometer	Distance vehicle has traveled (measured in hundredths of mile)

Intersections yielded similar patterns of correlation among distance for total time, brake application, and speed changes. This supports a well-known conclusion that intersections cause a fluctuation in the traffic and a degradation of the idealized driver-response pattern of minimum fluctuations in speed.

Examination of grade data did not yield any visual correlation pattern or range agreement for any of the eight responses. The vertical alignment as measured by the cutoff grade did not affect the driver response in any systematic way. A similar effort by Newhardt, Herrin, and Rockwell (20) to assess the effects of roadway geometry on driver responses showed that geometrics did not affect the driver response unless there were long, steep (G > 6 percent) grades or sharp (R < 600 ft) curves. The values of speed change, total time, and running time, each with accidents occurring, were in agreement in both pattern and range for all distance units. The correlations for 0.05 mile were low, but correlations at the greater distance were higher, and their patterns were very similar.

CONCLUSIONS AND RECOMMENDATIONS

The foremost result from this research is the demonstration of a technique for analysis of driver-vehicle-roadway interaction. The results are most vividly displayed when the significant sections were added together. A very large number of significant sections were found; but when one set of runs was superimposed on the other, little agreement was found with respect to the location of the significant deviations.

A relation between the various measures given in Table 4 and accidents was not found. As stated previously, accidents are a very rare event when compared to the number of observations made by the drivers of the instrumented car. Accident statistics are commonly given in units per million vehicle-miles, whereas the total distance traveled by the instrumented car going both directions was less than 360 miles. The total exposure during the time was 67,000 vehicle-miles, including from 9:00 p.m. to 6:00 a.m. when the vehicle was not present. The analysis of data on this study indicates that, on a relatively well-designed roadway, the driver does not disrupt his driving actions in a systematic way but essentially drives with a random pattern to his actions. Thus the driver-vehicle-roadway interaction measured by this instrumentation will not locate accident sections.

This lack of agreement has three possible sources: the basic hypothesis, the analysis technique, or the data.

The literature reviewed earlier justifies the basic hypothesis, but it is very simplistic. Many attempts have been made in all areas of human endeavor to develop models of interaction given a series of events. Human behavior cannot be modeled by simplistic models; thus more complex models are required. A complex model is especially indicated when one is dealing with a rare event such as a vehicle collision. The theory still has promise, but this research did not contribute to the empirical support of theories based on acceleration noise, change of speed, or quality of flow. Research should be continued in these areas because a mechanical method of quantifying traffic flow is needed for safety and traffic flow analysis.

The analysis technique used was based on standard least squares methods. This method minimizes the sum of the squared deviations from the mean, which is a very plausible way to determine a value for a particular segment of road. This method does present unreasonable values when count data are used, and this can be remembered when considering the results of the significance testing. The testing procedures were standard F-tests. The entire analysis procedure has been well accepted by both engineers and statisticians, and it has been found to yield good results.

The validity of the data is questionable. As mentioned previously, the Drivometer is largely a digital instrument; i.e., it counts the number of times a certain threshold is reached. This threshold is very critical. Table 4 gives the values required to trigger an event. Three of these values are measures along a continuum and are restricted by the graduation of the measuring instrument. Two of the events, trip and running time, are measured in seconds. The third, odometer, is in hundredths of a mile. However, the distances and speeds involved in a 0.10-mile reduction make this too coarse a division; i.e., 6 sec is 60 mph, and 7 sec is 51 mph. It is unrealistic to have a 9-mph speed fluctuation in 0.10 mile; thus time should have been measured on a finer scale. The odometer measurements in hundredths of a mile and the brake application readings are adequate.

The other five events measured a threshold value. The selection of this threshold value and the type of measure are quite critical to arriving at any significant events. Recent research by Newhardt, Herrin, and Rockwell (20, p. 58) indicates that one item, speed changes, was probably measured with too short an interval. They used an analog-instrumented vehicle to study speed variations and found that the normal interdriver and intradriver standard deviations of travel speeds were in excess of 3 mph. Thus, the 2½-mph speed changes, selected for the Drivometer as significant, will contain a large number of normal fluctuations that are not a real indication of flow perturbations but of random variations in the driver's speed (acceleration noise components). The digitizing of the total magnitude of the change is marked by the sampling process, which is based on a distance and large-time scale. A basic problem in analog-to-digital conversion is the establishment of appropriate sampling frames so that true information is acquired and extraneous information is excluded. For example, a continuous 6-mph speed change that might be significant would appear as two changes, which could also be the normal fluctuation of the driver's speed. Thus a significant percentage of the speed deviations will be normal driver fluctuations and cannot be attributed to any other source.

Newhardt, Herrin, and Rockwell (20, p. 45) also stated that horizontal and vertical geometry did not affect the driver's selection of speeds until the horizontal and vertical geometry exceeded that found on US-70. The combination of their findings with the analyses performed herein proves that the measurements taken by this instrumentation cannot be correlated with the horizontal and vertical geometry of US-70.

The slight correlations found among speed changes, running and total times, and accidents reinforce the trend toward the development of a meaningful relation. Soloman (22) found in a study of accidents on rural highways that the likelihood of one's being involved in an accident increased greatly with deviations from the mean traffic speed. The Research Triangle Institute (21, p. 20) verified this in a study in Indiana. It was found that, for nonintersection accidents, a driver deviating more than 15 mph from the mean traffic speed was more than six times more likely to be involved in an accident than a driver operating at a speed within this speed range.

Time per unit distance is the reciprocal of speed and would reflect the hazard indicated by speed deviations. It is relatively easy to measure and has the added benefit of being continuous and nonzero for any movement sequence so long as the vehicle is in the process of completing the transverse of a road section.

A primary recommendation is that there should be additional research effort directed at these three items in both theory and application. These coarse data do not allow one to speculate as to possible alternatives, but an initial research effort should be directed toward better instrumentation in which the statistical base can be developed. This base can indicate those areas in which further theory development is needed. This instrumentation study should also consider other uses of the equipment, such as driver licensing. However, this research does not indicate a use for any data other than time, speed change, and odometer reading.

Further studies of this type should have instrumentation capable of measuring to 0.01 sec. Such studies should utilize a variable speed change apparatus and be activated by a fifth wheel so that it could be interchanged from vehicle to vehicle without major modifications and would enable multiple uses of the transporter unit.

The following conclusions are presented subject to consideration of the poor quality of the basic data that were used in analysis demonstrations:

1. The dispersion of the correlations of the various driver and vehicle responses with grades or intersections plotted over the six distance increments indicates that driver-vehicle responses cannot be correlated with these items,
2. The slightly higher correlation among speed changes, running time, total time, and accidents further reinforces the development of a meaningful relation between speed deviation and accidents found by prior investigators (7, 11, 21, 32), and

3. Speed changes, although providing a measure of speed variation, do not in themselves correlate with grade, intersections, or accidents.

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URBAN TRANSPORTATION IMPROVEMENTS THROUGH LOW-COST TRAFFIC ENGINEERING MEASURES

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ABRIDGMENT

•AN area in West Philadelphia that has serious and diversified traffic problems was selected as a class project in a graduate traffic engineering course at the University of Pennsylvania. The area was a part of a major Early Action Program planned by the Delaware Valley Regional Planning Commission (DVRPC). The suggested solutions are limited to improvement of existing facilities through regulation and minor redesigns, such as defined by the Traffic Operations Program for Increasing Capacity and Safety (TOPICS).

ANALYSIS OF STUDY AREA

The study area in West Philadelphia, primarily residential in character, is bounded by Baltimore and Woodland Avenues and 40th and 45th Streets. It is situated at the convergence of three important two-way arterials: Baltimore, Chester, and Woodland Avenues, representing the bottleneck of a triangular commuter corridor. Four heavily traveled double-track streetcar lines on the avenues proceed into the tunnel east of 40th Street between Baltimore and Woodland Avenues (known as the "portal"). They have near-side stops at each intersection. A common phenomenon in the area is the tendency for automobile flow along the avenues to be in platoons led by the streetcar.

Parking is allowed along most curbs. Traffic signals operate on a two-phase 60-sec cycle without coordination. Pedestrian crossings are often very long, and there are no signals for pedestrians.

For the study the following data were collected: intersection counts during the a.m. and p.m. peaks, streetcar and passenger counts, speed and delay studies, parking survey, and a physical survey. From these data, the major traffic problems in the area—congestion, low travel speeds, and inadequate safety for streetcars, automobiles, and pedestrians—were found to be caused by the following factors:

1. Streetcar-automobile conflict on Baltimore, Chester, and Woodland Avenues, where both modes use the single lane available for each direction;
2. No traffic signal progression and no signal override provision for the streetcars so that they often suffer double delay although they carry 72 percent of the total passenger volume through the area during the peak hours;
3. Congestion and backup across Baltimore Avenue and 40th Street, a weaving section where streetcars need 40 percent green time, because of the poorly designed streets and the portal area traffic; and
4. The presence of oblique, unsignalized intersections, difficult merges, and dangerous pedestrian crossings.

Planning, Evaluation of Alternatives, and Selection of Proposed Plan

The major planning objectives were to increase speeds of public transportation and automobile traffic in the area; increase capacity of the network, particularly at the most critical points; increase pedestrian convenience and safety; and reduce negative impact of traffic on the area.

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All these objectives had to be achieved without violating the overall project requirement that only low-cost improvements be considered.

Methods for achievement of the objectives include changing traffic flows by establishing one-way operation and closing some sections, changing streetcar line routings and providing tracks separated from other traffic at stops or whole street sections, introducing channelization and lane markings to improve flows through intersections, introducing modern traffic signals and other traffic control devices, and improving parking regulations and ensuring safe pedestrian movements.

Several alternative plans were considered and evaluated on the following set of quantitative and qualitative criteria: directness of automobile and streetcar movements, traffic flow conflicts, transit separation and priority, streetcar-automobile conflicts, pedestrian safety and convenience, level of service for automobile traffic, retention of curb parking, cost, and compatibility with extension of plan to adjacent areas. As a result, the comparative evaluation plan shown in Figure 1 was selected.

THE PROPOSED PLAN

The automobile volumes were reassigned to the revised network, and the obtained flows were used for capacity and signalization analysis. Generally, level of service A is obtained throughout the network. The new 45th Street weave is carrying only 70 percent of the traffic of the present 40th Street weave, yet it has three lanes, is not intersected by streetcars, and is over twice the length. Traffic can proceed on both Chester and Baltimore Avenues without being affected by the stopped streetcar because there are two lanes for one-way movement.

Intersection Design

Substantial revision of the intersections was made without any significant widening of streets. Through improved design it was possible to use only two-phase signals, so that traffic delays were kept to a minimum. Modal separation was a feature that was particularly stressed.

An example of the proposed intersection design is shown in Figure 2. At this intersection, automobiles and streetcars are completely separated by channelization and two-phase signal operation. Pedestrian crosswalks are shortened considerably, and a 14-sec phase is allowed at a pedestrian crossing on Baltimore Avenue east of 45th Street. Streetcars can also actuate this signal in order to enter the westward through-traffic lane after stopping at 45th Street. The existing traffic signal at the Baltimore Avenue-45th Street intersection is eliminated. Volume-capacity analysis for this intersection shows that, during a.m. and p.m. peak periods, the intersection will operate at level of service A on all approach legs. Forty-fifth Street, widened by 2 ft, has three 11-ft wide lanes with an exclusive streetcar lane protected by a raised curb. This street operates during peaks at level of service B.

Transit

The revised network requires relocation of 1,700 ft of track, practically eliminating the streetcar-automobile conflicts. Instead of the existing 27 stop locations, there would be only 15 although the average interstop distance is only increased from 605 to 725 ft. Only 3 stops remain in the single traffic lane on a street, compared to 23 at present.

Pedestrians

No crosswalk in the study area is longer than 44 ft compared to 80 ft at present, and for this a minimum crossing time of 19 sec including 11 sec for clearance has been allowed.

Traffic Regulation

The planning of efficient signalization presented one of the most interesting aspects of this project. Signal phasing and timing were developed with the objectives of providing

Figure 1. Street network for proposed plan.

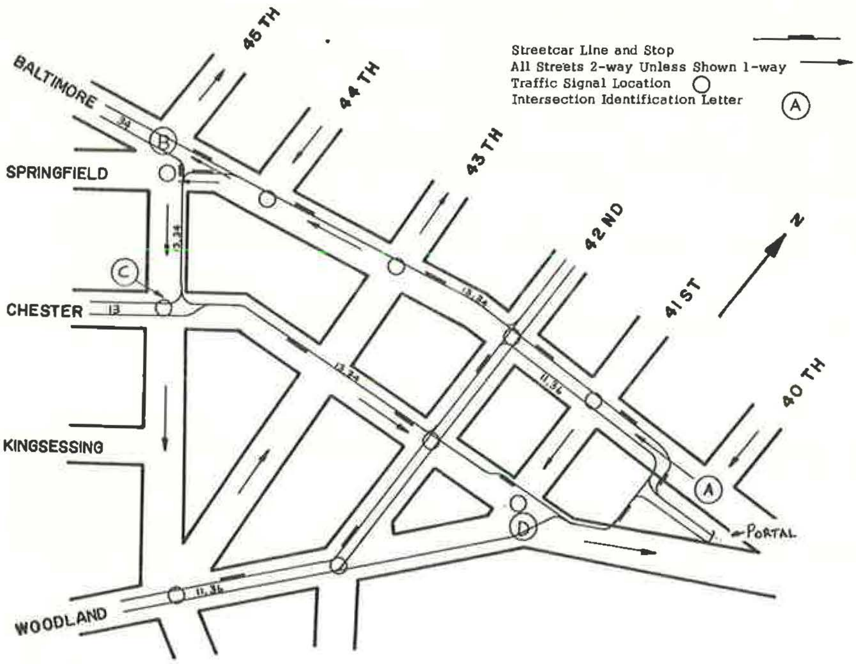
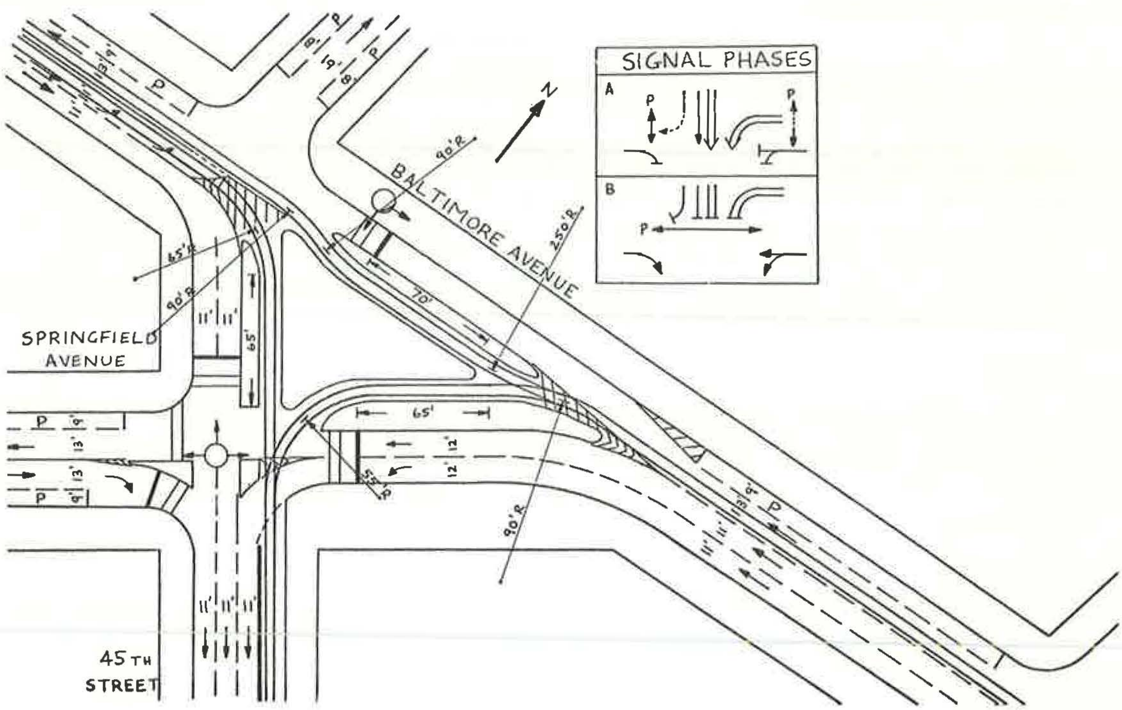


Figure 2. Proposed intersection design.



maximum possible separation of different modes and different movements, ensuring at least level of service B during the peak hours, providing progression for all major movements, and determining timings for minimum person delay, i.e., generally giving priority to transit vehicles.

A progression speed of 25 mph was chosen and wide through-bands were provided for all major flows. For transit priority, it is foreseen that signal preemption devices will be introduced for streetcars to call on the signals when they want to cross the intersection; the call can give them green from the beginning or end of the green time for the other phase.

User Benefits

Because of the limited scope of the study, no comprehensive evaluation of user benefits has been undertaken. However, some estimates have been made of its major component, travel-time savings. Without preempted signals, the average savings to streetcars are 33 percent and 26 percent of their total running time during a.m. and p.m. peaks respectively. Using preempted signals, which would cause slight increases in automobile travel times, these figures increase to 44 percent and 39 percent respectively. Automobile-time savings are even more impressive; they average 56 percent and 60 percent for a.m. and p.m. peaks respectively. These amounts are highly significant; peak-hour savings alone, without and with signal preemption, represent annual time savings of 78,500 and 101,000 person-hours respectively. This includes only the major through movements.

It is possible that the projected speed increases would not be fully realized because the improved conditions would attract higher traffic volumes. This could change the form of benefits: Somewhat smaller time savings and increased convenience would be experienced by a greater number of users. The total benefits would therefore probably still remain in the same range.

SUMMARY AND CONCLUSIONS

The area in West Philadelphia selected for this project represents a typical old-fashioned set of streets designed before motorized traffic. Almost no adjustment to accommodate motorized traffic had been made to these streets. Most streets are two-way without signal coordination. Complicated intersections are not channelized. Streetcar lines, representing the optimal mode because of heavy passenger volumes and tunnel operation from the area to City Hall, and automobile flows both are traffic problems.

The plan adopted on the basis of analysis of all important traffic aspects foresees a number of innovations such as improvement of network flow through one-way street operation, consolidation of streetcar lines to fewer but higher-type sections, separation of their stops to locations not conflicting with traffic, channelization of several intersections, and introduction of modern coordinated signals with transit priority feature. In summary, the proposed plan would virtually eliminate streetcar-automobile conflict, increasing reliability and safety of both; substantially reduce uncontrolled conflicts of automobile flows; result in an estimated speed increase of 50 percent for streetcars and 100 percent for automobiles; increase considerably network capacity in the area; provide for safe and convenient pedestrian movements; reduce the number of parking spaces by 14 percent (the only significant negative effect); and be conducive to extension into adjacent westward areas.

Because the plan is consistent with TOPICS, it involves a relatively low investment, is conducive to immediate implementation, and would be, according to rough estimates, highly cost-effective. Thus, this project clearly shows in general how badly underutilized urban streets can be improved to increase capacity, speed, and safety at a fraction of the cost that new facilities would require.

This plan is currently being considered by the various agencies planning the improvements in this corridor.

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COMPUTER SIMULATION MODEL FOR EVALUATING PRIORITY OPERATIONS ON FREEWAYS

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The computerized model PRIFRE can be used to evaluate any number of different reserved-lane strategies. Although it was primarily intended to evaluate one-way normal priority-lane operations on the same side of the freeway median as the unreserved lanes, it can be used, with some manual interfacing, to evaluate wrong-way reversible lanes, separate bus roadways, and freeway design improvement strategies. Basic assumptions inherent in the model are given. Significant advantages of the PRIFRE over other models are described. The input, output, methodology for using the model, and interpretation of output are described. Traffic performance measures output to the user include single trip times, queuing times, total travel time, total travel distance, and messages. Search procedures and selection of best strategy are discussed, and areas for further research and improvement are indicated.

•THE urban freeway-expressway networks of cities in the United States typically contain congested segments during peak periods. If widened, these segments are frequently soon congested again; additionally, these segments may be bridges or tunnels for which the costs of providing increased vehicular capacity or parallel links are likely to be prohibitive. Automobile congestion and storage in the central city during working hours are also limited, and it is doubtful that much additional street and parking capacity can be provided. Lastly, public attention today is being focused more and more on the aesthetic and ecological disbenefits of overdependence on the automobile, with some observers calling for an outright ban on the use of automobiles in central cities.

One means of alleviating these transportation problems is to explore innovative methods of moving people rather than vehicles. Although entirely new systems to transport people could be constructed at considerable cost, the existing highway network could accommodate many more persons at a much lower cost if a proper redistribution of people to higher occupancy modes could be achieved (1). The reserved-lane concept is one promising method of attempting to achieve this goal; high-occupancy modes—buses and car pools—are given special preference in congested segments of freeways and expressways. This is accomplished by establishing separate lanes for these modes, allowing these vehicles to bypass traffic bottlenecks. Reserved lanes could be simply exclusive bus lanes; however, in most instances a single lane reserved exclusively for buses would be considerably underutilized from a vehicular capacity standpoint. For instance, 60 buses per hour carrying 50 passengers each would considerably underutilize a freeway lane that might be able to carry as many as 800 buses per hour, yet the lane would still be carrying 3,000 passengers per hour, more than might be expected from normal automobile traffic. Therefore, it will usually be more practical to use priority lanes for both buses and car pools and thereby increase the total benefits.

Some of the objectives for improving the urban transportation system through the use of priority-lane operations are to maximize the flow of people, minimize the total

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travel times, improve enviornmental factors such as air and noise pollution, minimize total travel costs, minimize the number of vehicles entering the downtown CBD, improve the quality of travel, and improve the safety of travel. Priority-lane operations should accomplish some or all of these objectives, particularly maximizing the flow of people on existing systems and improving the environment. A significant aspect of priority-lane operations is that it could offer a real choice in travel modes from the user's standpoint by equalizing total travel times by bus and automobile, something that our present transportation systems lack. A subsequent shift to buses and car pools, if exercised by enough people, could reduce traffic congestion significantly, postpone or eliminate the need to build additional freeways, and provide for an inexpensive mass transit alternative to a fixed-guideway rapid transit system.

HISTORICAL BACKGROUND

Traffic engineers around the country today are faced with investigating the many possible priority-lane strategies applicable to their particular situations. Many man-hours of hard work are required to analyze all of the more promising possibilities. A time-saving analytical tool is needed to evaluate and compare the various strategies.

Since 1968 a series of analytical models has been developed at the Institute of Transportation and Traffic Engineering (ITTE) at the University of California, Berkeley, to assist in the evaluation of reserved-lane schemes. These models differ only in their degree of sophistication and extensiveness of applicability. The basic philosophy of each is the same: The total travel time (passenger-hours) for the normal operation condition (no reserved lanes) is compared to the sum of the separate total travel times in the reserved and unreserved lanes for priority operation. The passenger demand for both operations is assumed to remain constant during the peak period.

The first of these models, which sets the outline for the remaining models, was an exclusive bus lane model developed in 1968 by May (4). It was a rudimentary model. The peak-period demand was assumed constant in time and space, and a simple Greenshields flow submodel was employed.

In 1969, Stock (6) improved the model by incorporating the option of a more realistic peak-period demand over time; piecewise linear, triangular, or trapezoidal demand curves could be used. Additionally, a wide variety of speed-flow submodels could be used, including some based on curves given in the Highway Capacity Manual (Fig. 9.1, 5). This model was known as EXCBUS.

Next, Sparks and May (8) in 1969-70 broadened Stock's model to a full priority-lane model, permitting the evaluation of the mixed use of reserved lanes by both buses and car pools. The model, although now both a bus and car pool model, retained the EXCBUS name (19). Also, fairly extensive model validation was done, and the model was applied to a typical situation, the San Francisco-Oakland Bay Bridge. The effects of occupancy shifts, induced by the better level of service in the priority lanes, were also investigated for the first time. The Sparks-May model was used by Alan M. Voorhees and Associates in a feasibility analysis of using priority lanes (15).

As convenient as the preceding models were to use, they lacked the realism of having a demand pattern that could change over distance, as actually happens at the off- and on-ramps of a freeway. In addition, the existing priority-lane models did not consider the effects of capacity changes over distance, such as will occur at grades, lane drops, ramp merges and diverges, and with weaving. Thus the need for a more realistic model was apparent. Such a model for normal freeway operations has been developed at ITTE by Makigami, Woodie, and May (10) as an aid for the evaluation of freeway improvements. This model, known as the freeway model, or FREEQ, does consider the effects of changing demands and capacities over both time and distance.

The latest research culminated in June 1972 with a very sophisticated and useful model named PRIFRE (20). This model has been computerized and can evaluate any number of different reserved-lane strategies.

MODEL DESCRIPTION

PRIFRE was developed primarily to evaluate one-way normal priority-lane operations, i.e., reserved lanes on the same side of the freeway median as the unreserved lanes. However, with some manual interfacing, PRIFRE can be used to evaluate wrong-way reversible lanes, separate bus roadways, and freeway design improvement strategies.

Basic Model Assumptions

Basic assumptions inherent in the model are as follows:

1. Traffic is treated as a compressible fluid where vehicles are not considered individually.
2. Within each time interval, traffic demands remain constant and do not fluctuate within that time interval.
3. Once the traffic demands are loaded onto the freeway, the vehicle demands propagate downstream instanteously, subject of course to capacity constraints.
4. Capacities of subsections, including weaving sections and merging points, are estimated using the Highway Capacity Manual methods.
5. No weaving will be allowed between priority lanes and nonpriority lanes. The reasoning behind this is that no effective formula has been devised to calculate the weaving effect between two lanes moving at differential speeds of 20 to 30 mph. Thus, throughout PRIFRE (the priority-lane model), the priority sections are treated as an isloated roadway with no entry or exit except at the beginning and end of the section.
6. The freeway model, FREEQ, is indeed an accurate model of a freeway under nonpriority operations. It has been validated against actual freeway operations in several cities.
7. No queuing will be allowed at the entrance to the priority lanes. That is, if the demand exceeds capacity for a priority lane, the excess vehicles will be changed to nonpriority status.

Major Improvements to the Model's Realism

Significant improvements over the earlier EXCBUS model are as follows:

1. The introduction of time slices to the study period and subsections to the study section to allow for the handling of a wider range of possible demand patterns, off- and on-ramp traffic, and changes in the capacity of the study section over time and location;
2. The introduction of a varying demand pattern for buses rather than the previously assumed uniform or constant proportion demand pattern;
3. The introduction of both automobile and bus vehicle-occupancy distributions that are a function of time and not constant as currently assumed;
4. The use of three speed-flow relations—one for normal traffic, one for traffic in the reserved lanes, and one for traffic in the unreserved lanes; and
5. The introduction of truck equivalency factors to compensate for the effects of truck traffic on traffic flow.

It is felt that the PRIFRE model as it now exists represents the most comprehensive analytical tool available for evaluating priority operations on freeways.

Manual Checking of Program

An extensive program check was performed on the PRIFRE model to ensure that the computer simulation of freeway situations encountered was as prescribed in the mathematical model formulation and was an accurate representation of actual operations. All possible effort was made to include those features in the model program that facilitate the evaluation of priority-lane operations on freeways. The final assumptions and limitations of the model are documented in the PRIFRE report (20).

Input to the PRIFRE Model

In order to make a reasonable estimation of the travel time on a freeway, we must know the physical and operational characteristics of the freeway and put them into an approximate numerical expression.

In general, freeway sections exhibit a number of varying design and operational features. Thus, to establish a meaningful relation of the average speed of traffic as a function of freeway capacity and traffic demand, it becomes necessary to divide the freeway section into homogeneous subsections that exhibit the properties of constant capacity and demand over their lengths. It is also necessary to itemize the features that affect the capacity of each subsection, such as design speed, number of lanes, lane width, volume of buses, percentage of grade, grade length, number of priority lanes, and location of on- and off-ramps. Traffic factors, such as percentage of trucks, that affect subsection capacities and are hypothesized to be constant over the peak period should also be given in the same table. It is convenient for later analysis to list all of these elements in the format given in Table 1. These elements are used to calculate the capacity of each subsection.

Traffic demands are introduced in the study section in the form of origin-destination (O-D) tables. The entry into the study section and each on-ramp are considered as origins, and each off-ramp and the exit from the study section are considered as destinations. The origins and destinations are numbered consecutively from upstream to downstream.

Because traffic demands during a peak period usually vary, the peak period should be divided into a number of smaller time intervals. In general, a 15-min time interval should be used because 15 min is short enough to simulate the traffic demand change during the peak period and is still a reasonable time interval for predicting traffic demand patterns in the near future. It is therefore necessary to input O-D tables for each time interval during the study period. One O-D table is required for buses and another for other vehicles (Tables 2 and 3).

Although this method of treating traffic demand is complex, it yields the following desirable characteristics:

1. Actual demand patterns are more realistically simulated,
2. Travel times for individual O-D movements can be readily obtained and are essential for evaluating the effectiveness of improvements such as ramp control,
3. The resultant freeway priority-lane model exhibits a flexibility that will facilitate considerations of network traffic movements and patterns, and
4. It facilitates future growth forecasts because each O-D movement can be multiplied by a common factor.

Output From the PRIFRE Model

The PRIFRE computer program conveys many useful results to the user. The format of Table 4 includes the number of freeway lanes, reserved and unreserved, and their original and actual volumes, capacity, volume-capacity ratio, density, average speed, and individual subsection travel times. The number of vehicles in queue is listed for all on-ramps and merge points that have delays for that time slice. Table 4 gives output under normal operations only, and Table 5 gives output under priority operations. Tables 6 and 7 give the single trip time for priority and nonpriority trips respectively for each O-D movement. Under normal operations, Tables 6 and 7 are the simple product of the single trip time matrix multiplied by the O-D table, giving the total travel time in hundredths of vehicle-hours for each O-D movement. When a priority-lane situation exists, only single trip times will be printed out. The next output, which is always printed, is a summary table of incremental and accumulated freeway travel time, input delay, and total travel distance for both priority and nonpriority vehicles (Table 8).

METHODOLOGY FOR USING THE MODEL

For the evaluation of priority-lane operation schemes, it is first necessary to obtain a satisfactory simulation of the existing freeway operations. The procedure for the evaluation of priority-lane operations using the PRIFRE model is shown in Figure 1. Basically, a satisfactory simulation of the existing freeway operation is first obtained as the basis for comparing alternative priority-lane operation schemes as well as for

Table 1. Freewav subsection parameters.

Sub-section	No. of Lanes	Capacity	Reserved Capacity	Length (ft)	Speed-Flow/Capacity (curve no.) Normal	Unre-served	Re-served	Truck Factor	Subsection Description
1	4	10,000		200	3	3	3	0.97	Tollbooth
2P	4	7,200	1,500	400	4	4	4	0.97	Acceleration area, start reserved lanes
3P	4	7,200	1,500	5,400	5	5	5	0.97	Golden Gate Bridge
4P	4	6,890	1,500	600	5	5	5	0.97	
5P	4	6,751	1,500	200	5	5	5	0.97	Wrong-way bus crossover
6P	4	6,752	1,500	600	5	5	5	0.97	Neglect Vista Point on-off ramp
7P	4	6,500	1,500	600	5	5	5	0.72	Alexander (formerly Sausalito)
8P	4	6,500	1,500	3,000	5	5	5	0.72	
9P	4	6,400	1,500	1,050	5	5	5	0.72	Waldo Tunnel
10P	4	6,500	1,500	1,600	5	5	5	0.72	
11P	4	6,961	1,500	2,320	5	5	5	0.97	Spencer
12P	4	7,294	1,500	2,500	5	5	5	0.97	
13P	4	7,294	1,500	550	5	5	5	0.97	Rodeo
14P	4	7,294	1,500	4,850.	5	5	5	0.97	
15P	4	7,294	1,500	1,800	5	5	5	0.97	Marin City
16P	4	7,470	1,500	1,000	5	5	5	0.97	Capacity adjusted, large weave effect
17P	4	7,294	1,500	1,400	5	5	5	0.97	Bus crossover
18	4	7,294		3,000	5	5	5	0.97	Main line, Richardson Bay Bridge
19	3	5,820		800	5	5	5	0.97	
20	3	5,820		3,600	5	5	5	0.97	Main line, south of Tiburon

Table 2. Bus O-D matrix (in buses per hour).

On-Ramp	Off-Ramp 1	2	3	4	5	6	7	8	9	10
1	0	0	0	32	0	0	0	16	0	4
2			8	0	0	16	0	0	0	0
3				8	0	0	0	4	0	0
4					0	0	0	0	0	0
5						12	0	8	0	20
6							0	0	0	0
7							0	16	0	4
8							0	16	0	0
9								8	20	8
10								8	0	4
11									0	40
12										

Table 3. Automobile O-D matrix (in persons per hour).

On-Ramp	Off-Ramp 1	2	3	4	5	6	7	8	9	10
1	65	41	154	235	85	53	198	212	147	151
2	239	107	339	446	146	91	328	288	192	194
3		49	154	207	68	43	154	136	91	52
4			65	76	25	16	56	45	32	20
5				113	37	23	84	78	51	33
6						21	77	68	45	39
7							53	37	36	36
8							91	64	61	63
9								87	87	117
10									36	53
11									77	392
12										403

Table 4. Output for normal freeway operations.

Sub-section	Adjusted On-Ramp Volume	Adjusted Off-Ramp Volume	Original Demand	Volume	Freeway Capacity	Weave Effect	Volume-Capacity Ratio	Density (vehicle/mile/lane)	Speed (mph)	Travel Time (min)	Length (ft)	Queue (ft)	Rate of Flow of Excess Removal (vph)
1	5,966	0	5,966	5,966	10,000	0	0.60	39	38	0.06	200	0	0
2	0	0	5,966	5,966	7,200	0	0.83	32	47	0.10	400	0	0
3	0	0	5,966	5,824	7,200	0	0.81	33	44	1.42	5,400	731	142
4	0	0	5,966	5,824	6,890	0	0.85	63	23	0.30	600	600	142
5	0	0	5,966	5,824	6,751	0	0.86	67	22	0.16	300	300	142
6	0	194	5,966	5,824	6,752	0	0.86	67	22	0.31	600	600	142
7	0	0	5,772	5,630	6,500	0	0.87	64	22	0.31	600	600	142
8	40	0	5,812	5,670	6,500	0	0.87	64	22	1.54	3,000	3,000	142
9	0	0	5,812	5,670	6,400	0	0.89	63	23	0.53	1,050	1,050	142
10	0	491	5,812	5,670	6,500	0	0.87	64	22	0.82	1,600	1,600	142
11	0	0	5,321	5,179	6,961	0	0.74	72	18	1.47	2,320	2,320	142
12	291	19	5,612	5,470	5,470	0	1.00	61	30	0.95	2,500	0	0
13	0	0	5,592	5,441	5,470	0	1.00	61	30	0.21	550	0	0
14	0	63	5,592	5,451	5,470	0	1.00	61	30	1.84	4,850	0	0
15	0	0	5,528	5,388	5,470	0	0.98	55	32	0.46	1,300	0	0
16	511	831	6,039	5,809	6,500	910	0.89	42	46	0.25	1,000	0	0
17	0	0	5,175	4,979	5,470	0	0.91	37	44	0.36	1,400	0	0
18	180	379	5,355	5,159	5,470	0	0.94	44	39	0.87	3,000	0	0
19	0	0	4,953	4,700	5,820	0	0.82	34	47	0.19	800	0	0
20	120	4,900	5,063	4,900	5,820	0	0.84	35	47	0.88	3,600	0	0

calibrating model parameter values. The traffic performance measures of alternative schemes output by the PRIFRE model are then compared manually, and the best schemes are selected through a search process. The major steps of the evaluation procedure are described as follows.

Preparation of Input Data

Freeway Design Characteristics—The freeway design parameters that are necessary to evaluate freeway operations in the PRIFRE model are given in Table 1. The data shown are always for one side of a freeway only; data are input on eight separate card formats, as shown in chapter 3 of the PRIFRE report (20). Up to 50 subsections may be used, and the freeway section studied is usually limited to less than 10 miles, the distance a vehicle is able to travel in one 15-min time slice. The capacities for the freeway subsections can be determined by using the Highway Capacity Manual method or from actual volume measurements. The traffic flow data are usually not available to determine the capacity of the reserved lane on a freeway section under priority operations and must be assumed based on traffic experience and judgment (for the Marin 101 example in the PRIFRE report it was assumed that the capacity of the reserved lane approximates the conditions of tunnel traffic behavior).

The computer program contains five speed-flow/capacity curves—three from the Highway Capacity Manual for design speeds of 50, 60, and 70 mph; one from San Francisco-Oakland Bay Bridge (design speed of 55 mph); and one from I-80 Eastshore Freeway (design speed of 65 mph). The user may also input his own special speed-flow/capacity relation by supplying the appropriate data.

Traffic Characteristics—A study time period of sufficient duration is first selected, and O-D tables for both buses and other vehicles are then prepared. Three kinds of traffic data help in preparing a complete set of O-D tables for the input data to the model: volume counts, aerial photographs, and O-D surveys. The user then selects the appropriate values for bus-equivalency factor and bus-occupancy and automobile-occupancy distributions for each time slice.

Selection of Priority Strategies—The user selects the type of priority-lane strategy he wants to analyze; i.e., number of priority lanes, "wrong-way" lanes, separate busways, and so forth. Next he decides the location, beginning, and end of the priority lane. Then he selects the minimum vehicle-occupancy level for priority qualification status. This priority cutoff might be for buses only, or buses plus car pools with specified minimum-occupancy level. Next, the user considers occupancy shifts from nonpriority automobiles into priority car pools, e.g., 3, 6, and 9 percent shifts are chosen for evaluation and data input on the proper cards. Similarly, modal split shifts, i.e., automobile passenger to bus passenger, may also be chosen, but new O-D tables must be prepared to accomplish this. Finally, growth periods for anticipated traffic demands in future years may also be considered by choosing an appropriate factor by which all O-D table entries are multiplied uniformly.

Interpretation of Output

The PRIFRE program simulates various operation strategies and gives several traffic performance measures to the user including the following.

Single Trip Times—Single trip times for priority vehicles in the reserved lane should always be less than the corresponding single trip time for nonpriority vehicles—otherwise there will be little or no incentive to use the reserved lane. Occupancy shifts can be assumed to occur in direct proportion to the number of minutes saved in the reserved lane. If any single trip times are greater than 15 min, the length of one time slice, the model's limiting assumption concerning total travel time has been exceeded and results should be qualified.

Queuing—The traffic performance output should be examined for excessive queuing lengths and duration. If queuing extends out of the first freeway subsection or past the last time slice at the end of simulation, results should be qualified. The user may also have operation constraints of his own that may not be exceeded without interfering critically with the operation of the freeway. All queues should be checked to see if they are reasonable, using queuing contour maps if available.

Total Travel Times—The program automatically compares the total travel time under priority operations with the total travel time under the corresponding normal operation and gives the total travel time saved (+) or lost (-). A saving in travel time usually indicates a corresponding saving in all other categories selected as measures of effectiveness. Therefore, this output has been selected as the major measure of effectiveness for comparing priority-lane strategies under the search process shown in Figure 2.

Total Travel Distance—This output can be used to calculate vehicle operating cost savings, accident savings, and pollution savings, which are based on the number of vehicle-miles.

Messages—There are several warning messages built in the PRIFRE program that can signal the user of critical or unusual freeway performances, such as an overloaded off-ramp, ramp queuing delays, excessive queue lengths, queue collisions, or excess demand for the reserved lane.

Search Procedure

Generally, results from the many possible priority strategies can best be displayed by tabulating them in some graphical form. Total travel-time saving can be used as the basis of comparison for selecting the more promising strategies for further evaluation. In the search process, one returns to the original assumptions made for the freeway design characteristics, the traffic demand characteristics, and the selection of the priority strategy and adjusts these to suit his particular needs (dashed lines, Fig. 1).

Selection of Best Strategy

The user analyzes the model results, compares them with previous runs, and decides if further strategies or refinements are needed until he is satisfied that the most promising options have been reached. He can then prepare an evaluation summary table showing the best strategies by their benefits and costs. This table can then be presented to those policy-makers who will make the final decision regarding the selection and implementation of the "best" priority-lane operating strategy.

FUTURE RESEARCH

The experience gained during the development and application of the model makes it possible to suggest a number of ways in which further realism and operational ease could be added to the model. Some of these are as follows:

1. Improve the method of inputting raw data into the program. Present formats require much manual manipulation of field data.
2. Make provisions for weaving analyses at the beginning and end of the priority lane. Currently, no adjustments are made for weaving conflicts.
3. Allow lane changing to take place between the priority lanes and the nonpriority lanes. Currently, the program does not allow vehicles to freely enter or leave the priority lane except at the beginning and end of the priority section.
4. Improve the queuing subroutines and capacity analysis in the program in order to make them more efficient and better understood by the user.
5. Include a provision for automatic modal split shifts, i.e., a passenger shift to buses, similar to automobile occupancy shifts to car pools. Now one must manually create new O-D tables reflecting new modal splits, a time-consuming task.
6. Improve the model so as to allow a full network to be modeled. Now parallel facilities are ignored, although they might be potential alternate routes.
7. Improve the model so as to better handle special reserved-lane strategies, such as wrong-way bus lanes, separate busways, and special ramp entries. Some manual manipulation is now required to accomplish this.
8. Enlarge the measures of effectiveness, or objective functions, to include safety, operational costs, pollution costs, parking and congestion costs in CBD's, etc., besides the present travel time and distance.

Table 5. Output for priority-lane operations.

				Reserved Priority Operations							Unreserved or Normal Operations												
Subsection	Adj. On-Ramp Vol.	Adj. Off-Ramp Vol.	Original Demand	No. of Priority Lanes	Vol.	Capacity	v/c	Density[a]	Speed (mph)	Travel Time	Section[b]	No. of Lanes	Vol.	Capacity	Weave Effect	v/c	Density[a]	Speed (mph)	Travel Time	Length (ft)	Queue (ft)	Rate of Flow of Excess Demand (vph)
1	3,845	0	3,865								N	2	3,845	10,000	0	0.38	39	24	0.09	200	0	0
2	0	0	3,845	2	804	3,000	0.27	9	44	0.10	U	2	3,040	3,600	0	0.84	46	33	0.14	400	0	0
3	0	0	3,845	2	804	3,000	0.27	8	52	1.17	U	2	3,040	3,600	0	0.84	33	47	1.32	5,400	0	0
4	0	0	3,845	2	804	3,000	0.27	8	52	0.13	U	2	3,040	3,445	0	0.88	33	45	0.15	600	0	0
.																						.
.																						.
.																						.
14	0	100	3,716	2	804	3,000	0.27	8	52	1.05	U	2	2,347	3,647	0	0.64	50	24	2.60	4,850	4,850	564
15	0	0	3,616	2	804	3,000	0.27	8	52	0.23	U	2	2,247	3,647	0	0.62	75	15	1.01	1,300	1,300	564
16	522	495	4,138	2	804	3,000	0.27	8	52	0.22	U	2	2,760	2,769	966	1.00	46	30	0.38	1,000	0	0
17	0	0	3,554	2	804	3,000	0.27	8	52	0.30	U	2	2,274	3,647	0	0.62	23	49	0.33	1,400	0	0
18	220	236	3,179								N	4	3,299	7,294	0	0.45	18	50	0.68	3,000	0	0
19	0	0	3,505								N	3	3,063	5,820	0	0.53	21	50	0.18	800	0	0
20	119	3,182	3,624								N	3	3,182	5,820	0	0.55	21	50	0.83	3,800	0	0

[a]Vehicle per mile per lane. [b]N = normal and U = reserved.

Table 6. Travel time for one priority trip (in tenths of a minute).

	Off-Ramp						
On-Ramp	1	2	3	4	5	6	7
1	188	392	405	521	570	666	769
2		121	226	340	389	487	588
3			54	169	218	316	417
4				104	153	251	352
5					21	119	220
6						68	169
7							83

Table 7. Travel time for one nonpriority trip (in tenths of a minute).

	Off-Ramp						
On-Ramp	1	2	3	4	5	6	7
1	496	870	1,234	1,640	1,782	1,883	1,983
2		365	700	1,106	1,247	1,349	1,449
3			188	594	735	836	937
4				364	506	607	708
5					38	139	240
6						68	169
7							83

Table 8. Summary of travel times.

	Current Time Interval		Cumulative Values	
Factor	Vehicle-Hours[a]	Passenger-Hours[b]	Vehicle-Hours[c]	Passenger-Hours[d]
Freeway travel time (normal)	24	34	362	524
Freeway travel time (unreserved)	185	188	2,263	2,316
Freeway travel time (reserved)	14	35	370	934
Input delay (normal)	1,201	1,690	7,102	9,990
Input delay (unreserved)	110	158	1,874	2,694
Total travel time under priority operations	1,535	2,106	11,972	16,458
Total travel time under nonpriority operations	-	-	2,190	3,175
Travel time savings over nonpriority operations	-	-	-9,781.5	-13,283.0

[a]Total travel distance = 5,168 vehicle-miles.
[b]Total travel distance = 6,816 passenger-miles.
[c]Total travel distance = 87,707 vehicle-miles.
[d]Total travel distance = 125,301 passenger-miles.

Figure 1. Priority-lane operations evaluation procedure.

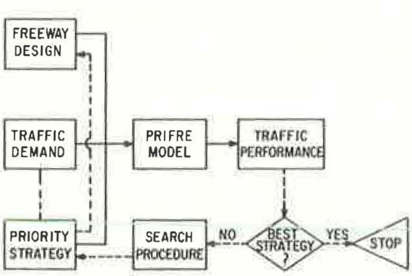
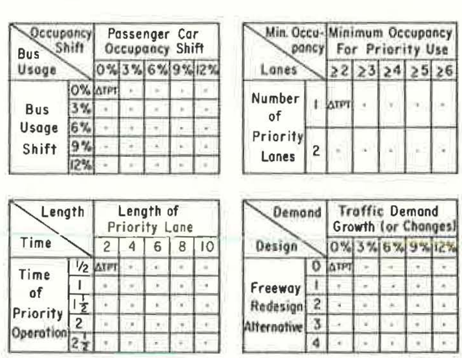


Figure 2. Measures of effectiveness data.



ΔTPT = Total Passenger Travel Time Difference

Along with these model refinements, extensive field research is needed to investigate priority-lane operations. Much can be learned from factual evaluations of existing and planned priority-lane demonstrations around the country. Also in the areas of education, safety, and law enforcement, research is needed to overcome the many present objections to priority-lane operations. Much information is needed on forecasting possible or probable occupancy shifts based on the user's perceived value of time and cost savings.

With more than 15 priority-lane projects now in operation around the country, it seems certain that much emphasis will be placed on evaluating these and other possible priority-lane operations in the next few years. Many new and diverse strategies will probably emerge. Traffic engineers and policy-makers will be called on to analyze and evaluate the strategy that best fits their particular area's needs. By using flexible, realistic computer simulation models, many more strategies and variations can be analyzed than would be possible if they were done by manual calculations. This diversity and sensitivity can only lead to a better insight and knowledge of possible priority strategies, and hopefully to sounder solutions, because more options can be considered for possible adoption and implementation.

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MODEL FOR IMPROVING TRAFFIC ENGINEERING DECISION-MAKING

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The purpose of this investigation was to examine the decision-making process by which traffic engineering decisions are made in urban areas and to develop a methodology that could be used to analyze and improve existing decision-making procedures. A conceptual model of decision-making was constructed that identified the traffic engineering decision-making system as consisting of three elements: traffic engineering functions, decision-making participants, and the decision-making process. The elements interact within boundaries established by a series of external constraints. The decision processes in 17 cities were reviewed through extensive personal interviews that were conducted with traffic engineers, city officials, and community leaders. On the basis of this analysis, it was concluded that the conceptual model provided a means of describing and evaluating decision-making techniques used by traffic engineers in the performance of their duties.

•IN recent years it has become apparent that there is a growing dissatisfaction with decisions that are made relative to the implementation of transportation projects in urban areas. The most striking manifestation of this discontent is the high degree of public opposition marshalled against proposals to construct large-scale projects such as new urban freeways.

The highly controversial decisions that result in open conflict between transportation professionals and organized citizen groups may be the most pronounced examples of breakdowns within the existing decision-making structure. Although these heavily publicized controversies command a sizable amount of attention, there has been considerable concern with the manner in which transportation decisions are made at all levels of government and within the private sector.

One agency having extensive transportation decision-making responsibility within a municipal government is the traffic engineering department. Originally, the functions of a traffic engineering agency were generally limited to the performance of traffic studies and the supervision of traffic regulations and controls. However, with the growth of urban automobile travel, these responsibilities have been increased to include other specific duties such as the operation of on- and off-street parking programs, planning and design of traffic facilities, and street lighting. These functions may vary from city to city because of legislative authority and resource availability but may also include the responsibility for coordinating efforts with other organizations doing traffic engineering work such as state and county highway departments and planning agencies. Also, all of these agencies may play a prominent role in the formulation and implementation of urban transportation policy.

The difficulties encountered by traffic engineers in solving day-to-day operational problems, such as parking and traffic control, are indications of possible failures in

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the decision-making process. One need only to observe the furor that occasionally surrounds so-called minor improvements, such as intersection widenings and removal of on-street parking, to see the dilemma confronting the engineer.

The need for reviewing this decision-making process in traffic engineering agencies was discussed in an article by Baerwald (1). He stated,

> Decision-making by government increased with travel growth, coupled with the inconsistency of private regulations and practices due to the wide variation evidenced in individual decision-making. The present trend is for higher levels of government to assume jurisdiction as our problems increase and the need for uniformity and more sophisticated procedures grows. Traffic engineering decisions today are made by a variety of people. The most common is action by elected officials. This action may be made with or without the recommendations of professional persons and groups. We also find action by appointed officials who have varying degrees of professional competence and experience. Sometimes we find that traffic engineering decisions are made by private groups or persons.

In addition to the frustration felt by traffic engineers when their recommendations based on professional expertise are ignored by elected officials whose concerns may be based on political considerations, there are other indications that the decision-making process may fail to generate satisfactory solutions. For example, engineers or other technical personnel may not consider important the socioeconomic or political implications of their decisions. Also, disagreements over the "best" solution to a traffic problem may be generated between engineers and professionals of other disciplines such as urban planning, architecture, or sociology or between engineers representing separate governmental agencies charged with transportation responsibilities.

The patterns of conflict previously mentioned may cause one to conclude that perhaps a point has been reached where it is not feasible to develop a satisfactory solution to a traffic problem. It may be that the environment in which decisions must be made is such that satisfaction of all groups concerned is not possible. Citizens may feel that they have been saturated with engineering recommendations and react by rejecting all plans without weighing their merits. Professionals may sense that they have been ignored by public officials, and public officials may find their well-defined, economically sound programs received with public frustration and bitterness instead of the anticipated enthusiasm.

A second, more optimistic conclusion may be reached regarding the existing dilemma in traffic engineering decision-making. This conclusion is that the professionals in engineering and other disciplines do not fully understand the workings of the decision-making process in urban government. A knowledge of the manner by which the process proceeds is necessary in order for the engineer to formulate effective strategies, strategies that will take into account the environment in which the decision is made and will lead to satisfactory solutions.

This paper, part of a more extensive research investigation (2), examines the decision-making process that is followed in urban areas to reach decisions on traffic engineering improvements.

STUDY OBJECTIVE AND SCOPE

The objective of the study was to develop a conceptual model representing the process by which traffic engineering decisions are made in urban areas. This model identifies the types of traffic engineering problems encountered by municipal governments and the process (or processes) by which these problems are resolved. Several key elements were considered in the construction of this model, such as the identification of the participants in the decision-making process, their roles, both formally as members of organizations and informally as individuals within the structure, and their contribution to the resolution of the problems under consideration. Other factors, such as the socioeconomic and political environments within which these decisions are made and the information flow within the process, were also considered.

The study focused on the role of the traffic engineer and the municipal traffic engineering department in the local government decision process. It was confined to 17 selected cities that had a full-time professional traffic engineer. The cities sampled were located in six states: Illinois, Indiana, Iowa, Michigan, Missouri, and Wisconsin. The population of these cities ranged from 40,000 people to 900,000 people as follows:

City Population	Number of Cities
500,000 to 900,000	2
100,000 to 500,000	8
50,000 to 100,000	6
<50,000	1

Traffic engineering decisions made by state highway departments and other governmental and private agencies were only considered insofar as they required the involvement of the city traffic engineer.

DEVELOPMENT OF THE MODEL

The analysis of a situation as complex and dynamic as governmental decision-making at the municipal level required an approach that has the capabilities to define and evaluate the numerous factors that influence the decision process. A conceptual model of traffic engineering decision-making was constructed that describes the decision-making process. It was essential that a method be developed that organizes the key variables influencing decision-making into logical patterns for the purpose of analysis.

Viewing a complex system at different levels required the construction of a hierarchy of models. At the top level was a coarse model identifying the major components of decision-making. Upon review of this model, secondary models of finer resolutions were built to examine the interactions between the elements of the major components. Finally, a third level of models was developed. These models were focused on the critical managerial functions undertaken during decision-making. This hierarchy of models is shown in Figure 1. Thus, the final model is in reality a system of nested models; each succeeding model is of a finer resolution than the one preceding it.

The initial step in the modeling process was the identification of the major components of decision-making. A procedure for classifying influencing factors was developed by viewing decision-making as a system of actions that can be grouped under several interacting subsystems within the overall systems framework.

The decision-making system can be represented by three interconnected subsystems operating within bounds established by a series of external or environmental factors. These subsystems are designated as the system of traffic engineering functions, the system of participants, and the process system. Figure 2 shows the relation among these three elements and the environment in which they exist.

The functions subsystem contains those traffic engineering activities performed in a community. It includes task-oriented functions such as traffic data collection, traffic operations, and traffic planning as well as the necessary administrative responsibilities necessary to carry out the work.

The system of participants includes those individuals and groups either who are involved in or who influence traffic engineering decision-making. This may include representatives of federal, state, and local governments, business leaders, organized citizens' groups, and individual citizens.

The decision-making process describes a series of distinct phases involved in reaching a decision. As shown in Figure 3 these phases are perception and identification of problem, interpretation of problem, analysis, evaluation and choice of alternatives, and implementation.

These steps are usually carried out in sequential order, but feedback to previous phases is an important aspect of the process. This feedback can be initiated at any step. For example, a project may be halted during implementation and returned for a reevaluation of alternatives, further analysis, reinterpretation of the problem, or reidentification of the problem. During the evaluation and choice of alternatives phase, a request may be made for additional analysis, for a reinterpretation of the problem, or for a reindentification of the problem. Likewise, during analysis, proposals may

Figure 1. Hierarchy of models.

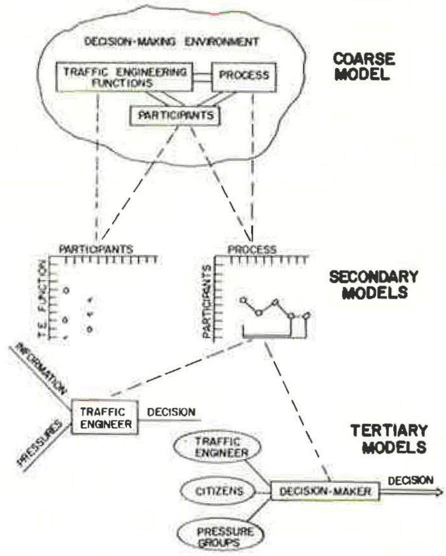


Figure 2. Traffic engineering decision-making system.

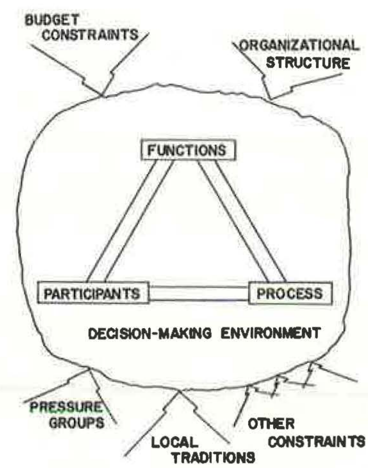
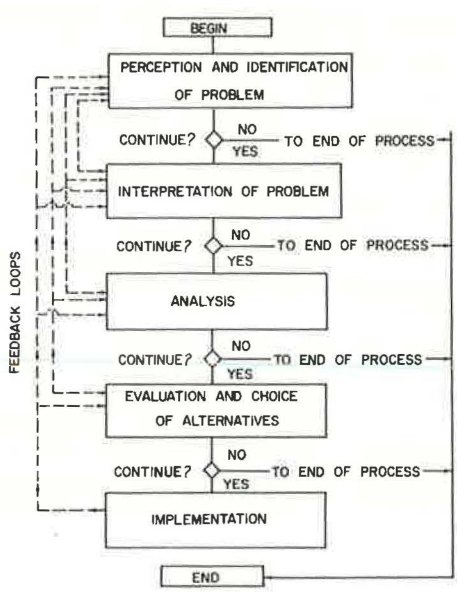


Figure 3. Decision-making process.



be returned to either of the two previous phases. During the interpretation of the problem, a request may be made to clarify the identification of the problem. After each step in the process, a decision is made either to continue or to end the process. If a decision is made to end the investigation, the process is stopped. If a decision is made to continue, the next phase of the process is begun.

By focusing on the functions performed during the decision-making process, the information obtained from examining the model's components could be synthesized. A functional approach to traffic engineering decision-making was undertaken to develop a means of constructing secondary and tertiary models of decision-making. This approach, used successfully in the analysis of construction management, was based on the assumption that the decision-making process can be described in terms of functions that can be related in a structure that ranks them in order of their importance.

Table 1 (functions chart) gives the individual functions undertaken during decision-making. These functions are divided into four groups on the basis of their importance to the completion of the overall traffic engineering task. The most important category of decisions is given in column 1. This column includes those functions that are performed in establishing the city's traffic and transportation policies and procedures. Column 2 gives those functions of slightly lesser importance that relate to the establishment of policies and procedures on individual traffic or transportation projects or both. In column 3 the functions at a lower level of importance necessary for controlling or coordinating individual traffic or transportation projects or both are listed. Column 4 lists those functions that are the least important relating to the collection or distribution of information for traffic or transportation projects or both.

In each column the specific functions are organized according to importance from top to bottom of the column. In several cases there may be debate over the order assigned to the functions. When there was not a clear and obvious order of importance, the items were listed in chronological order as they were performed in decision-making. In column 3, for example, the functions listed are approximately of equal importance and, therefore, are listed according to their order in the process.

Table 1 was used to develop models at the secondary level in the hierarchy of models. These models examine the relation among the elements of the system's major components. Two models were constructed. The first was a chart relating the level of decision-making participation to the traffic engineering functions in a city. The purpose of modeling this relation was to illustrate the variations in decision-making involvement for the range of traffic engineering tasks in a community. This information can be used as a preliminary means of pinpointing the individuals who actively participate in the process and whose actions or opinions are important to the resolution of problems. This information can also indicate which traffic engineering tasks may be the most difficult to perform because of the increased complexity resulting from the involvement of numerous persons.

Table 2 is an example of the chart that illustrates the relations between decision-making participants and the traffic engineering functions in a community. The degree of involvement of each individual or group can be determined by using Table 1.

A participant's involvement in a particular traffic engineering task can be measured by using the four categories of functions in Table 1 to indicate the degree of involvement. Four levels of involvement are shown in Table 2:

1. Level 1—Perform at least one function in column 1 of Table 1 in addition to functions in columns 2, 3, and 4.
2. Level 2—Perform at least one function in column 2 (Table 1) as well as functions in columns 3 and 4.
3. Level 3—Perform at least one function in column 3 (Table 1) as well as functions in column 4.
4. Level 4—Perform one or more functions in column 4 (Table 1).

Table 2 shows that in one particular city the traffic engineer performs functions at the highest levels of involvement for all traffic tasks except street lighting programs. In addition, it is shown that the public sector, elected or administrative officials, and other governmental units have extensive responsibilities for planning decisions.

This matrix relates the functions performed by the decision-making participants for one type of traffic engineering responsibility in a community. A map illustrating the flow of the process and the actions of the participants in this flow can be constructed within this diagram (Fig. 4). This is accomplished by analyzing the functions performed by the various participants with the functions charts. By plotting each individual's actions and relating them to the actions of others, the decision process can be modeled. Because it focuses directly on the process, the model can be used to analyze decision-making situations and identify any elements in the process that can be modified or improved. Models can then be built to represent those elements identified as important for the completion of the process.

Figure 5 is a functions chart that has been drawn for the performance of one traffic engineering responsibility, the installation of a stop sign by a traffic engineer in a hypothetical city. The shaded areas represent those functions performed by the engineer. Figure 4, which shows the possible paths that the decision process can follow in this hypothetical city, was constructed using Figure 5 and similar charts for other participants in the process.

For example, the first step in the process, the initial perception, can be initiated by those in the public sector, elected or administrative officials, the city engineer, the police, or the traffic engineer. This perception can then be communicated to the traffic engineer in a number of ways. A citizen may make his request for a traffic improvement directly to the traffic engineer, or he may submit his request to a citizen's group or to a city elected or administrative official. These individuals would then submit these requests to other intermediaries or to the traffic engineer. Figure 4 shows these communications paths in the perception and identification phase. The remaining steps in the decision process are performed by the traffic engineer because he has been granted complete authority by the city council to carry out this function.

During the second phase of the process, the traffic engineer determines a method of handling the request and decides if a study is needed. If the request does not merit further study, he informs the initiator of his decision. If he decides a study is necessary, he proceeds to the next phase of the process and completes the analysis. At this point, he decides if a change is warranted. If he decides a change is not warranted, he can either conduct further study, redefine the problem and begin a new investigation, or inform the person who initiated the request that he has determined the control device to be unnecessary. If his analysis indicates that the device is warranted, he can generate appropriate action. In this case, he can examine the possible installation of two-way or four-way stop signs. In generating viable alternatives and developing an acceptable solution, he weighs the desires of the police, city officials, and public sector. He subsequently may decide not to install a new device and initiate feedback, or he can implement the change. Although Figure 4 represents a relatively uncomplicated decision process, diagrams for the performance of decision-making under more complicated conditions can be constructed using the same procedures.

On the basis of the insights provided by the model relating the decision-making participants to the process, it is possible to use the functions charts to determine factors for further analysis. The modeling process at the third level is focused directly on the interactions among individuals. The second level of modeling indicated the constraints within the environment or within the formally defined process that influenced the outcome of a decision. At the third level, the influence of the individual's desires and actions on the final decision can be determined.

Figure 6 is an example of the type of model that could be constructed at this level. This model, showing the initial step in the evaluation and choice of alternatives, was constructed from the functions charts of individuals in another hypothetical city. The traffic engineer had the responsibility for carrying out this step, but he had to weigh the pressures from individuals and groups from the public sector, desires of elected officials, and desires of the police in evaluating alternatives to determine the need for a stop sign. Each of these sources of input has some interest it is attempting to protect. The engineer's task is to evaluate the importance of all these considerations along with the other information (the results of analysis, the needs of the road user, and his professional judgment) available to him.

Table 1. Traffic engineering decision-making functions chart.

City Traffic and Transportation Policy and Procedure (1)	Individual Traffic and/or Transportation Project Policy and Procedure (2)	Control or Coordinate Individual Traffic or Transportation Project (3)	Information Collection or Distribution (4)
Determine city traffic or transportation policy or both	Make final decision on implementation	Determine scope of study	Perceive problem or request traffic engineering assistance
Define scope of traffic engineering activities	Make intermediate decision on implementation	Assign responsibility for study	Communicate problem to city official
Determine decision-making procedures for traffic engineering projects	Determine if funds are available to implement project	Supervise study	Communicate problem to traffic engineer
Set traffic engineering budget	Assign priorities to examine requests for traffic engineer study or assistance	Coordinate study	Present recommendations to decision-makers
Assign priorities for implementing traffic engineering projects	Identify a request as warranting action by the traffic engineer	Perform study	Advise decision-makers
Determine qualifications for hiring traffic engineer	Determine if a traffic engineering study is needed	Analyze data	Maintain traffic engineering records
Hire traffic engineer	Determine if results of study dictate that change is warranted	Supervise implementation	Collect data
Determine qualifications for hiring traffic engineering staff	Evaluate alternative designs	Coordinate implementation	Communicate results of decision-making process to initiator of request
Hire traffic engineering staff			Use newspapers and other news media to conduct public relations
			Attend city council meetings
			Maintain informal liaison with city officials
			Maintain liaison with newspaper reporters or representatives of other news media
			Attend citizens' group meetings
			Attend meetings of business groups
			Attend meetings of service clubs

Figure 4. Decision-making process for installation of traffic signal.

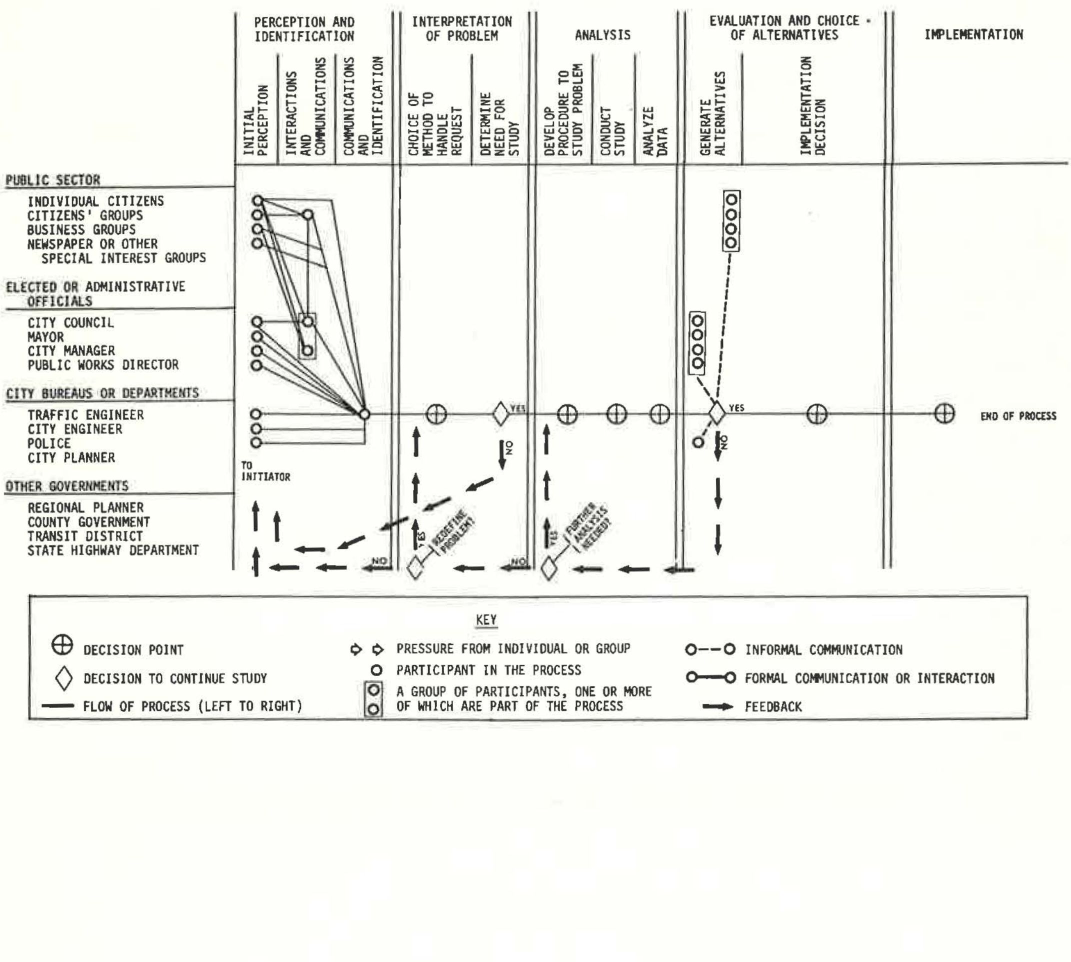


Table 2. Relations between traffic engineering functions and decision-making participants.

Function	Public Sector				Elected or Administrative Officials			
	Individual Citizens	Citizens' Groups	Business Groups	Newspapers or Other Interest Groups	City Council	Safety Committee of Council	Mayor	Public Works Director
Collection and analysis of traffic data								
Conduct traffic surveys and studies					4	4		
Conduct accident studies					4	4		
Traffic regulations								
Formulate regulations	4	4	4	4	1	1	4	4
Regulate street uses	4	4	4	4	1	1	4	4
Control parking	4	4	4	4	1	1	4	4
Regulate street use by transit	4	4	4	4	1	1	4	4
Control devices								
Determine need for and design signals	4	4	4	4	1	1	4	2
Install, operate, and maintain signals	4	4	4	4	1	1	4	2
Determine need for and design signs and markings	4	4	4	4	1	1	4	2
Install, operate, and maintain signs and markings	4	4	4	4	1	1	4	2
Operate parking control devices	4	4	4	4	1	1	4	2
Street lighting								
Design	4	4	4	4	1		4	1
Install					1		4	1
Operate and maintain	4	4	4	4	1		4	1
Traffic design								
Design intersection or other improvements	4	4	4	4	1	1	4	1
Design off street parking facilities	4	4	4	4	1	1	4	1
Traffic planning								
Long range	4	1	1	1	1	1	1	1
Short Range	4	1	1	1	1	1	1	1
Street regulations	4	4	4	4	1	1	1	1

Function	City Bureaus or Departments				Other Governments and Organizations			
	Traffic Engineer	City Engineer	Police	City Planner	Regional Planner	State Highway Department	County Govern- ment	Transit Agency
Collection and analysis of traffic data								
Conduct traffic surveys and studies	1							
Conduct accident studies	1		4					
Maintain or have access to accident records	1		4					
Traffic regulations								
Formulate regulations	2	4	4					
Regulate street uses	2	4	4					
Control parking	2	4	4					
Regulate street use by transit	2	4						
Control devices								
Determine need for and design signals	2	4	4			1[a]		
Install, operate, and maintain signals	2	4	4			1[a]		
Determine need for and design signs and markings	2	4	4			1[a]		
Install, operate, and maintain signs and markings	2	4	4			1[a]		
Operate parking control devices	2	4	4			1[a]		
Street lighting								
Design	4	2						
Install	4	2						
Operate and maintain	4	2						
Traffic design								
Design intersection or other improvements	2	2	4			1[a]		
Design off street parking facilities	2	2	4					
Traffic planning								
Long range	1	2		1	1	1	1	
Short range	1	2		1	1	1	1	
Street regulations	1	2		1				

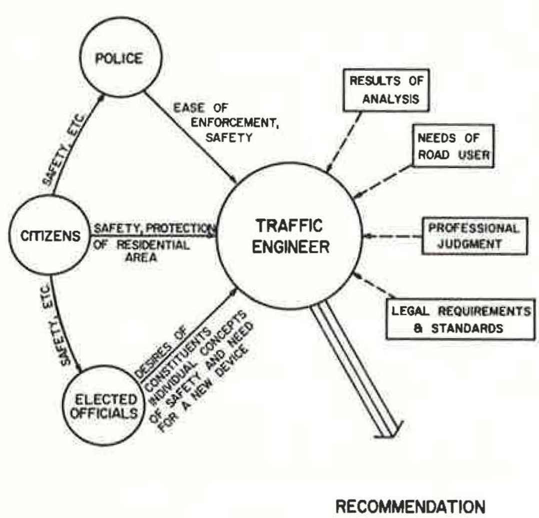
Note: 1 = set city traffic policy or procedure, 2 = set policy or procedure for individual traffic project, 3 = control or coordinate individual project, and 4 = information collection or distribution.

[a]On state highways.

Figure 5. Functions chart for installing traffic signal.

CITY TRAFFIC AND TRANSPORTATION POLICY AND PROCEDURE	INDIVIDUAL TRAFFIC AND/OR TRANSPORTATION PROJECT POLICY AND PROCEDURE	CONTROL OR COORDINATE INDIVIDUAL TRAFFIC OR TRANSPORTATION PROJECT	INFORMATION COLLECTION OR DISTRIBUTION
DETERMINE CITY TRAFFIC AND/OR TRANSPORTATION POLICY	MAKE FINAL DECISION ON IMPLEMENTATION	DETERMINE SCOPE OF STUDY	PERCEIVE PROBLEM OR REQUEST TRAFFIC ENGINEERING ASSISTANCE
DEFINE SCOPE OF TRAFFIC ENGINEERING ACTIVITIES	MAKE INTERMEDIATE DECISION ON IMPLEMENTATION	ASSIGN RESPONSIBILITY FOR STUDY	COMMUNICATE PROBLEM TO CITY OFFICIAL
DETERMINE DECISION-MAKING PROCEDURES FOR TRAFFIC ENGINEERING PROJECTS	DETERMINE IF FUNDS ARE AVAILABLE TO IMPLEMENT PROJECT	SUPERVISE STUDY	COMMUNICATE PROBLEM TO TRAFFIC ENGINEER
SET TRAFFIC ENGINEERING BUDGET	ASSIGN PRIORITIES TO EXAMINE REQUESTS FOR TRAFFIC ENGINEER STUDY OR ASSISTANCE	COORDINATE STUDY	PRESENT RECOMMENDATIONS TO DECISION-MAKERS
ASSIGN PRIORITIES FOR IMPLEMENTING TRAFFIC ENGINEERING PROJECTS	IDENTIFY A REQUEST AS WARRANTING ACTION BY THE TRAFFIC ENGINEER	PERFORM STUDY	ADVISE DECISION-MAKERS
DETERMINE QUALIFICATIONS FOR HIRING TRAFFIC ENGINEER	DETERMINE IF A TRAFFIC ENGINEERING STUDY IS NEEDED	ANALYZE DATA	MAINTAIN TRAFFIC ENGINEERING RECORDS
HIRE TRAFFIC ENGINEER	DETERMINE IF RESULTS OF STUDY DICTATE THAT CHANGE IS WARRANTED	SUPERVISE IMPLEMENTATION	COLLECT DATA
DETERMINE QUALIFICATIONS FOR HIRING TRAFFIC ENGINEERING STAFF	EVALUATE ALTERNATIVE DESIGNS	COORDINATE IMPLEMENTATION	COMMUNICATE RESULTS OF Q-M PROCESS TO INITIATOR OF REQUEST
HIRE TRAFFIC ENGINEERING STAFF			USE NEWSPAPERS AND OTHER NEWS MEDIA TO CONDUCT PUBLIC RELATIONS
			ATTEND CITY COUNCIL MEETINGS
			MAINTAIN INFORMAL LIAISON WITH CITY OFFICIALS
			MAINTAIN LIAISON WITH NEWSPAPER REPORTERS OR REPRESENTATIVES OF OTHER NEWS MEDIA
			ATTEND CITIZENS' GROUP MEETINGS
			ATTEND MEETINGS OF BUSINESS GROUPS
			ATTEND MEETINGS OF SERVICE CLUBS

Figure 6. Tertiary model of interactions.



In addition to describing decision-making situations in detail, these models at the third level of the hierarchy provide a means for improving the process. For example, in Figure 6 it is apparent that citizens in this community exerted heavy pressure at several points in the municipal government for the installation of additional stop signs. Aware of this, the engineer can develop methods for reducing this pressure. He can work closely with the police and attempt to convince them that additional signs are not warranted and would be difficult to enforce. He can also try to educate the elected officials or the citizens themselves on the proper application of these devices. This example of a third-level model illustrated the type of information obtained from the model and its use.

UTILIZING THE MODEL TO IMPROVE TRAFFIC ENGINEERING DECISION-MAKING

To function effectively, the city traffic engineer must be aware of the many factors that influence the decision-making process. The traffic engineer performs his technical activities in a complex environment in which the desires of individual citizens, business leaders, elected representatives, and vested interest groups may plan an important role in determining how the final decision is reached on a traffic engineering problem.

The task confronting the traffic engineer is one of satisfying his objective function, subject to the constraints imposed by the environment in which he operates. This objective function is the engineer's concept of what should be done to improve the traffic movement and storage system in the community. Satisfying this objective is not always possible because other individuals view transportation from entirely different perspectives. Downtown businessmen, for example, ideally desire a transportation system that has unlimited high-speed access to the edge of the shopping district, restricted capacity in the downtown area so that potential shoppers can drive past their places of business and "window shop," and acres of free parking directly in front of their stores. Homeowners desire easy access to high-speed roadways to allow them to travel to and from places of work and shopping without delay, but they want their own immediate neighborhoods free from traffic. The traffic engineer must weigh these conflicting factors in determining solutions to traffic problems.

A definition of "good" decision-making is very difficult to construct because it depends on individual concepts of what constitutes a good traffic engineering improvement. Because of the dynamic nature of decision-making, it is even difficult to define good decision-making from the perspective of a single individual such as the traffic engineer. In some instances, the engineer may be willing to modify a proposal without demeaning his professional integrity to meet constraints imposed by interest groups and to increase the probability of receiving approval of altered programs. At other times, the traffic engineer will ignore the concerns of interest groups so that he can develop programs that are technically sound and are directed toward benefiting the community at large.

Good decision-making from the traffic engineer's perspective must be viewed in relative terms as a process in which he attempts to implement the best technical solution and to gain support for his programs in light of the constraints present when the decision is made. The models can assist the traffic engineer in attaining these goals by identifying the constraints that are present and by pointing out how the engineer can maximize his objective function.

For the engineer desiring a review of his decision-making system, the models can provide a framework that could be used to schematically describe the decision process in his community and his role in the process. He can begin by examining the decision-making functions he performs for each traffic engineering activity in the community by using the functions charts developed in the previous section. By constructing functions charts for the other decision-making participants in the municipality, he can build a matrix such as the one given in Table 2. For those tasks that the engineer has determined to be critical in the performance of his duties, the engineer can use the functions charts to develop models of the decision-making process using the format shown in

Figure 4. By identifying the points where decisions are made and pressures applied, the engineer can begin to analyze the specific steps in the process and determine if there are elements or procedures or both that can be improved.

DATA COLLECTION FOR MODEL EVALUATION

Information on traffic engineering decision-making was collected in 17 cities to evaluate the conceptual model as a tool for describing real-world behavior. Each city chosen had a full-time traffic engineer who was a member of the Institute of Traffic Engineers. Two different formats were used to collect the data. First, traffic engineers in each of the cities included in the survey were contacted and requested to return a mail questionnaire that was designed to collect the following types of data:

1. The type of city government;
2. The location of the traffic engineering unit within the overall governmental structure;
3. The organization, legal authority, and functions of the traffic engineering agency; and
4. The size of the traffic engineering agency in terms of manpower and budget.

The second and most fruitful means of data collection utilized was the personal interview. In addition to conducting a lengthy interview with the traffic engineer, approximately five to seven decision-making participants in each city were also interviewed. The list of these other individuals varied from city to city but included mayors, city managers, businessmen, city engineers, newspapermen, and other professionals.

The conceptual model of decision-making was used to structure a list of questions to be asked of the study participants. This list was designed to determine the strengths and weaknesses of the decision process in the communities being studied. Each person interviewed was asked a similar series of questions concerning his involvement in the various phases of the process. This list was deviated from only when the interviewer perceived that it would be fruitful to allow the person being interviewed to explain particular situations in detail.

Everyone interviewed was assured that the data were being collected in a confidential manner and that they would not be identified in the published results of the investigation. The responses to all questions were written down at the time of the interview and were later transferred to tape recordings. The purposes of taping the interviews were to record the interviewer's feelings regarding the tone of the interview and the degree of cooperation received and to facilitate data reduction.

Decision-making procedures in several cities were examined to determine if successful traffic engineering decision-making strategies could be identified.

By examining charts similar to Figure 4 constructed for a common function in a number of communities, it was possible to identify strategies that traffic engineers utilized to improve their decision-making performance. These strategies varied widely from city to city because traffic engineers altered their decision-making techniques to meet local constraints.

Although the limited sample size did not permit the drawing of conclusions regarding these strategies, several methods were used repeatedly by traffic engineers to improve their decision-making role. These methods included the following:

1. Anticipation of the impact of the proposed activity—A knowledge of the political situation and the groups or individuals affected by a decision can provide a means for the engineer to plan what he can actually hope to accomplish.
2. Awareness of the limits of political influence—By anticipating the support he can receive from elected officials or other decision-makers, the engineer can avoid making a recommendation that has little chance of being accepted. However, at times he may be required to stand behind his recommendations in the face of heavy political opposition to establish, on record, his professional position on a proposed change.
3. Willingness to sacrifice—The engineer must be aware of the potential compromises and trade-offs that may be necessary in order to get a program approved and implemented. In this case, the strategy of "losing a battle to win a war" has proved

to be an effective method in gaining support for a key program—so long as professional standards and ethics are not compromised. The engineer must also realize that he may not always receive the credit he deserves for improving transportation. He must be willing to let others take credit for his successful programs and occasionally accept the blame for the failures of political officials.

4. Encouragement of involvement in decision-making—At times, it may be necessary to enlist the participation of key government officials or citizens in the decision-making process, even though the contributions of these individuals would not substantially affect the outcome of the final decision. By creating the feeling among these individuals that their opinions are important in setting transportation policy, the engineer may be able to gain their support for other projects where their influences could be a critical factor in determining if the engineer's recommendations are accepted.

5. Adoption of modern management techniques to increase the efficiency of the traffic engineer's department—By operating more efficiently, the traffic engineer will be better prepared to serve the public's needs.

CONCLUSIONS

The following conclusions were drawn from the results obtained in this study:

1. Using systems concepts, it was possible to develop a generalized modeling method to describe the traffic engineering decision-making process in urban areas;
2. When tested with data obtained from a survey of traffic engineering decision-making in 17 cities, the models provided a method of describing this type of decision-making behavior in local governments;
3. The models have direct practical application because they can be used by traffic engineers to structure and analyze decision-making situations to determine strategies that can be utilized to improve the probability of gaining acceptance or support for their programs; and
4. The data collection techniques developed for this project, which included extensive personal interviews and the use of mail questionnaires, provided a feasible method of gathering the information necessary to describe and analyze the progress of traffic engineering decision-making in a community.

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